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DIVISION DE EDUCACION CONTINUA

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Las autoridades de la Facultad de Ingeniería, por conducto del jefe de la División de Educación Continua, otorgan una constancia de asistencia a quienes cumplan con los requisitos establecidos para cada curso.

El control de asistencia se llevará a cabo a través de la persona que le entregó las notas. Las inasistencias serán computadas por las autoridades de la División, con el fin de entregarle constancia solamente a los alumnos que tengan un mínimo de 80% de asistencias.

Pedimos a los asistentes recoger su constancia el día de la clausura. Estas se retendrán por el periodo de un año, pasado este tiempo la DECFI no se hará responsable de este documento.

Se recomienda a los asistentes participar activamente con sus ideas y experiencias, pues los cursos que ofrece la División están planeados para que los profesores expongan una tesis, pero sobre todo, para que coordinen las opiniones de todos los interesados, constituyendo verdaderos seminarios.

Es muy importante que todos los asistentes llenen y entreguen su hoja de inscripción al inicio del curso, información que servirá para integrar un directorio de asistentes, que se entregará oportunamente.

Con el objeto de mejorar los servicios que la División de Educación Continua ofrece, al final del curso deberán entregar la evaluación a través de un cuestionario diseñado para emitir juicios anónimos.

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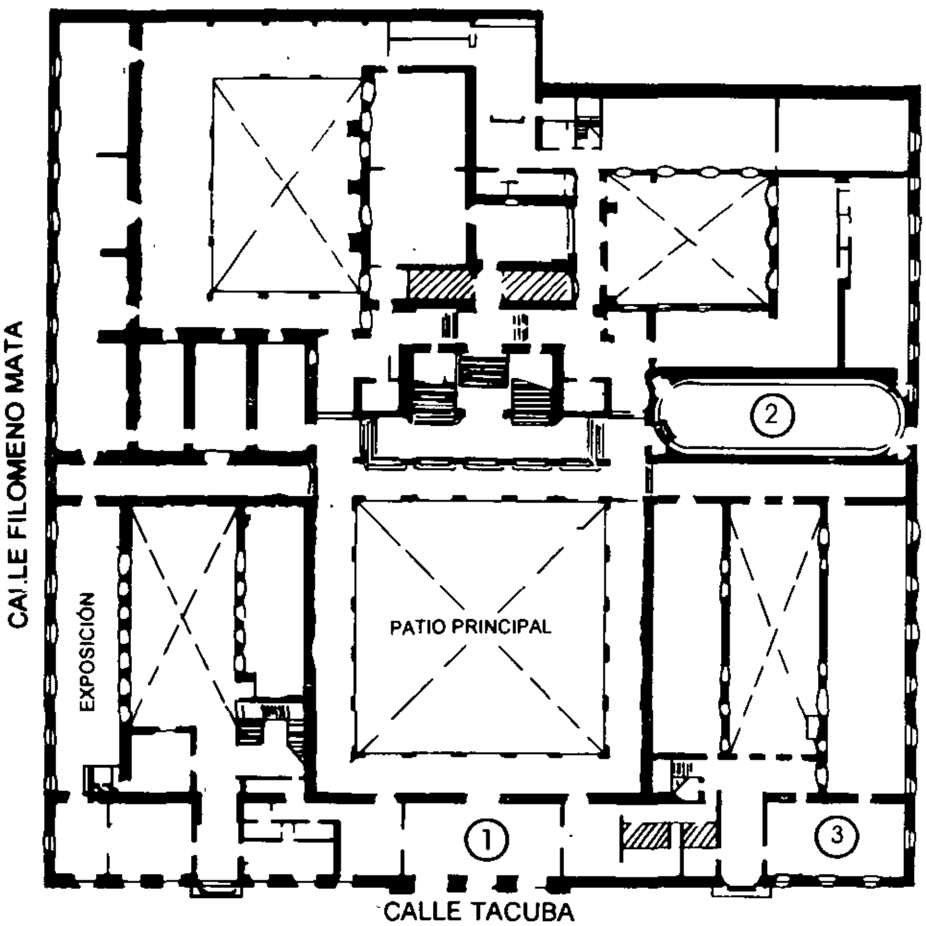

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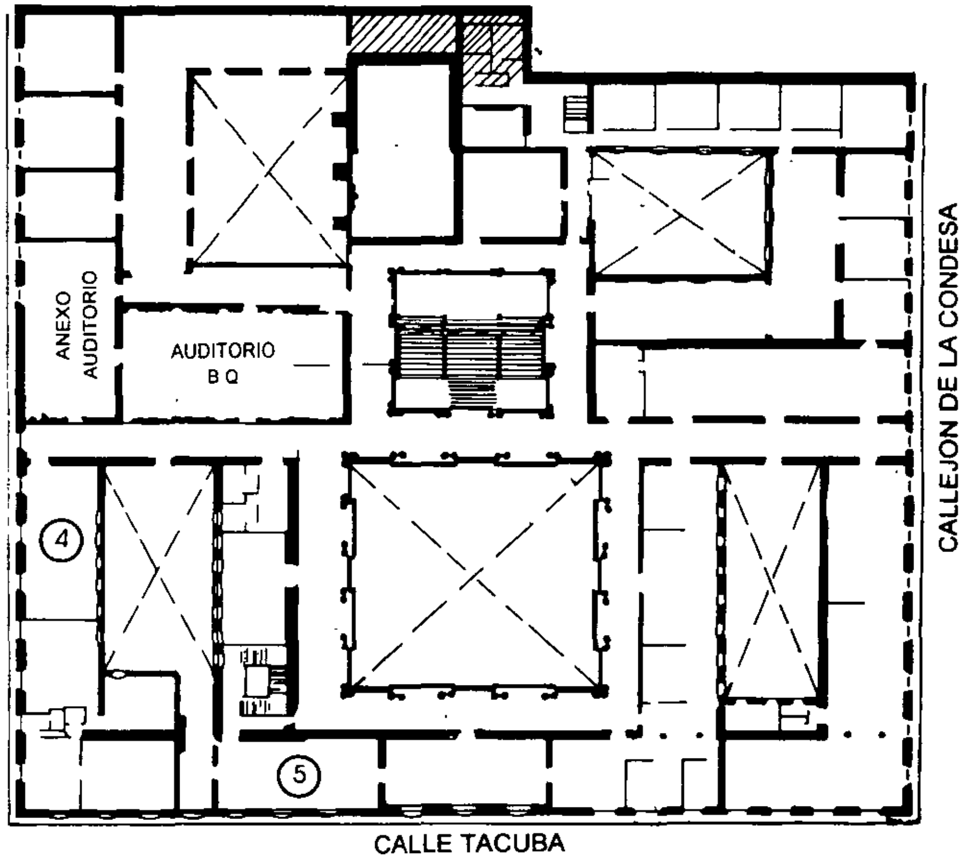

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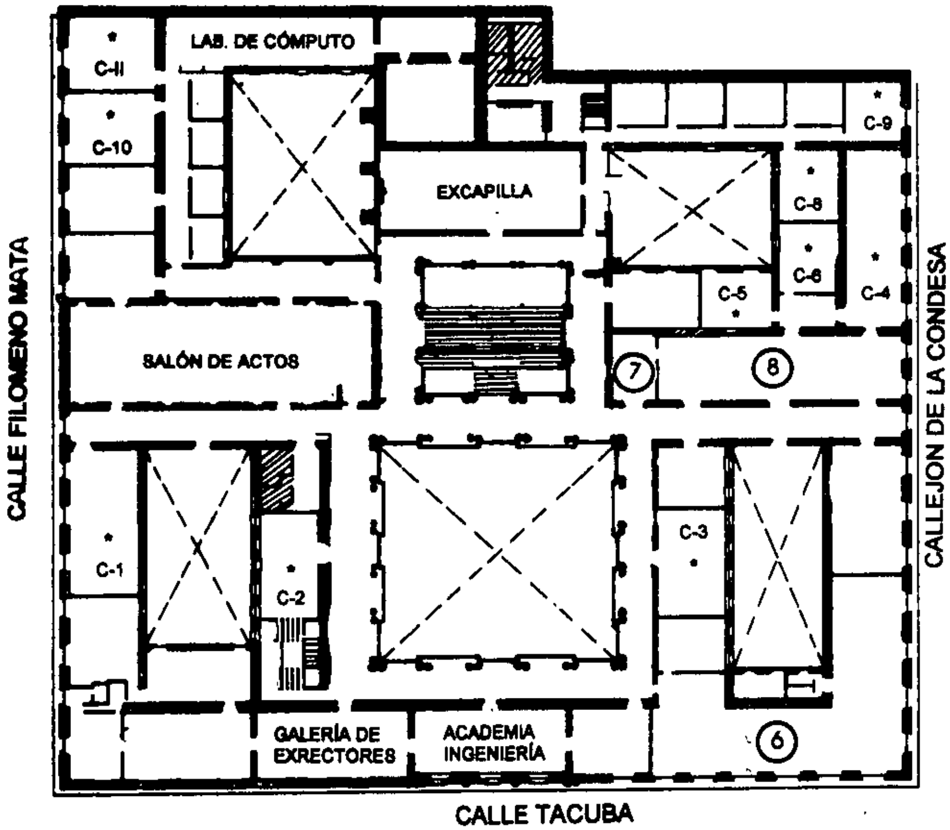

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FACULTAD DE INGENIERIA U.N.A.M.
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CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

SOLDADURA

ING. MARIO BARRETO MORALES
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

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SOLDADURA

TEORIA Y PRACTICA DE LA SOLDADURA CON OXIACETILENO

DEFINICION DE SOLDADURA

Es un proceso de trabajo en el cual los metales son llevados hasta su punto de fusión por medio del calentamiento para conseguir una nueva unión muy resistente.

DIFERENTES TIPOS DE SOLDADURA Y CORTE

Los tipos más frecuentes de soldadura son:

a) Soldadura con Gas
b) Soldadura de Arco
c) Soldadura de Arco y Gas
d) Soldadura de Resistencia

Otros tipos de soldadura más especiales son:

e) Soldadura atómica y de Hidrógeno
f) Soldadura de Termita
g) Soldadura Fría
h) Soldadura Ultrasónica
i) Soldadura de Capa de Electrones
j) Soldadura de Fricción
k) Soldadura de Laser
l) Soldadura de Plasma

Los tipos frecuentes de corte térmico son:

a) Corte de Gas
b) Corte de Arco

El proceso Oxígeno-Acetileno debe estudiarse primero debido a:

1) Los fundamentos de la soldadura con gas incluyen fundamentos importantes

para la mayoría de las otras formas de soldadura.

2) El proceso Oxígeno-Acetileno es un proceso manual, lento y fácil de controlar en comparación con otros procesos.

CARACTERISTICAS DE LA FLAMA DE OXIACETILENO

Para obtener la flama con más alta temperatura, mayor limpieza y mayor aportación de calor, es necesario utilizar el equipo que suministre, oxígeno puro para facilitar el proceso de oxidación.

Para realizar una buena soldadura se deben cumplir las siguientes consideraciones:

a) La temperatura de la flama debe ser suficientemente alta para fundir los metales.

b) Debe suministrarse suficiente calor para compensar las pérdidas de calor.

c) La flama no debe "quemar" (oxidar) el metal.

d) La flama no deberá aportar polvo ó materiales extraños a el metal.

e) La flama no deberá aportar carbón al metal.

f) Los productos de combustión no deberán ser tóxicos (venenosos).

La cantidad de calor se determina por la cantidad (pies cúbicos por hora) de gases quemados. Para obtener más calor el orificio de la boquilla deberá ser más grande y se suministrará gas a mayor presión en la boquilla. Si se usa una boquilla grande o una chica, como quiera que sea la temperatura de la flama es la misma. Debe recordarse que la cantidad de calor generado y por lo tanto el espesor del metal el cual puede ser soldado, dependera de la cantidad de gas combustible quemado por unidad de tiempo. Por consiguiente la cantidad de calor depende del tamaño del orificio de la boquilla.

Existen varios gases que se usan comercialmente para producir flamas para soldadura y corte:

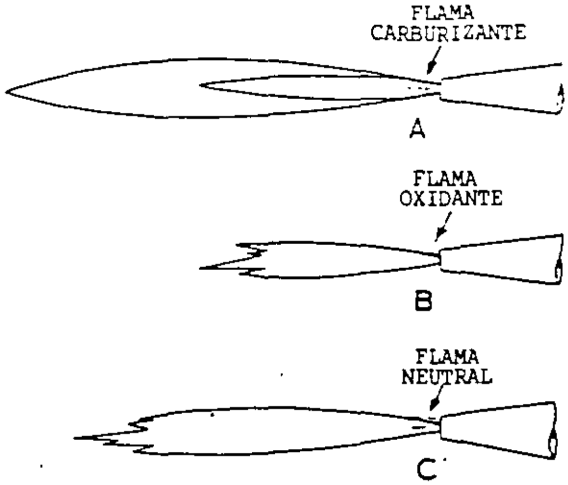
1) El Oxígeno-Acetileno
2) El Oxígeno-Hidrógeno

3) El Oxígeno-Gas Natural

4) El Oxígeno-Gas de Petróleo licuado

Flamas oxidizantes ó carburizantes no deseables se presentan cuando las proporciones de Oxígeno y Acetileno son erróneas. Si se utiliza más Acetileno se forma una flama carburizante.

La siguiente figura muestra los tipos de flamas:



La denominada FLAMA NEUTRAL es la correcta y no carburiza ni oxidiza el metal. Esta FLAMA NEUTRAL es el resultado de una perfecta proporción y mezcla de oxígeno y acetileno. Estos dos gases se unen de tal manera que el oxígeno quema el carbón y el hidrógeno en el acetileno y libera únicamente calor y gases poco dañinos.

El hollin en una soldadura puede provenir de dos fuentes:

a) Polvo en los Gases
b) Polvo en los equipos

Debe checarse siempre la calidad de la pureza de los gases.

La flama neutral de oxiacetileno puede tener una temperatura de 5600° a 5900° F. Una flama oxidizante producirá una temperatura ligeramente mayor.

A continuación se enlistan varios metales y su temperatura de fundición:

METAL	PUNTO DE FUNDICION °F
Aluminio	1215

Latón	1640
Bronce	1650
Cobre	1920
Fierro, fundición gris	2200
Plomo	620
Acero (0.20%C) SAE 1020	2800
Zinc	785

La siguiente tabla de tamaños de varillas de soldadura y tamaños de boquillas para soldar varios espesores de metal, nos da datos aproximados para obtener resultados satisfactorios.

ESPESOR DEL METAL	DIAMETRO DE LA VARILLA DE SOLDADURA	TAMAÑO DE LA BOQUILLA	PRESIONES OXIGENO	PRESIONES ACETILENO
1/16	1/16 - 3/32	60 - 69	4	4
1/8	3/32 - 1/8	54 - 57	5	5
1/4	5/32 - 3/16	44 - 52	8	8
3/8	3/16 - 1/4	40 - 50	9	

Para desconectar el maneral. Si el operador desea suspender el uso del equipor por unos minutos, será únicamente necesario cerrar las válvulas del maneral y dejar este a un lado. Sin embargo, si el equipo no va a hacer usado inmediatamente, el equipo deberá ser totalmente desconectado, por lo que deberá hacerse lo siguiente:

1) Cierre las válvulas del maneral y preferentemente la de Acetileno primero.

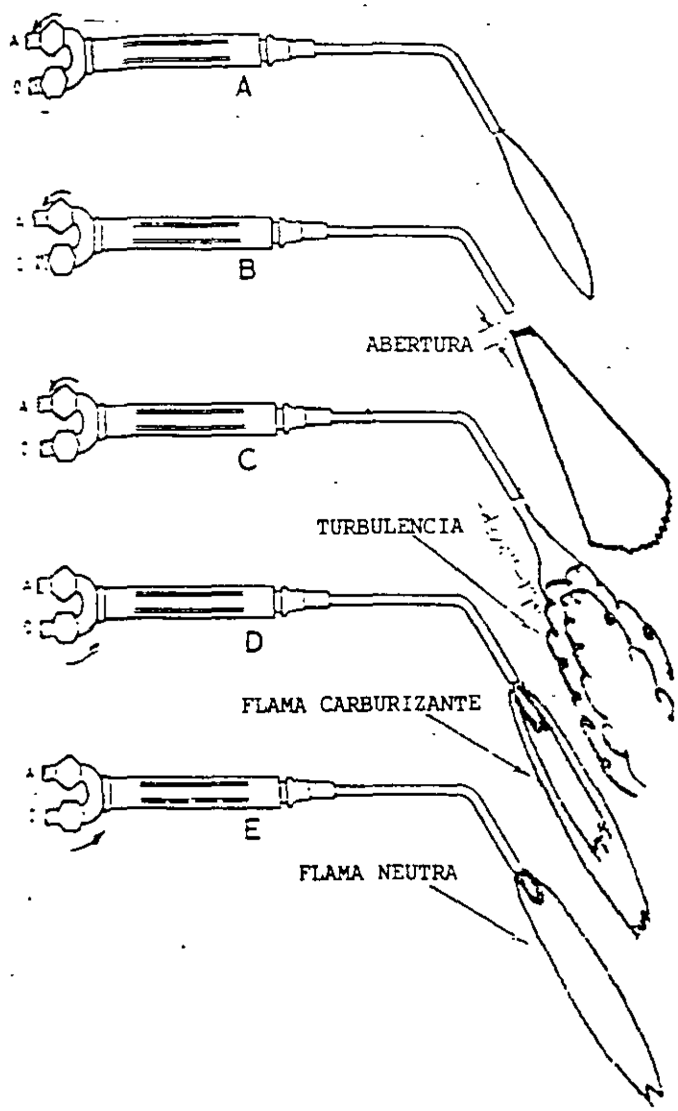
2) Cierre suavemente las válvulas de los cilindros.

3) Abra las válvulas del maneral.

4) Espere hasta que los manómetros de alta y baja presión de los reguladores de oxígeno y acetileno, indiquen cero.

5) Gire las tuercas de ajuste del Oxígeno y Acetileno de los reguladores -- hasta que esten "cerradas".

6) Cierre suavemente las válvulas del maneral y cuelgue el mismo.



Pasos recomendados para encender el maneral para soldadura con Acetileno. - **A.** Abra la válvula de Acetileno lentamente y encienda el Acetileno con un - encendedor de chispa. **B.** La cantidad correcta de Acetileno esta fluyendo si la flama inicia lejos de la boquilla cuando el maneral esta hacia abajo. o, **C.** Como se muestra aqui, una turbulencia es creada en la flama de Acetileno y el humo es eliminado. **D.** Comience a girar la válvula para el oxígeno. **E.** Continúe girando la válvula para el oxígeno hasta que la mitad de la flama esta eliminada y un cono interior redondeado aparece.

POSICIONES Y MOVIMIENTOS DEL MANERAL

El maneral debe ser mantenido a un ángulo de 30 a 45º grados con relación - al plano de trabajo. Lo anterior también dependerá del tamaño de boquilla - usada, el espesor del metal y otras condiciones para soldaduras. La flama - se coloca sobre el plano de trabajo en la dirección de la soldadura, precalentando el metal antes de que este bajo la flama de alta temperatura.

FUNDICION DEL METAL BASE ("TORCHEO O CALDEO")

Antes de iniciar cualquier clase de soldadura, se recomienda que el principiante practique la fundición del metal base que en lo sucesivo denominaremos "caldeo". El "caldeo" es una importante y fundamental parte de la soldadura debido a que en la mayoría de las operaciones de soldadura un "caldeo de metal fundido" es llevado a través de la junta de las partes que serán - soldadas. Esta fundición se presenta realmente en la mayoría de las formas de soldadura tanto con gas como con arco eléctrico. Las características del "caldeo de metal fundido" indican la penetración, ajuste del maneral, manejo y movimiento del maneral. Estas características del "caldeo" las cuales son juzgadas por medio de la observación, guían al soldador experimentado - para depositar una excelente soldadura.

El tamaño (diámetro) del "caldeo" estará con proporción a su profundidad; - por consiguiente el operador puede juzgar la profundidad o penetración de - una soldadura por la observación y control del tamaño del "caldeo de metal fundido". Sobre metales muy delgados la penetración o profundidad del "caldeo" será mayor en proporción a el ancho que en el caso de metales de grueso espesor.

La apariencia de la superficie de el "caldeo" indicará la condición de ajuste del maneral. La FLAMA NEUTRAL cuando esta fundiendo el metal dara una -- apariencia constante y uniforme del "caldeo". La orilla del "caldeo" lejos del maneral tendrá bolitas pequeñas brillantes e incandescentes las cuales se moveran activamente alrededor de la orilla del caldeo. Si estas bolitas estan sobredimensionadas la flama no es NEUTRAL. También si la soldadura de "caldeo", hierve y salpica excesivamente, puede ser que exista un ajuste pobre de la flama y se presente una mala calidad de soldadura en el metal que se esta soldando.

La flama en forma de cono interior redondeado debe estar colocada dentro de la región del "caldeo" en todo momento. El ajuste correcto de la flama evita que el oxígeno en la atmósfera entre en contacto con la superficie de la zona de "caldeo" y pueda causar una condición de oxidación.

En la gráfica se puede observar como llevar a cabo un correcto procedimiento de "caldeo".

TIPOS DE SOLDADURAS CON OXIACETILENO REALIZADAS CON VARILLA DE APORTE DE SOLDADURA

Las juntas básicas de soldadura son:

1) De ranura o abertura (Buttweld)

2) De filete (Fulletweld)

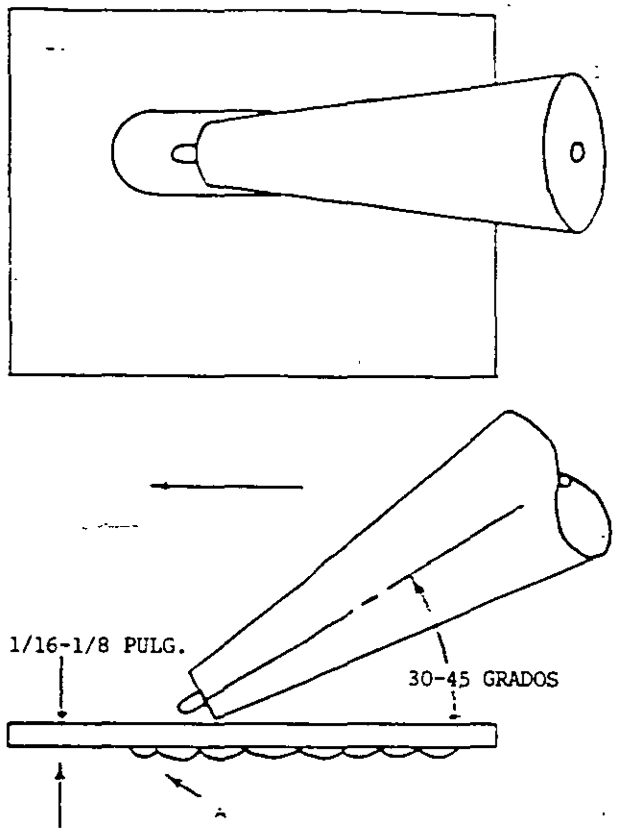
Las posiciones básicas para soldar son:

1) Horizontal sobre una superficie horizontal

2) Horizontal sobre una superficie vertical

3) Vertical sobre una superficie vertical

4) Sobrecabeza



Procedimiento para efectuar la soldadura por "caldeo de metal fundido":
Esta figura muestra la posición correcta del maneral en relación al metal - base durante un ejercicio. El detalle "A" muestra la penetración en la parte inferior del metal base.

Tipo de soldaduras realizadas sin el uso de varilla de aporte de soldadura. El tipo de soldadura mas común de este tipo es la que se muestra en la siguiente figura:

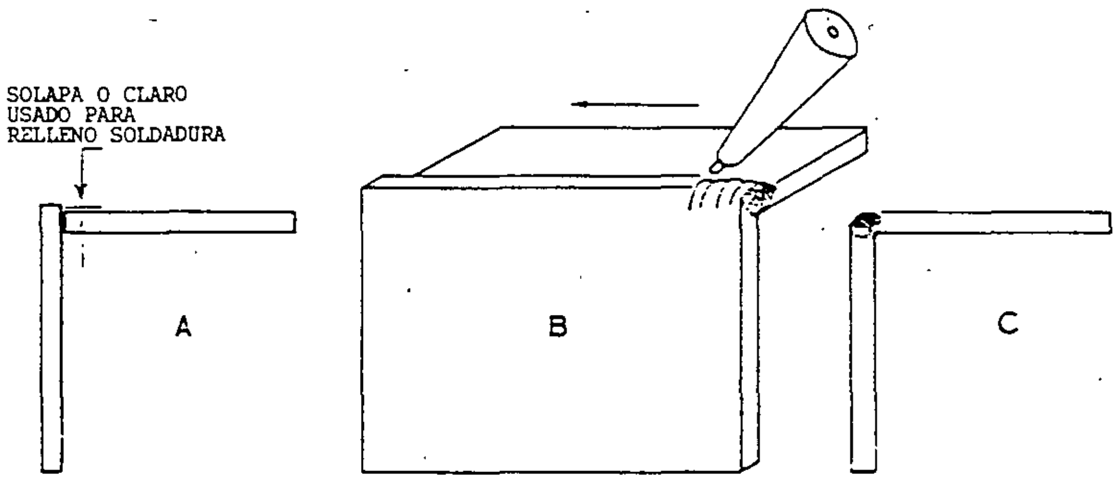


Fig. _____ Etapas en la ejecución de una junta de esquina exterior sin varilla de aporte de soldadura. A. Metal en posición para soldar. B. Soldadura en proceso. C. Apariencia de la soldadura terminada.

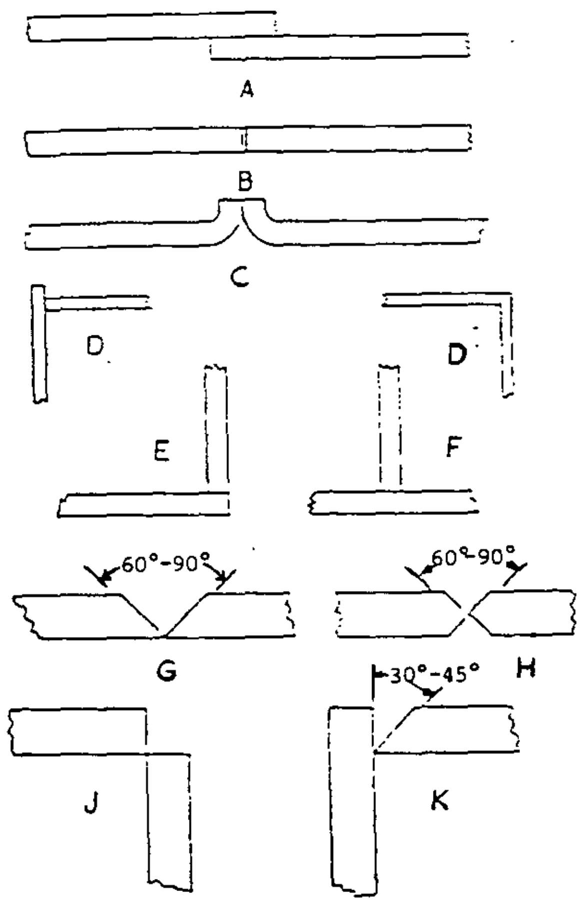
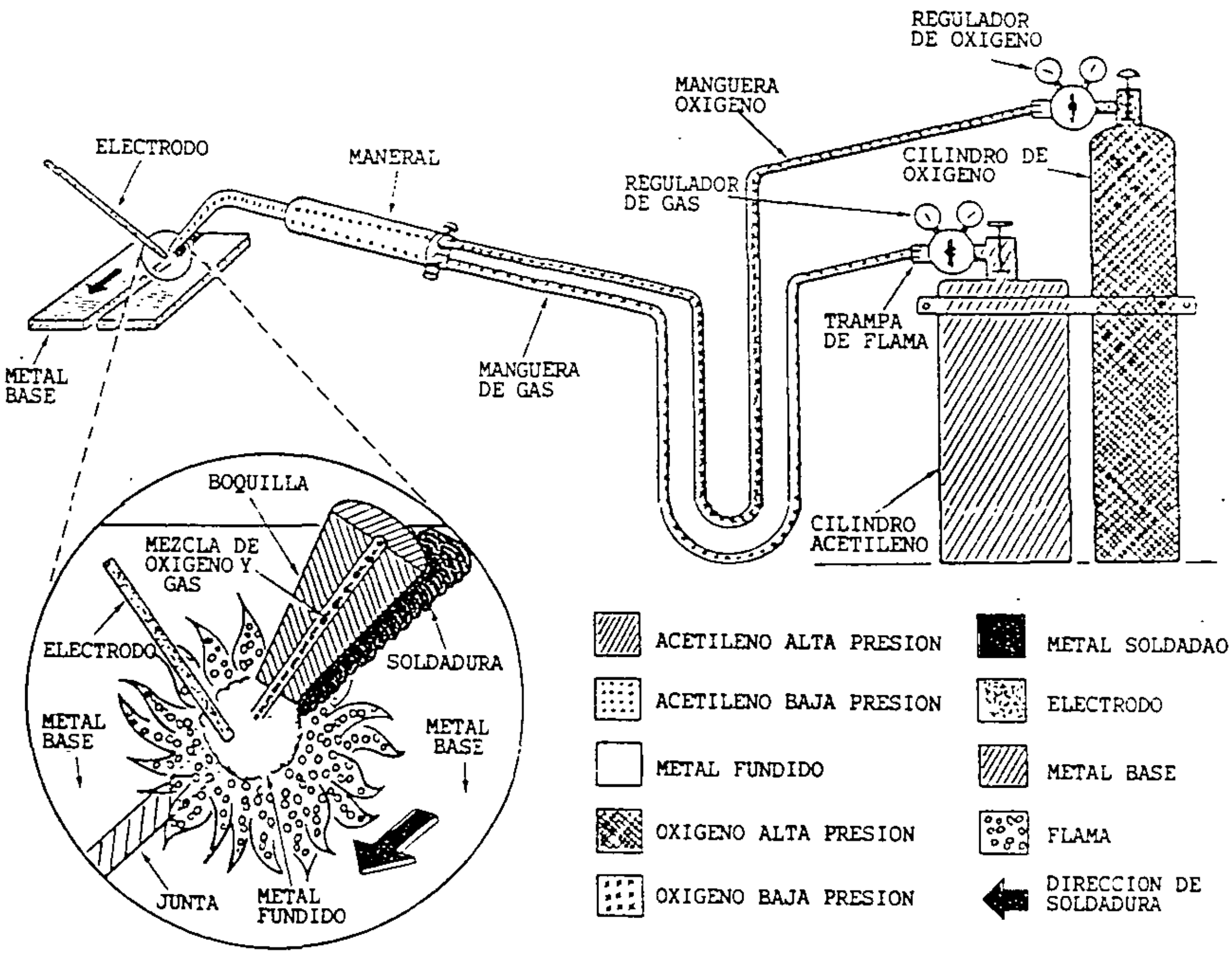
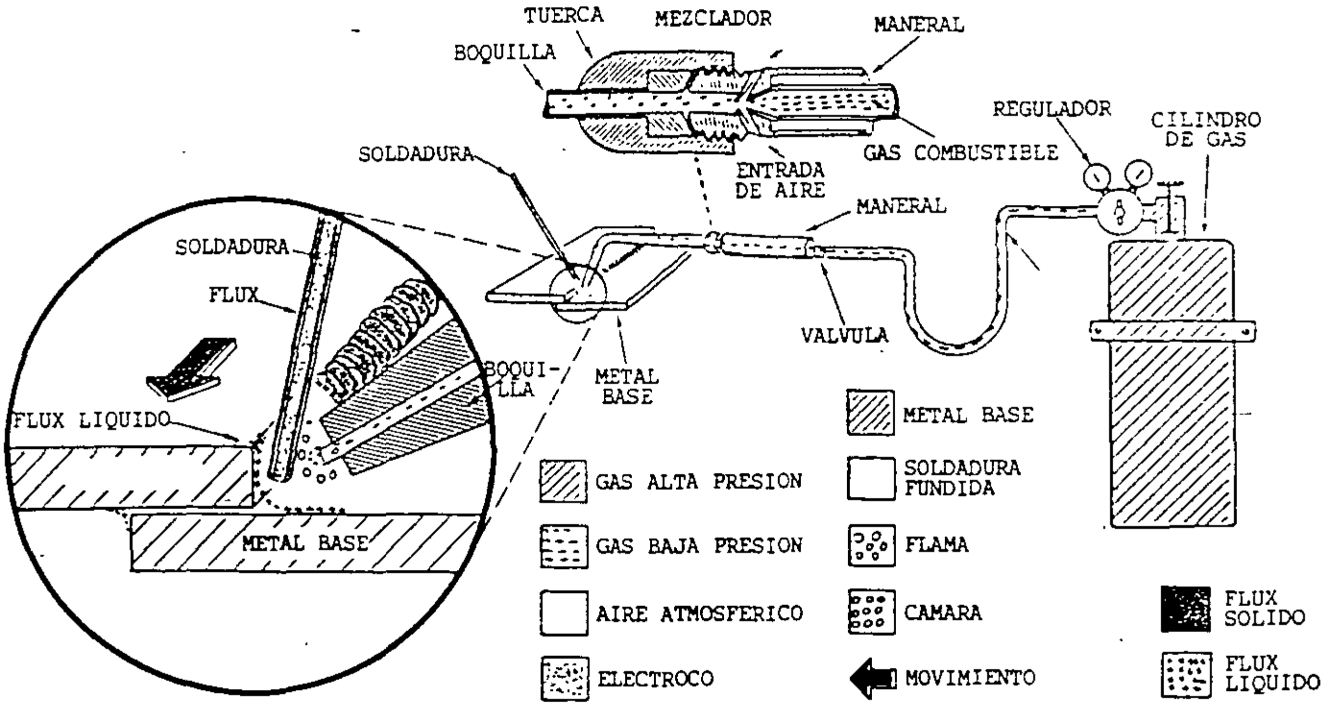


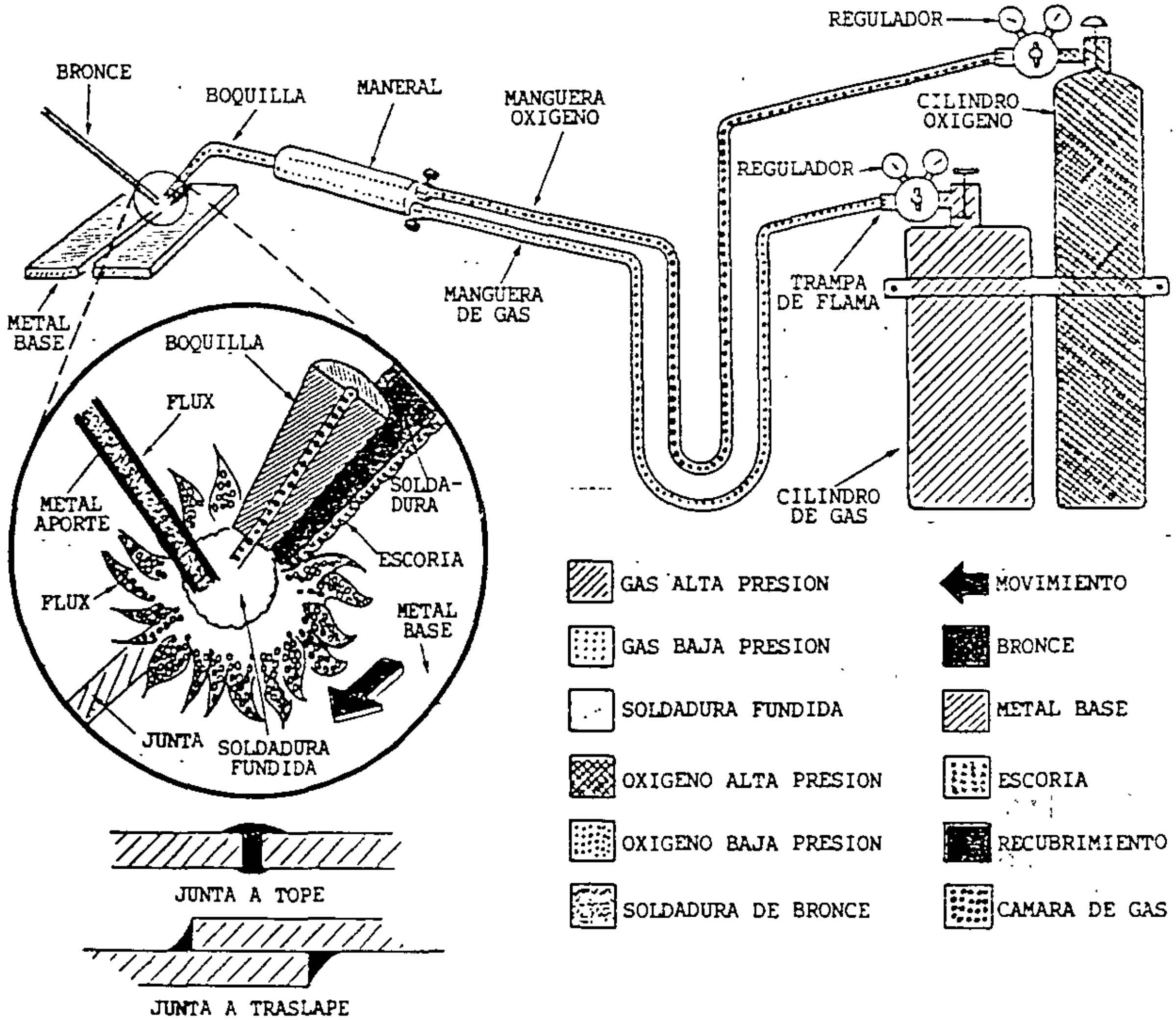
Fig. No. _____ Diseños de algunas juntas tipicas para soldadura; **A.** Junta - de tralape de lámina de acero en posición plana. **B.** Junta de abertura ó ranura de lámina de acero en posición plana. **C.** Junta de BRIDA en posición -- plana. **D** y **D** juntas de esquina exterior. **E** y **F.** Juntas de esquina interior (F comunmente la nombran Junta T). G, H, J y K. Diseños de juntas para -- placa metálica. Observar que para soldar las juntas **A, B** y **D** , se requiere utilizar varilla de aporte de soldadura. Para soldar las juntas C y D no se necesita varilla de aporte de soldadura debido a que las partes de metal -- son fundidas entre si formando una cama y uniendo ambas piezas.



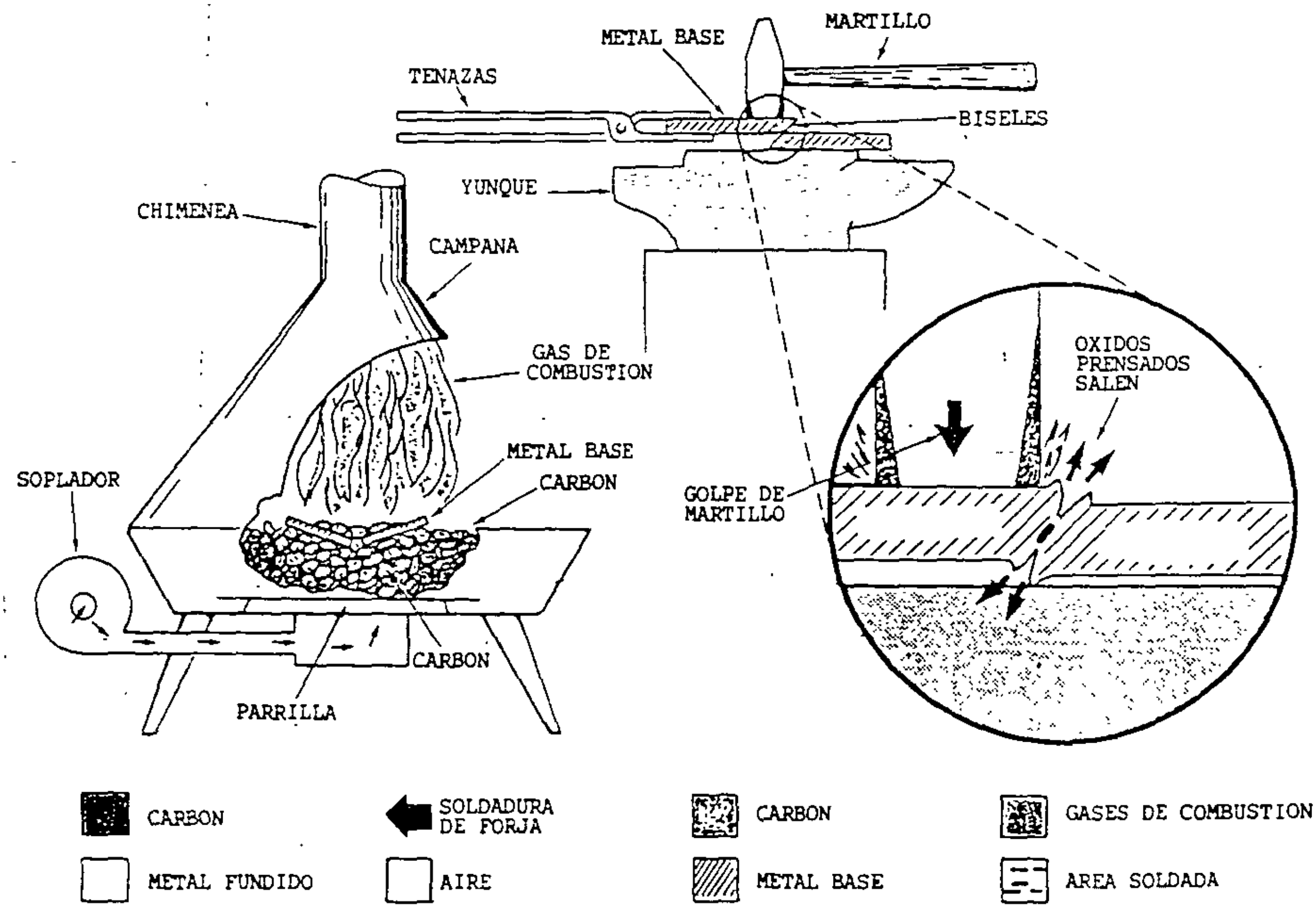
SOLDADURA CON EQUIPO OXIACETILENO (OAW)



SOLDADURA AIRE-GAS (TS)



SOLDADURA OXIGENO-GAS (TB)



SOLDADURA DE FORJA

OXICORTE CON GASES

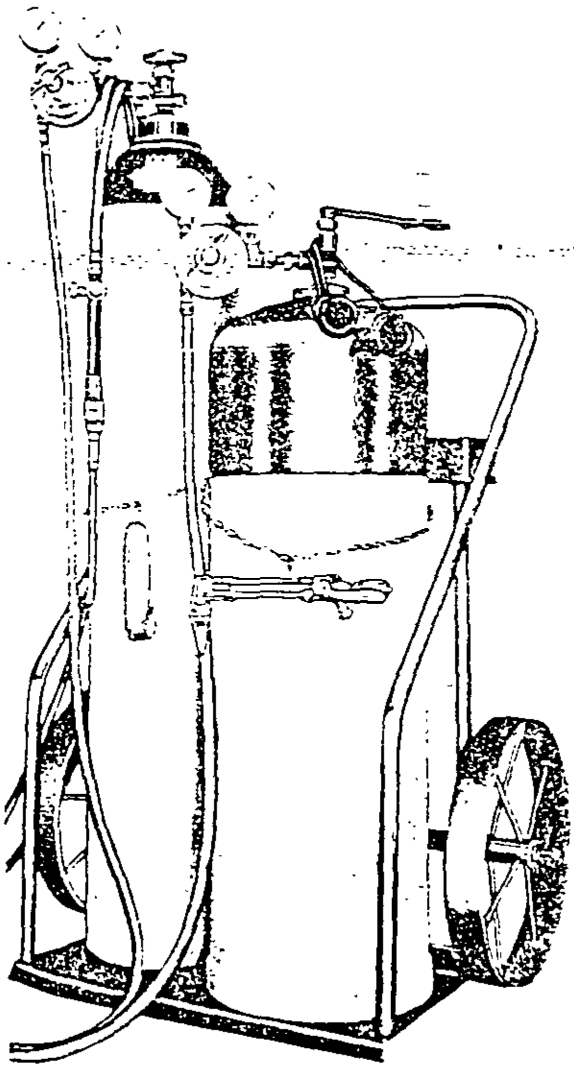
Para el corte de metales se emplean varios procesos, entre los cuales el más usual es el llamado corte con oxiacetileno, en el cual es empleada una flama de oxiacetileno para calentamiento del metal y un chorro de oxígeno para realizar el corte. El arte del corte con oxiacetileno ha progresado rapidamente, siendo ahora posible realizar cortes tanto en placas metálicas muy delgadas como de gran espesor, así como en cortar placas colocadas en capas al mismo tiempo para incrementar la producción en ciertos procesos industriales.

El corte con oxiacetileno es particularmente usado para obtener elementos estructurales para la fabricación de maquinaria y estructuras metálicas para edificios a partir de placas metálicas, este procedimiento da elementos precisos que posteriormente son soldados, resultando un procedimiento económico, que da elementos de gran resistencia y buenos acabados.

El proceso de corte oxiacetileno consiste en usar una ó más flamas de oxiacetileno para calentar un punto de una pieza de acero a una temperatura " rojo cereza " (aproximadamente 1800° F).

La flama de oxiacetileno es ajustada empleada en la misma forma que es usada para soldar, cuando el punto calentado llega a la temperatura " rojo cereza " el chorro de oxígeno es lanzado rapidamente este chorro corta el metal, entonces el soplete cortador es movido en dirección del operador, realizando la operación de corte. La flama de precalentamiento es mantenida encendida durante el proceso de corte manteniendo calor extra en el metal.

Un mineral de corte es similar a uno para soldar, pero adicionalmente tiene un conducto para el chorro de oxígeno, en el dibujo podemos apreciar las distintas partes que lo componen. Debido a que la presión del oxígeno es mayor que la usada para soldar, un regulador de oxígeno para alta presión deberá de ser usado, y mangueras adecuadas para esta presión deberán emplearse.



ESPESOR METAL PULG.	ORIFICIOS DE PRECALENTAMIENTO	ORIFICIOS DE CORTE	PRESION OXIGENO PSIG.	PRESION ACETILENO PSIG..	VELOCIDAD PULG/MIN.
1/8-3/8	70	67	20-30	3	14-18
3/8-3/4	58	62	30-40	5	12-15
3/4-1	57	54	40-45	5	10-12
1 1/2-2	68	51	45-50	5	9-10

Fig. presiones de oxígeno-acetileno para corte de placa de acero.

El corte en metales puede dividirse en dos grupos :

1.- Metales en los que sus óxidos tienen una temperatura de fundición más baja que el metal.

2.- Metales en los cuales sus óxidos tienen una temperatura más alta que el metal.

Prácticamente todos los aceros caen dentro de la primera clasificación y presentan dificultades mínimas para su corte.

En el segundo grupo que incluye acero fundido, algunas aleaciones de acero tales como el acero inoxidable y metales no ferrosos, presentan complicaciones porque los óxidos tienen una temperatura mayor de fundición que el metal, lo que hace casi imposible su corte.

Se requiere mucha habilidad para realizar cortes con equipo de oxiacetileno en acero fundido, existiendo otros procedimientos como son :

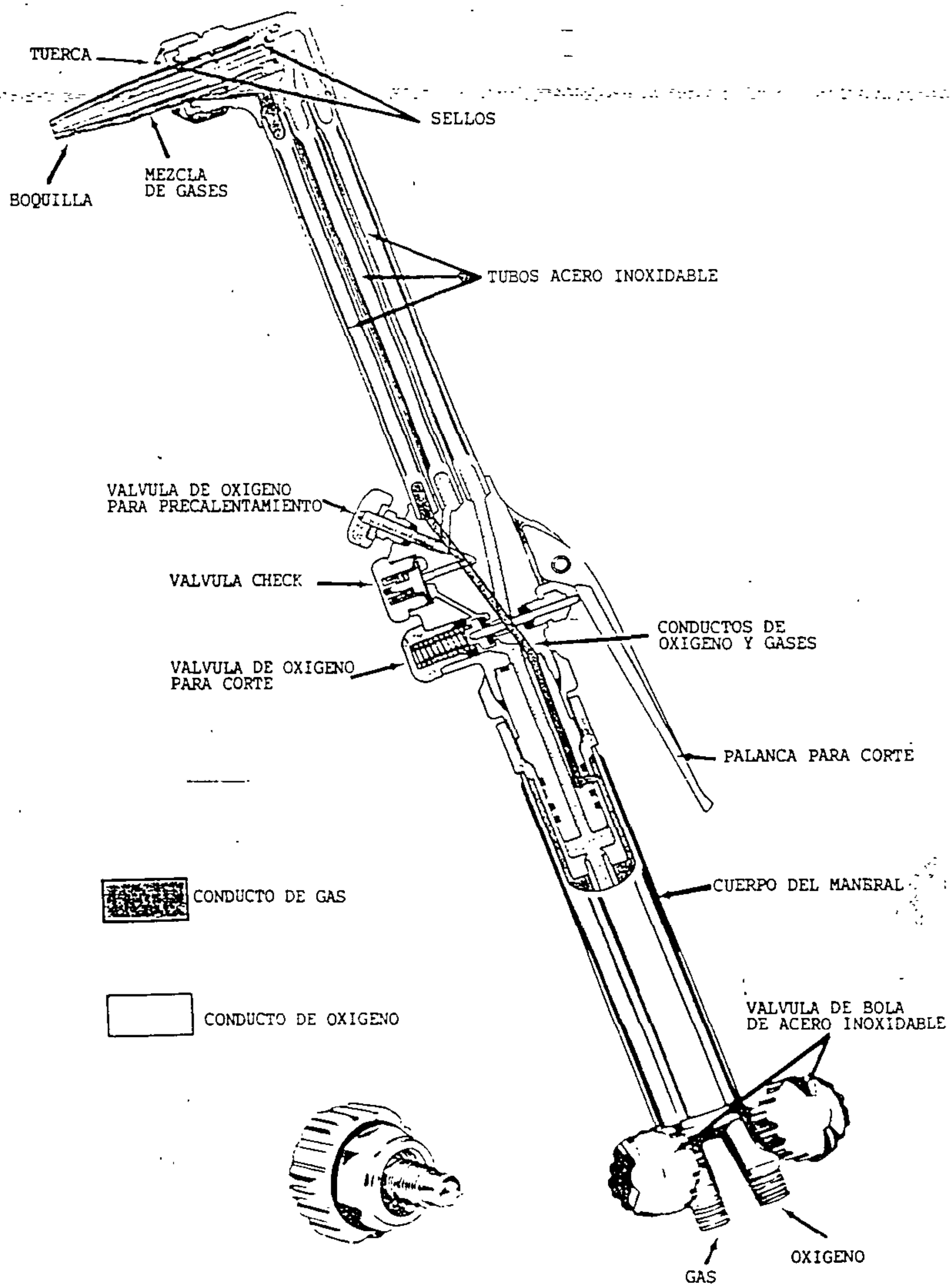
Arco metal	(MAC)
Arco aire	(AAC)
Arco oxígeno	(AOC)
Lanza oxígeno	(LOC)
Oxígeno-gas bajo el agua	(OFGUC)
Gas inerte	(GTAC)
Flux oxígeno	(FUC)
Arco plasma	(PAC)

Anexo se muestran los procedimientos de los mismos.

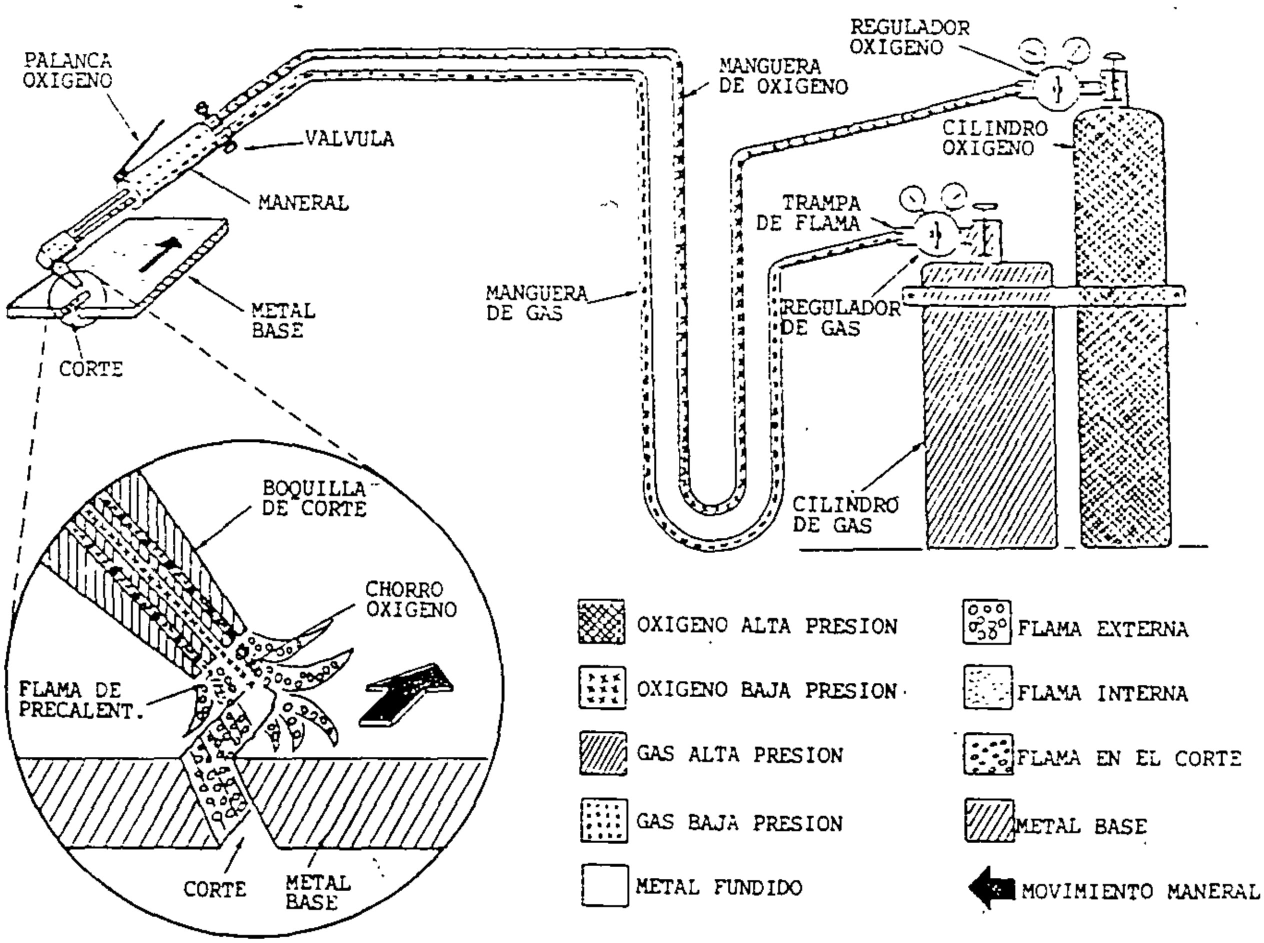
Al manejar equipos de corte es conveniente seguir y aplicar las siguientes prácticas de seguridad para evitar accidentes.

1. Siempre use gafas con los vidrios filtrantes adecuados al utilizar un soplete encendido.
2. Use guantes del tipo de puño acampanado de mayor longitud que los normales, y que sean de piel resistente al calor, para proteger sus manos y muñecas..
3. Tenga cuidado de que su ropa no esté aceitosa y de que los bolsillos y puños no estén abiertos y listos para recibir chispas o escoria caliente.
4. Use una careta resistente al calor, o una careta con casco
5. No utilice equipo que sospeche esté defectuoso.
6. Nunca utilice un cerillo o el metal caliente para encender o volver a encender un soplete.

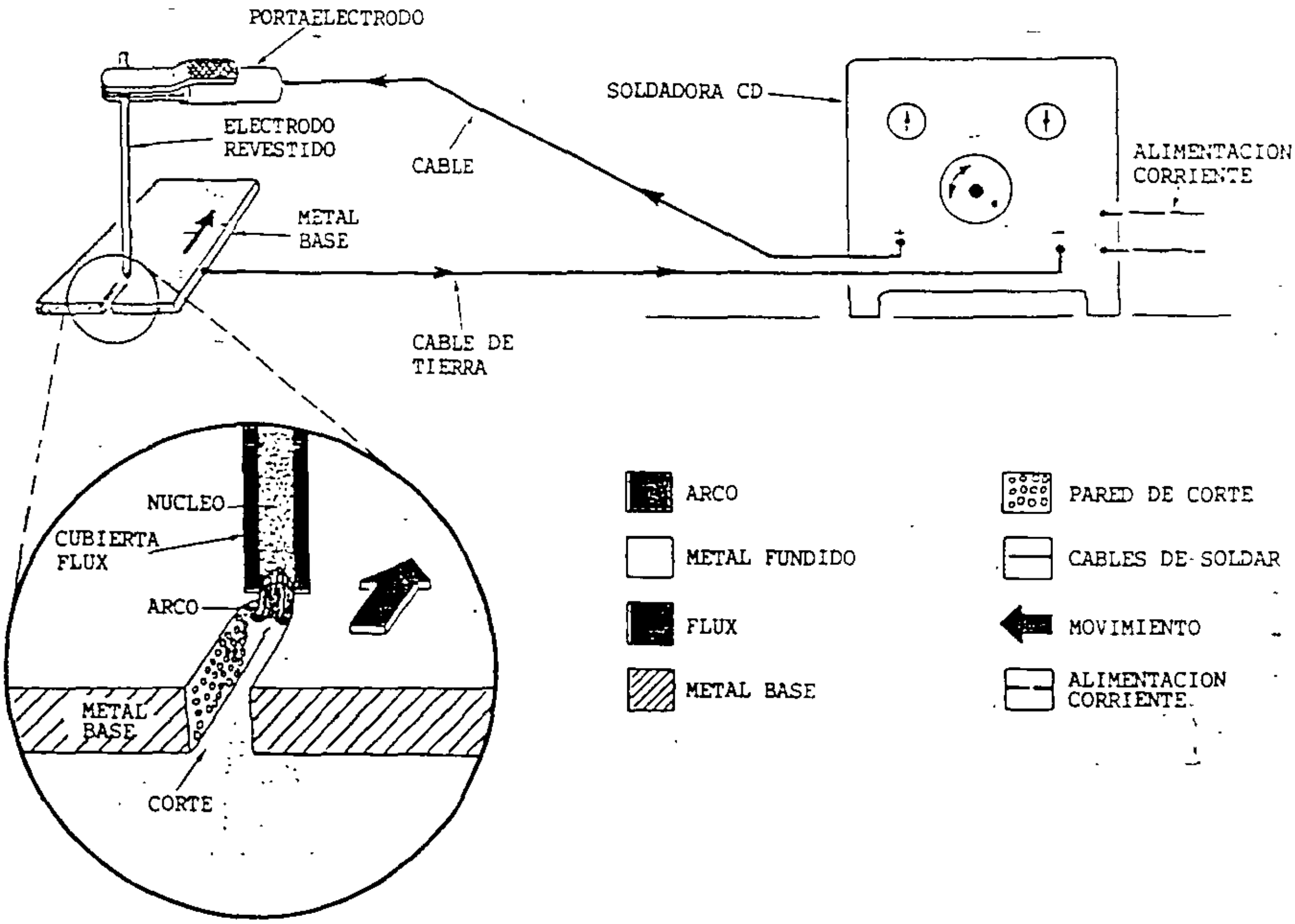
7. Nunca use acetileno a presiones manométricas superiores a 15 libras por pulgada cuadrada.
8. Abra siempre por completo las válvulas del cilindro de oxígeno.
9. Nunca abra las válvulas del cilindro de acetileno más de 1 1/2 vueltas.
10. Use sólo la llave de tuercas que fue surtida con el cilindro para abrir sus válvulas.
11. Mantenga siempre la llave de tuercas de la válvula del cilindro de acetileno sobre la válvula misma, hasta que haya terminado el trabajo y se haya purgado la manguera.
12. Conserve a mano en todo momento extinguidores adecuados contra incendio.



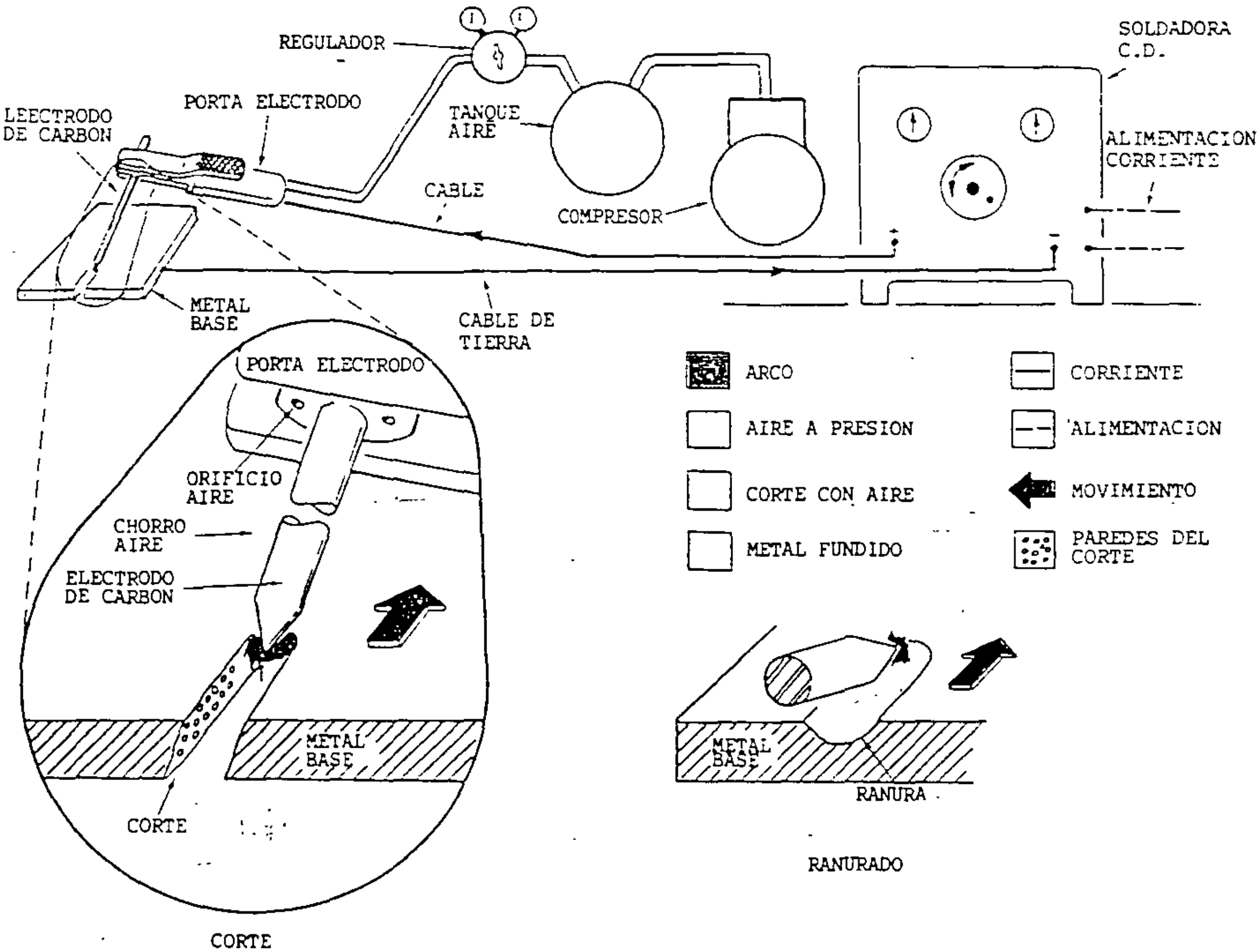
VISTA DE UN CORTADOR ENSAMBLADO EN UN MANERAL DE SOLDAR



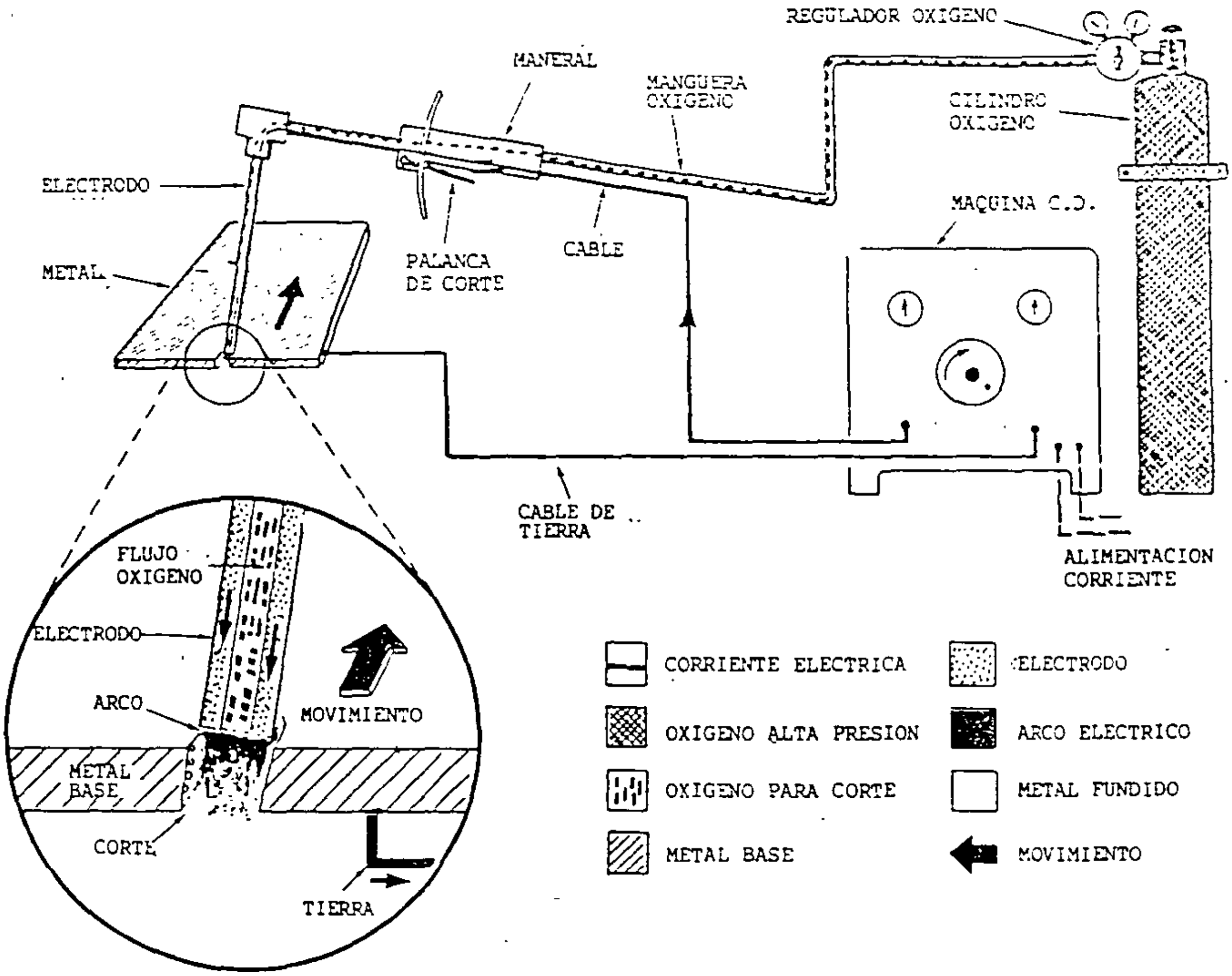
CORTE CON OXIGENO-GAS (OFC)



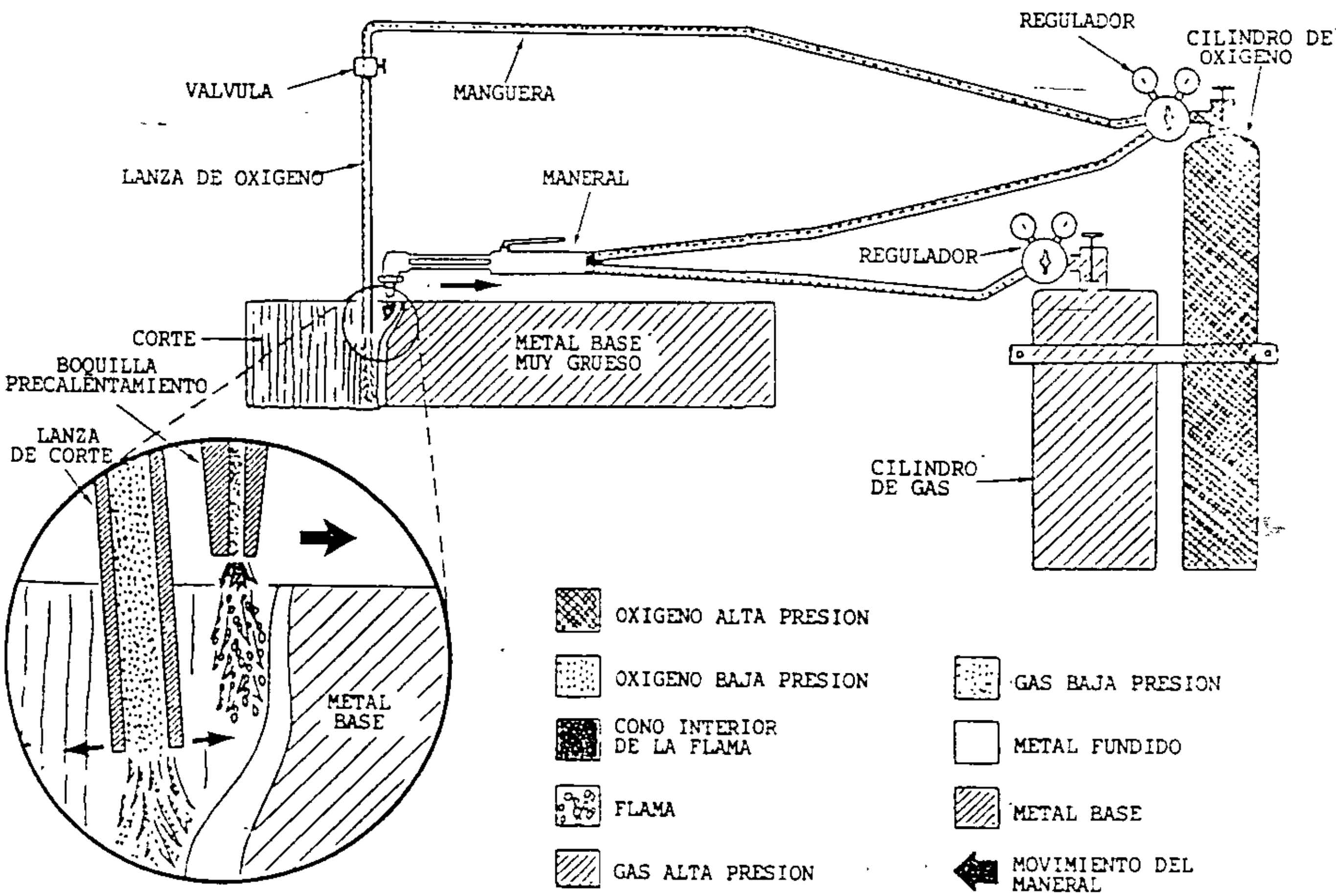
CORTE CON ARCO ELECTRICO (MAC)



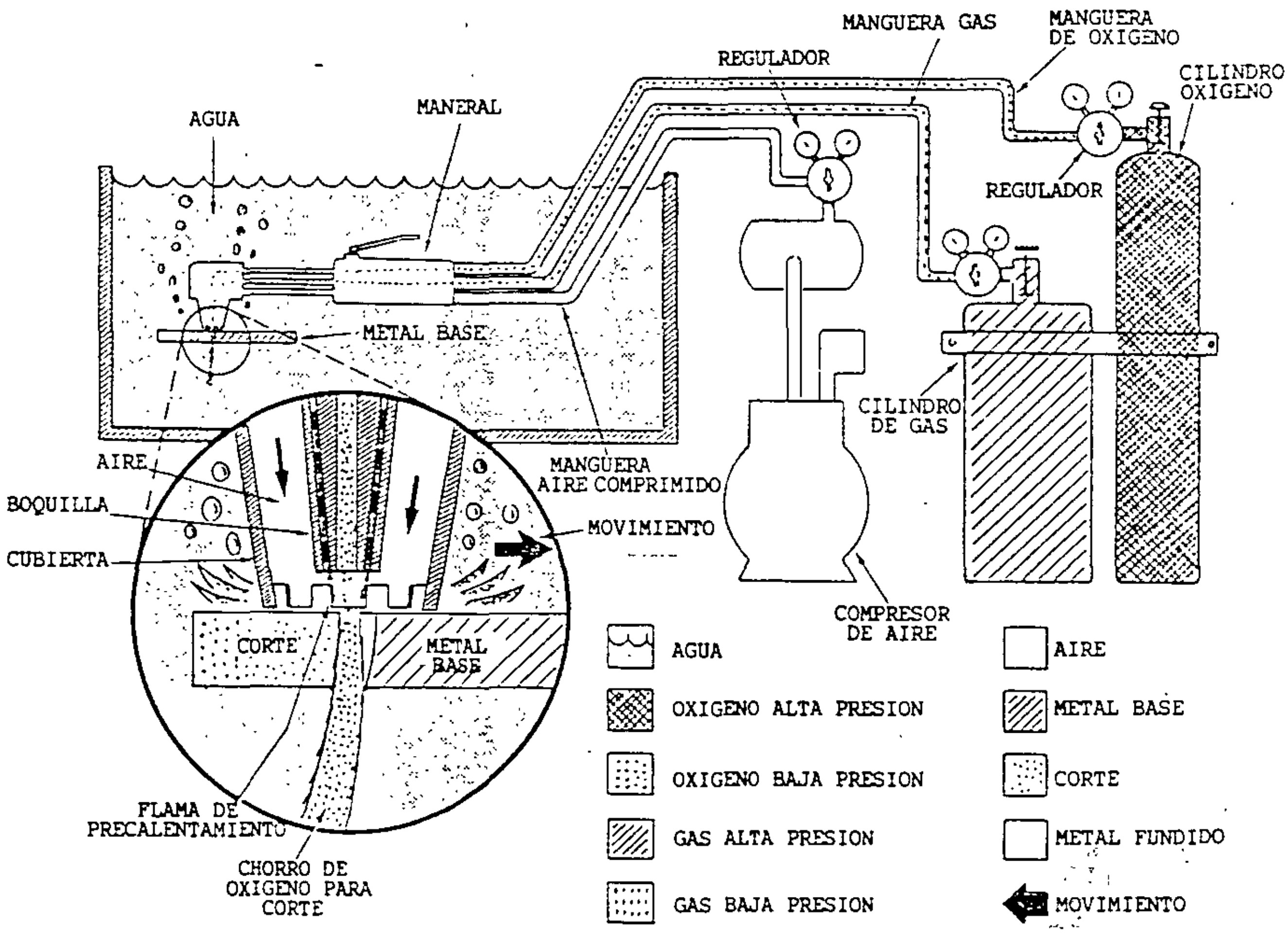
CORTE CON ARC-AIR (AAC)



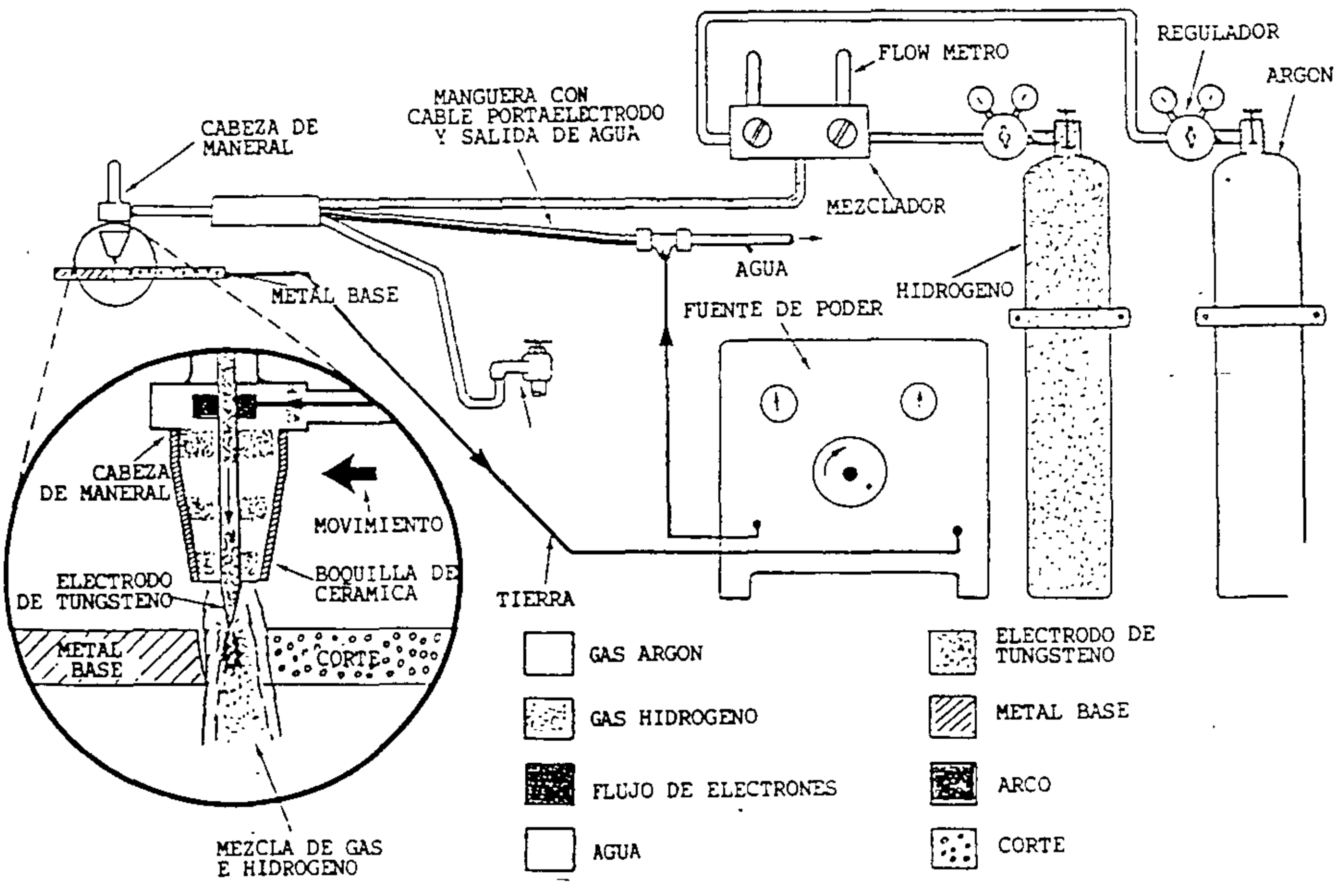
CORTE CON ARCO-OXIGENO



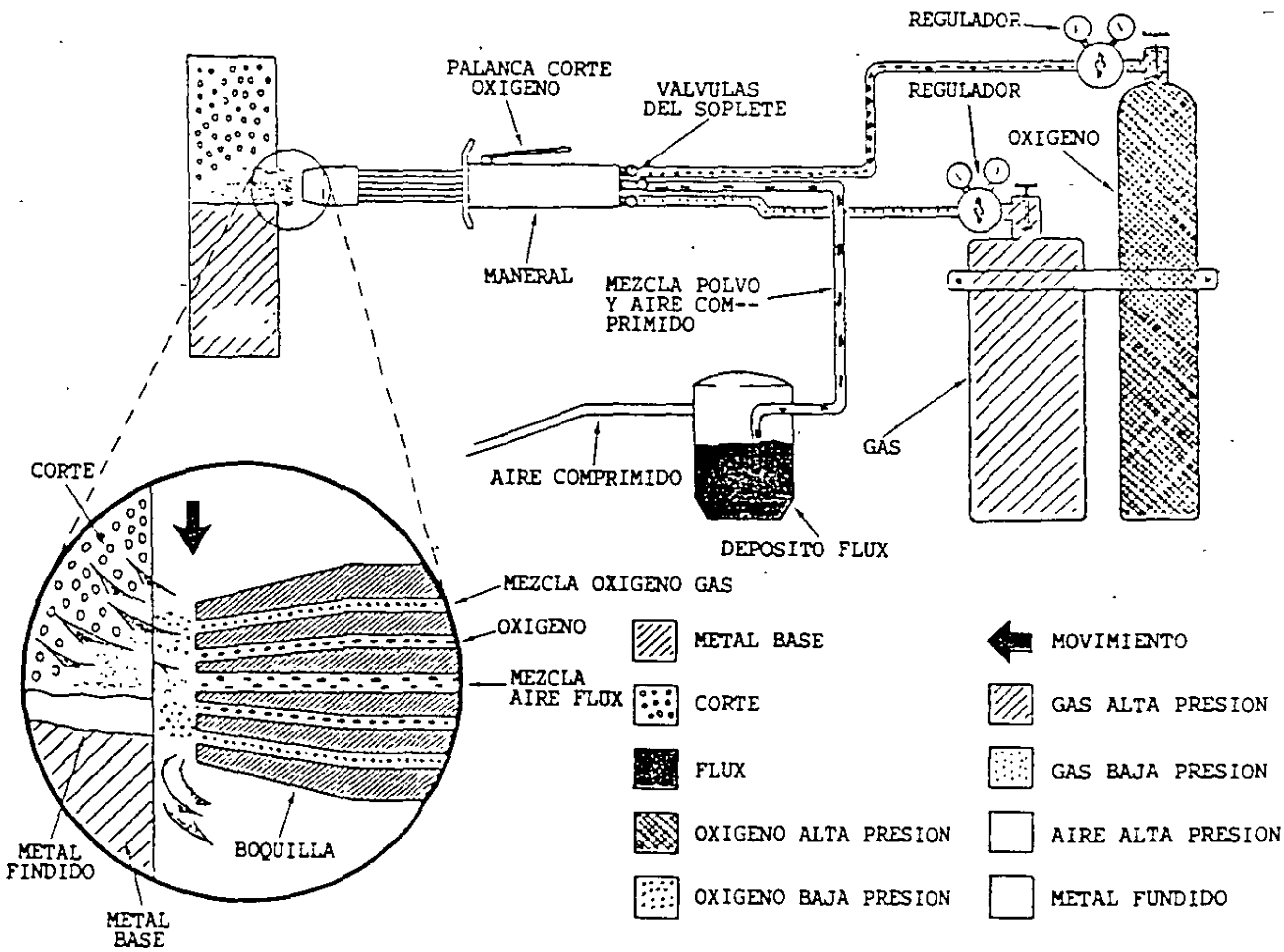
CORTE CON LANZA DE OXIGENO (LOC)



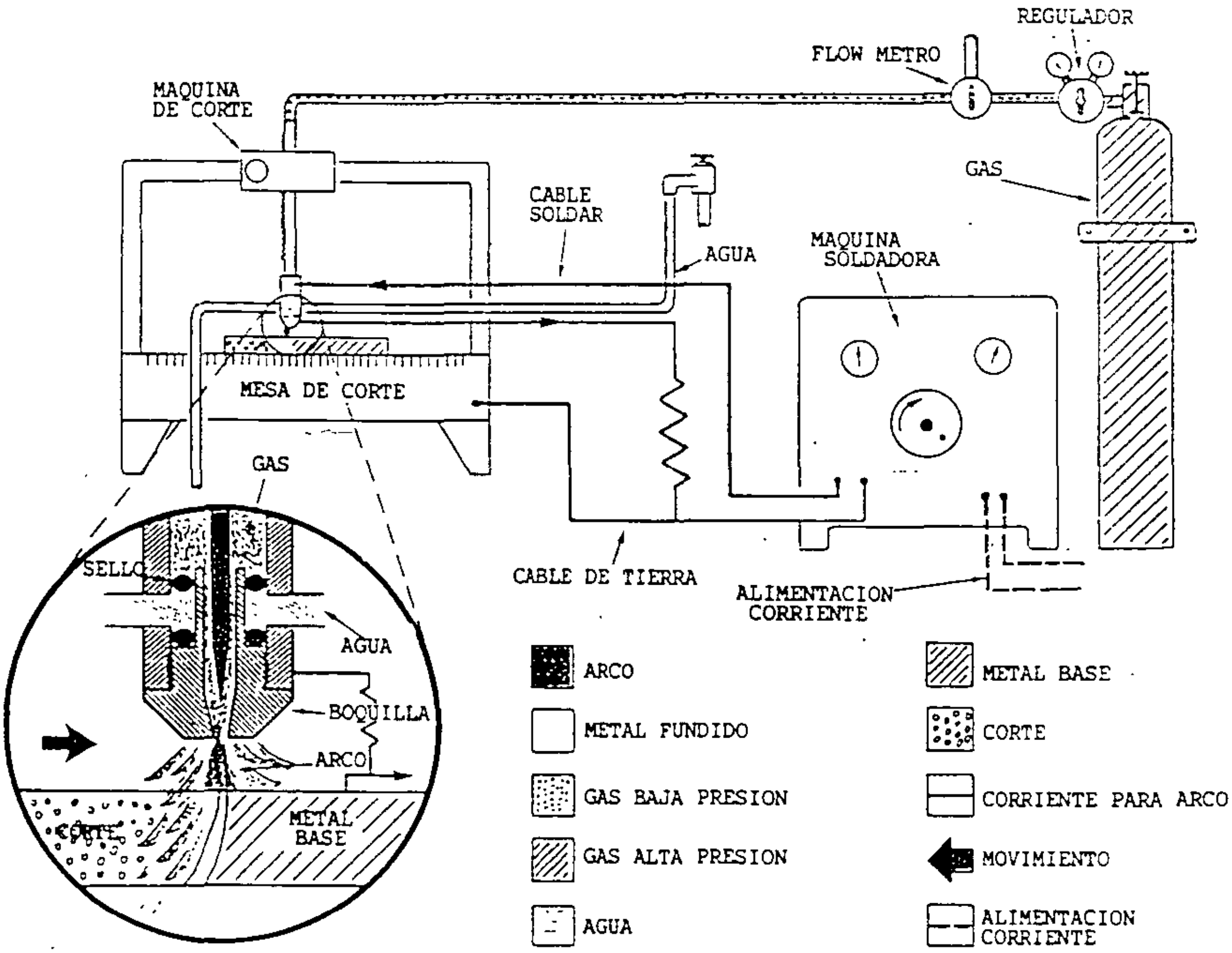
CORTE SUBMARINO CON OXIGENO-GAS (OFGUC)



CORTE CON ARCO-GAS TUNGSTENO (GTAC)



CORTE OXI-FLUX (FOC)



CORTE CON PLASMA (PAC)

SOLDADURA DE ARCO
CON CORRIENTE DIRECTA

La AWS define la soldadura de arco eléctrica :

" Un grupo de procesos de soldadura dentro de los cuales la unión se produce por calentamiento eléctrico, con un arco eléctrico o arcos, con o sin la aplicación de presión, y con o sin metal de relleno " al tocar el electrodo con el metal base se produce un arco eléctrico capáz de alcanzar de 6500 a 7000° F.

Los primeros electrodos se elaboraron desnudos y presentaban mucha dificultad para mantener el arco. La atmósfera abierta no permite la realización de una buena estabilidad de arco debido a la presencia constante del fenómeno de la oxidación.

En la fig.________ se muestra la forma en que se produce el arco eléctrico.

En la fig.________ se puede observar el detalle de un arco eléctrico con un electrodo desnudo.

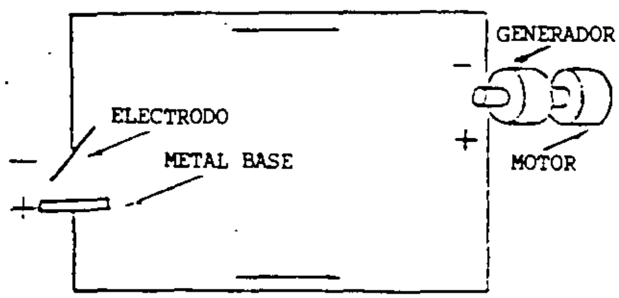


FIG.________ CIRCUITO DE SOLDADURA DE ARCO CON ELECTRODO METALICO.

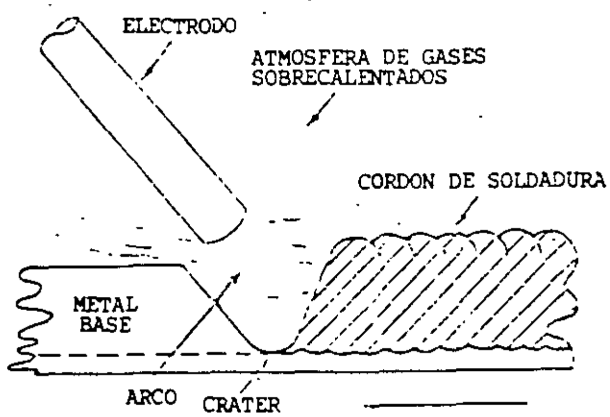


FIG.________ SOLDADURA DE ARCO DE UN ELECTRODO DESNUDO EN PROCESO

Los electrodos revestidos permiten que el arco tenga mayor estabilidad porque crean una atmósfera de protección que ayuda a expulsar las impurezas del metal fundido y desarrolla gases inertes, los cuales mantienen las superficies exteriores del metal fundido libres de oxidación. Los elementos o componentes del revestimiento, forman una dura incrustación ó escoria que protege la soldadura de la oxidación a la vez que la enfria.

En la fig.________ se puede observar el detalle del arco de un electrodo revestido.

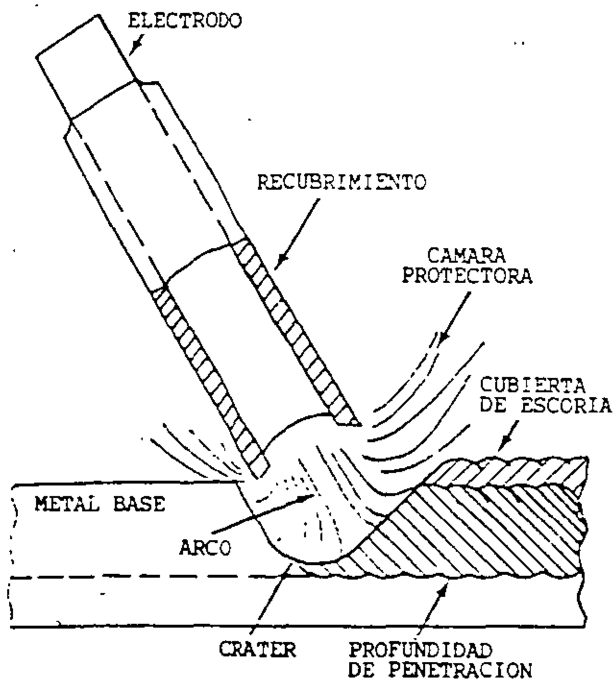


FIG.________ SOLDADURA DE ARCO DE UN ELECTRODO REVESTIDO EN PROCESO.

En la fig.________ se muestra una tabla que correlaciona el espesor del metal, el tamaño de electrodo, el amperaje requerido y el voltaje a utilizar.

ESPESOR DEL METAL	TAMAÑA DEL ELECTRODO	AMPERES PARA SOLDAR (EN PLANO)	VOLTAJE
1/16 - 1/8	3/32	50 - 90	15 - 17
1/8 - 1/4	1/8	90 - 140	17 - 20
1/4 - 3/8	5/32	120 - 180	18 - 21
3/8 - 1/2	3/16	150 - 230	21 - 22
1/2 - 3/4	7/32	190 - 240	22
3/4 - 1	1/4	200 - 300	22

FUNDAMENTOS DE SOLDADURA DE ARCO CON CORRIENTE DIRECTA Y POLARIDAD DIRECTA.

El circuito de soldadura que se puede observar en la fig.________ se conoce como circuito de polaridad directa. Es conocido que los electrones estan fluyendo de la terminal negativo (catodo) de la máquina a el electrodo. Los electrones continuan su viaje a través del metal base hacia la terminal positiva (anodo) de la máquina.
Aproximadamente dos tercios del total de calor producido es liberado en el metal base mientras que el tercio restante es liberado para el electrodo.
La elección de la corriente directa (polaridad) depende de muchas variables tales como el material del metal base, posición de la soldadura, material del electrodo y el componente de su revestimiento.

FUNDAMENTOS DE LA SOLDADURA DEL ARCO CON CORRIENTE DIRECTA Y POLARIDAD INVERTIDA.

Es posible y algunas veces deseable, cambiar la dirección del flujo de electrones en el circuito de la soldadura por arco cuando los electrones viajan desde la terminal negativa (catodo) a el metal base, este circuito es conocido como corriente directa polaridad invertida. En este caso los electrones regresan a la terminal positiva (anodo) de la máquina desde el lado del electrodo arco, tal como se muestra en la fig.________.

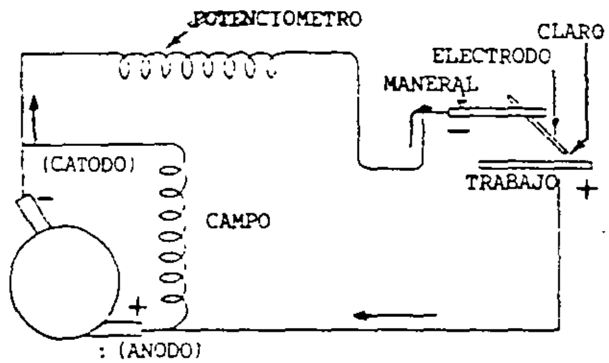


FIG.________ DIAGRAMA ELECTRICO DE CORRIENTE DIRECTA, POLARIDAD INVERTIDA.

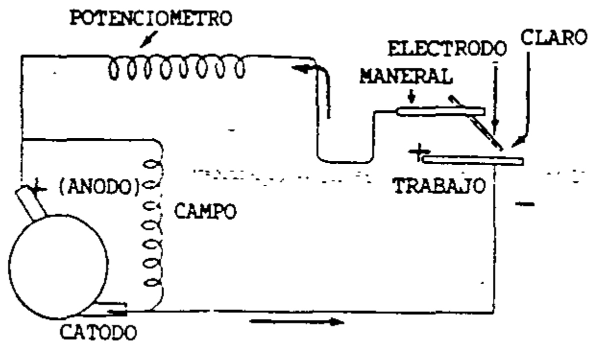
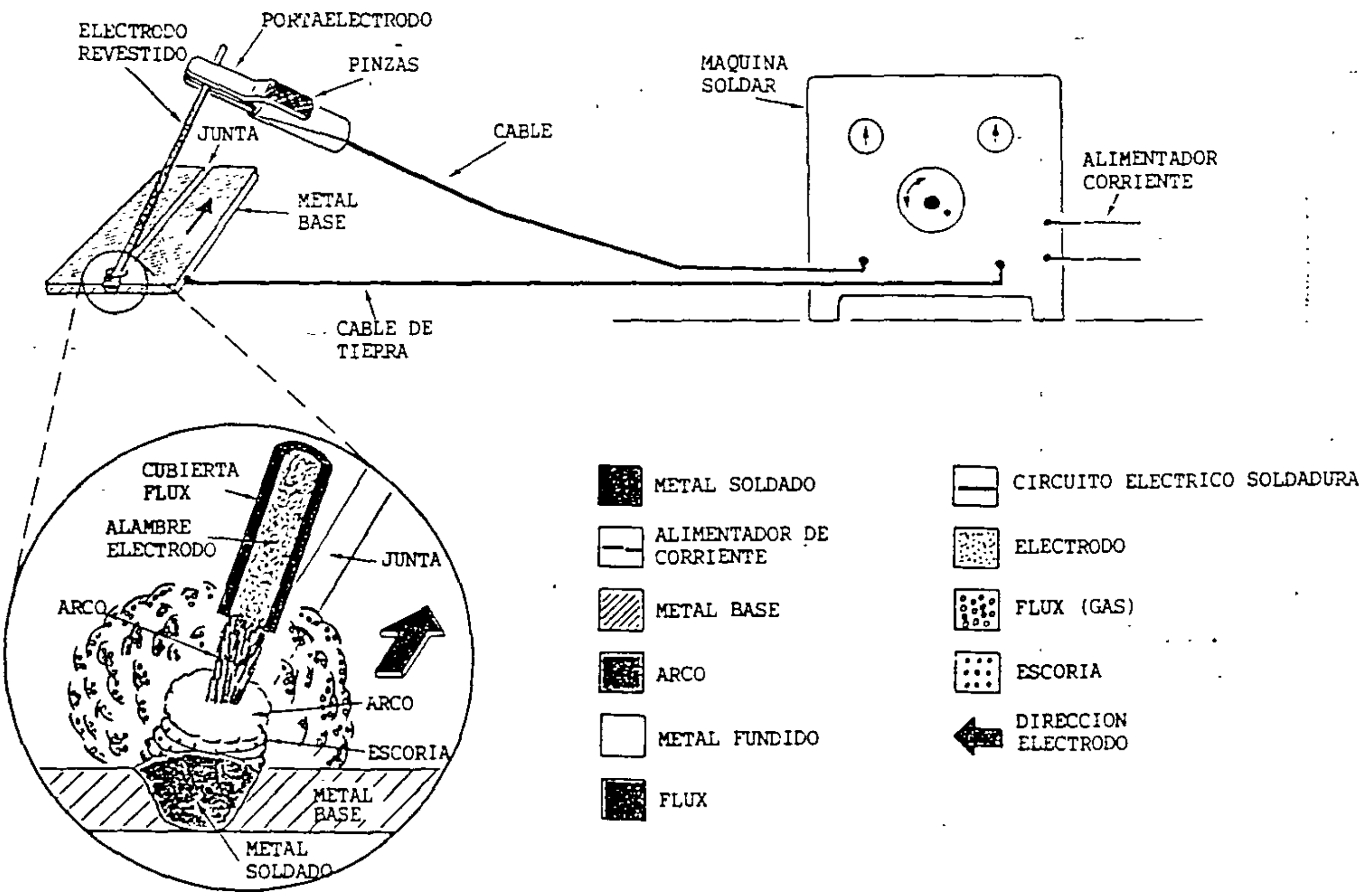


FIG.________ DIAGRAMA ELECTRICO DEL CIRCUITO DE SOLDADURA DE ARCO, CORRIENTE DIRECTA Y POLARIDAD INVERSA. OBSERVE QUE EL FLUJO DE ELECTRONES VIAJA DEL METAL BASE A EL ELECTRODO.

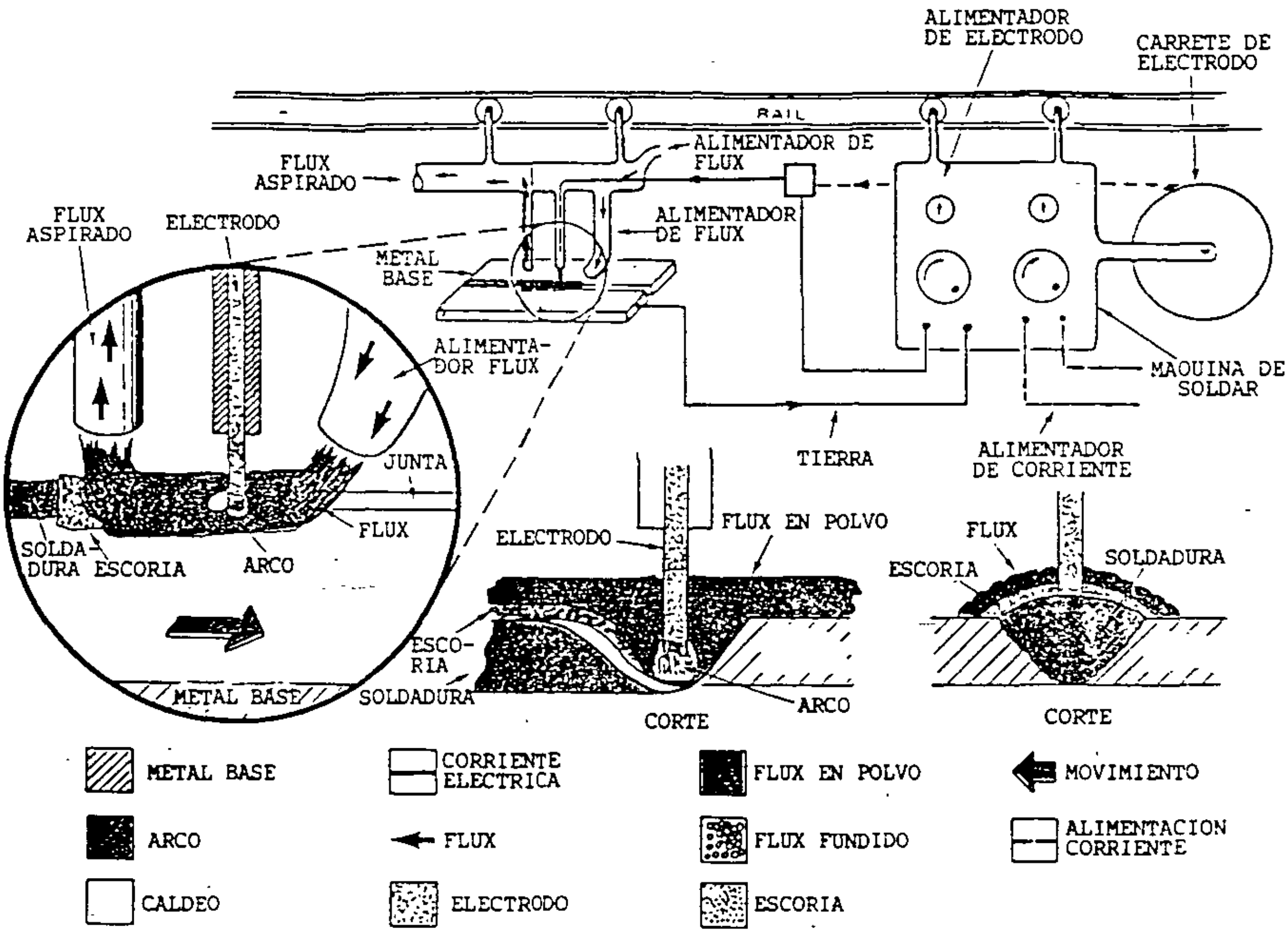
Cuanto se utiliza la polaridad invertida, un tercio del calor generado en el arco es liberado al metal base y dos tercios son liberados a el electrodo.

Con dos tercios de calor liberados al electrodo en la polaridad invertida, el electrodo metálico y los gases de protección del arco, son sobrecalentados. Este sobrecalentamiento provoca que el metal fundido del electrodo viaje a través del arco a alta velocidad.

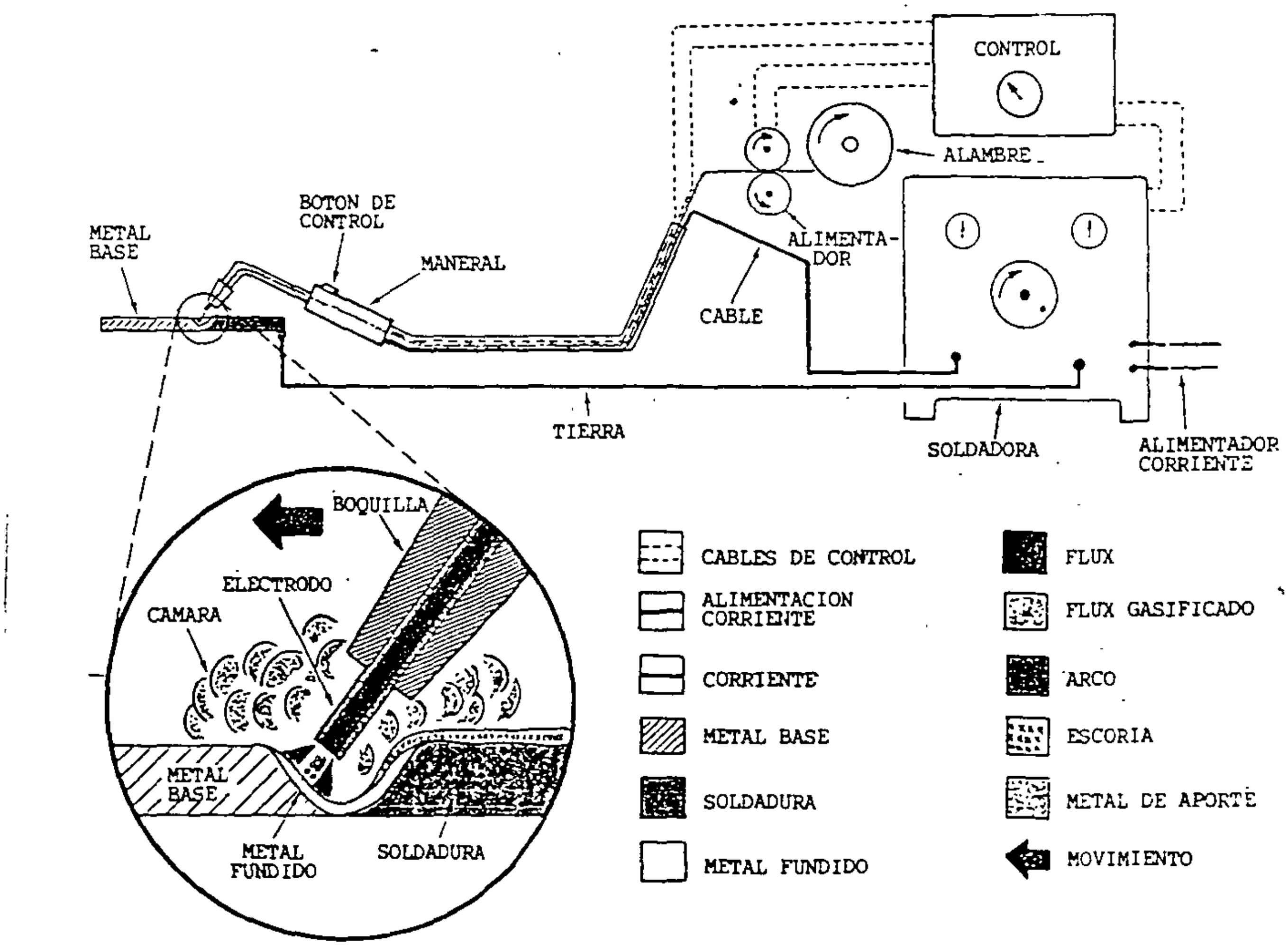
Una penetración profunda resulta de la fuerza de la alta velocidad del arco.



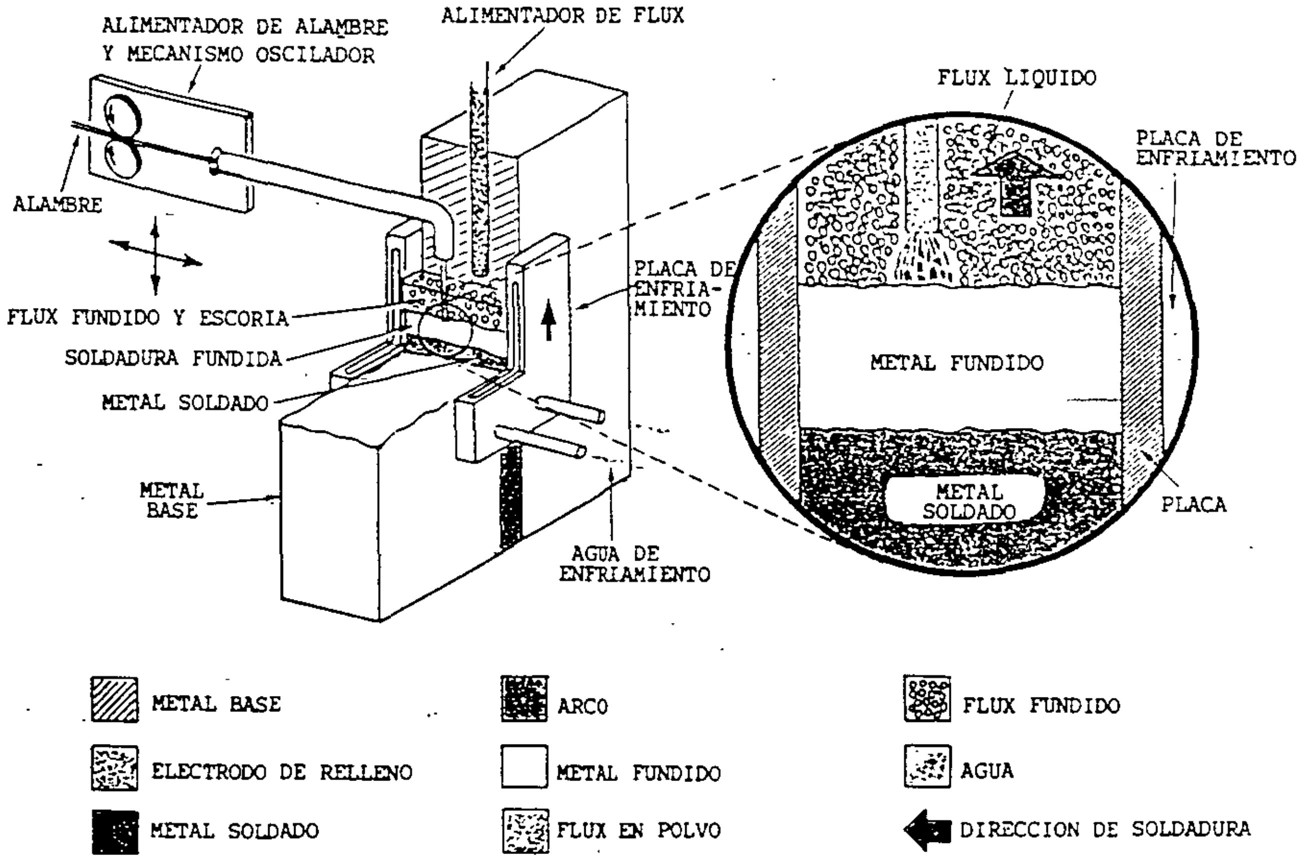
SOLDADURA ARCO ELECTRICO (SMAW)



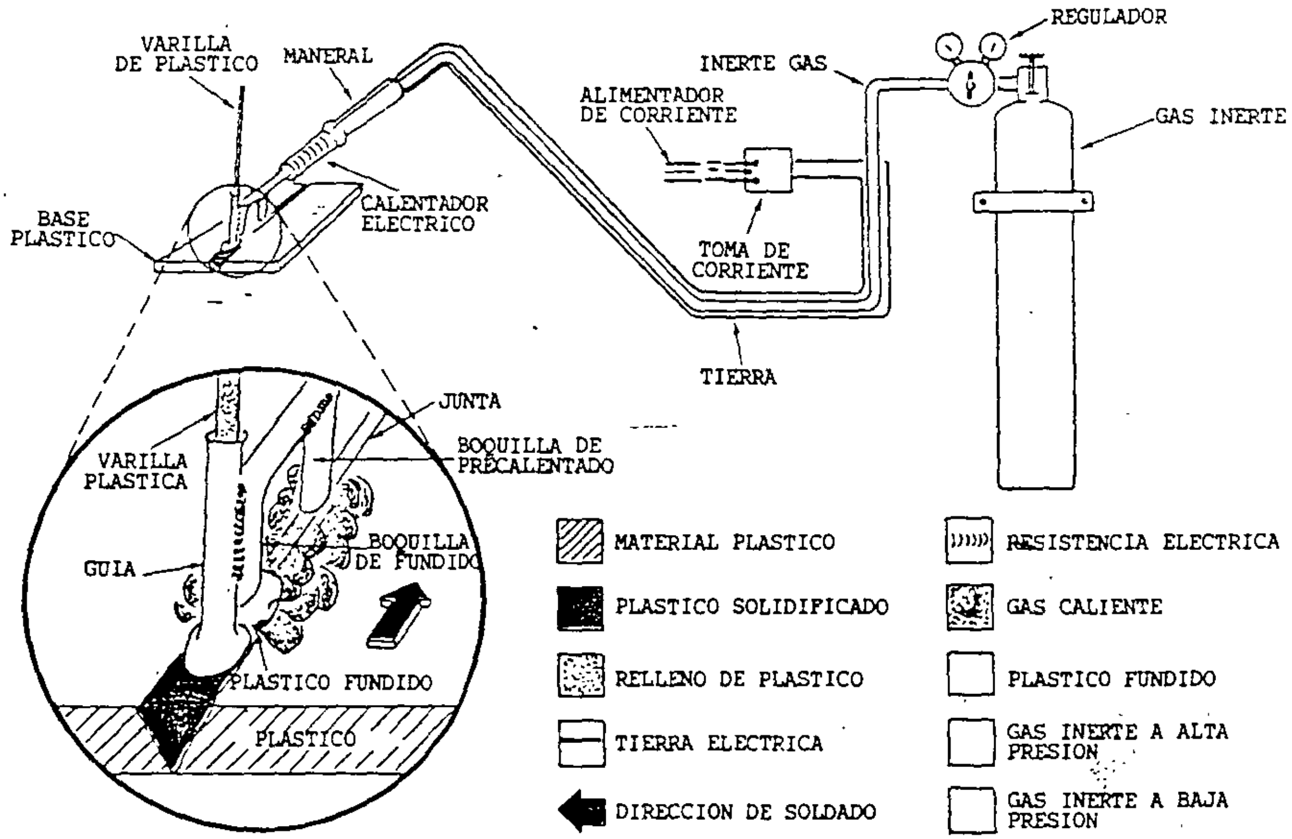
SOLDADURA ARCO SUMERGIDO (SAW)



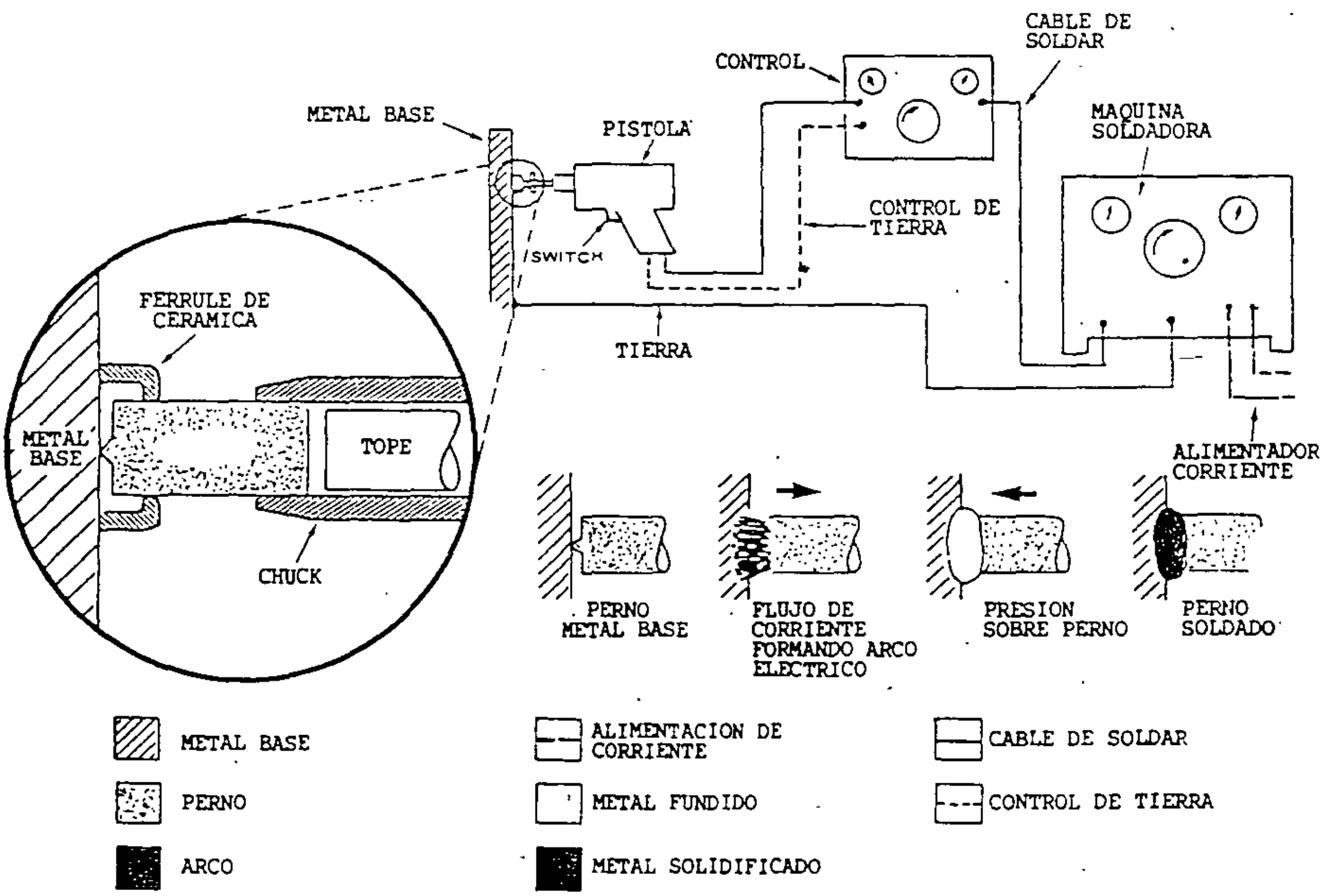
SOLDADURA CON FLUX-ALAMBRE (FCAW)



SOLDADURA ELECTRO-ESCORIA (ESW)



SOLDADURA DE PLASTICO



SOLDADURA PERNOS (SW)

SOLDADURA DE ARCO CON GAS

En la soldadura de arco con gas, un gas inerte es alimentado hacia la soldadura. Esto esta hecho para expulsar el aire atmosférico alrededor de la soldadura. Por otro lado, el oxígeno se combinaría con los metales fundidos y formaría óxidos los cuales debilitarían la soldadura.

El Helio, Argón, Dióxido de Carbón, ó una mezcla de estos puede ser usada como gases de protección. Sustancias tales como oxígeno, hidrógeno, nitrógeno y vapor de agua en la atmósfera de la soldadura, reducen la calidad de la soldadura.

Actualmente el gas Argón es más empleado que otros debido a su condición de gas pesado. Un gas ligero como el Helio cuando es calentado en el arco eléctrico, se vuelve más ligero aún y fluye lejos del arco. El gas Argón en condiciones de calentamiento permanece en el arco y mantiene a el aire fuera de la atmósfera de la soldadura.

El principio de la soldadura de arco con gas es muy simple. El portaelectrodo esta diseñado para suministrar un flujo de gas de protección tal como el Dióxido de carbono, Helio o Argón, el cual rodea el arco eléctrico.

El gas de protección mantiene el oxígeno y otros contaminantes, lejos del metal fundido a alta temperatura. También mantiene otros elementos activos en la atmósfera, lejos del metal fundido. Con la eliminación de la oxidación y otras impurezas, las soldaduras son posibles sobre metales los cuales son imprácticos o muy difícil de soldar.

El principio de soldadura de arco con gas puede ser usado manualmente, semiautomáticamente ó completamente automático.

Durante la soldadura de arco con gas el portaelectrodo impulsa gas alrededor del electrodo. Así como fluye, expulsa el aire atmosférico lejos del electrodo y del metal fundido.

La soldadura de arco con gas tiene tres grandes ventajes sobre las formas usuales de soldadura de arco. Estas ventajas son:

1) Es más veloz, minimizando la distorsión

2) Soldaduras más limpias

3) La facilidad para soldar metales que se consideraban muy difíciles o --

casi imposibles de soldar

Los costos por porcentajes, de los accesorios para realizar soldaduras de arco con gas, son aproximadamente:

- El electrodo de Tunsgteno (cuando se usa) 3%
- La energía eléctrica 5%
- El gas de protección 92%

Las corrientes eléctricas que se utilizan para este proceso son:

1) Corriente directa, polaridad directa

2) Corriente directa, polaridad invertida

3) Corriente alterna

Cuando se utiliza corriente directa polaridad directa, se obtiene buena penetración debido a que la corriente de electrones fluye hacia el trabajo, concentrando el calor en la zona de trabajo.

Cuando se usa corriente directa polaridad invertida, se obtiene una buena acción de limpieza pero la penetración no es mayor debido a que la mayor parte del efecto de calentamiento toma lugar en el electrodo de Tungsteno (anodo). Este proceso es el mejor usado sobre secciones delgadas de Aluminio, Magnesio y otros materiales difíciles de soldar.

Cuando se utiliza corriente alterna con alta frecuencia, es posible obtener tanto buenea penetración como buena limpieza.

SOLDADURA DE ARCO CON ELECTRODO DE TUNSGTENO Y GAS

Este proceso utiliza un electrodo de Tunsgteno que no se consume. El electrodo es montado en un portaelectrodo especial el cual esta diseñado para suministrar un flujo de gas de protección alrededor del arco.

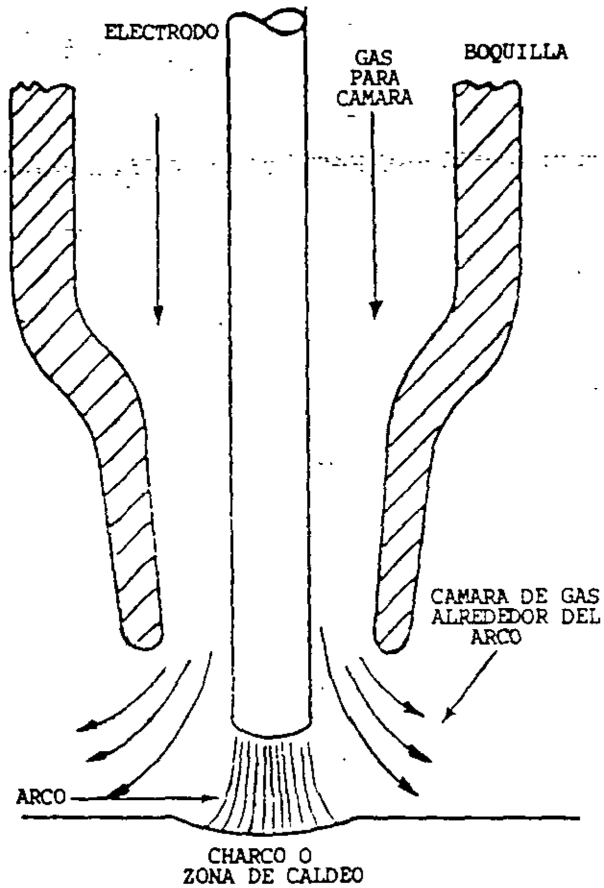


Fig. _____ principio de la soldadura de arco con protección de gas.

Los equipos para soldar con gas pueden estar enfriados por aire ó por agua.

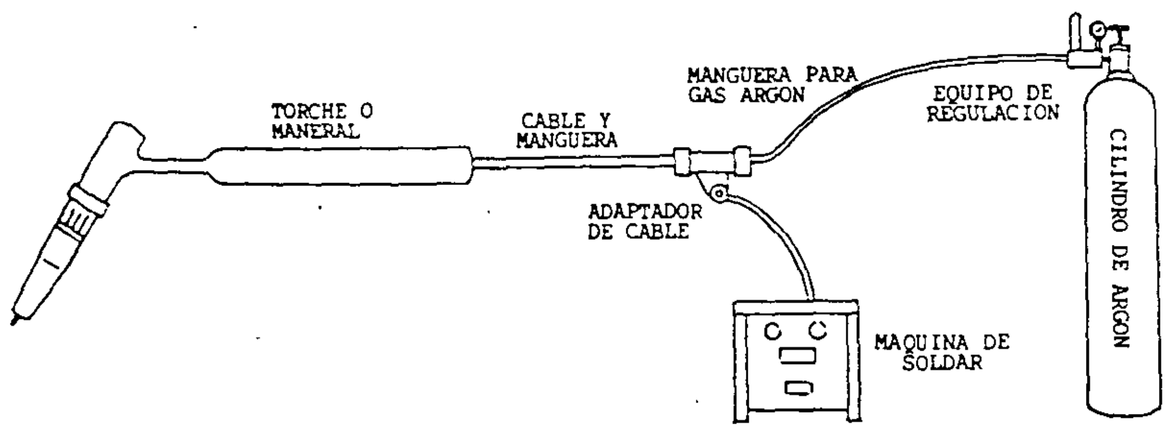


Fig. _____ equipo completo para soldadura de arco con protección de gas. — Esta unidad tiene un portaelectrodo enfriando por aire.

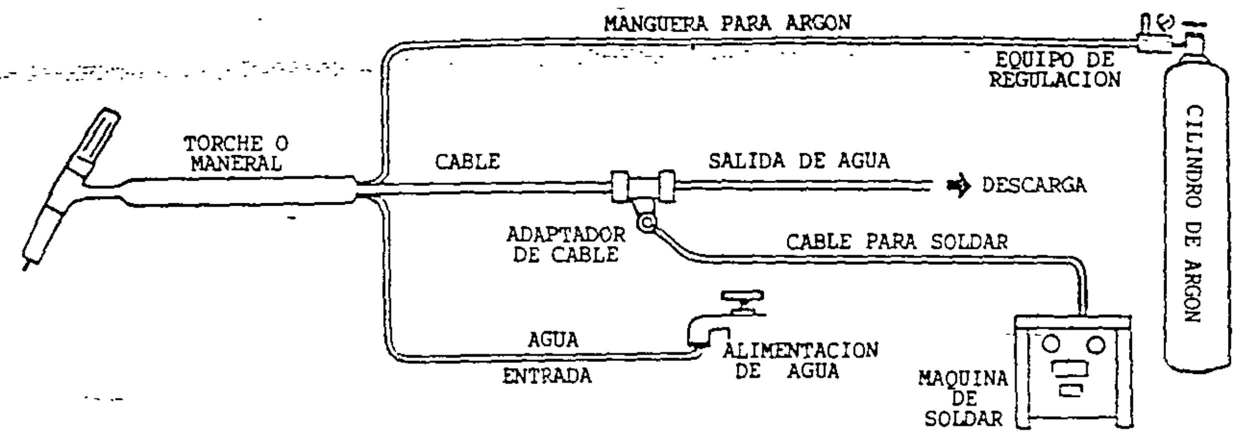


Fig. _____ Esquema de un equipo completo para soldadura de arco con protección de gas. Esta unidad esta enfriada por agua.

A continuación se anexan varios esquemas y tablas que serán de utilidad --- para el uso de la soldadura de arco con gas de protección.

	CORRIENTE DIRECTA			CORRIENTE ALTERNA	
DIAMETRO DEL TUNSGTENO	POLARIDAD DIRECTA HELIO	POLARIDAD INVERTIDA HELIO	POLARIDAD DIRECTA ARGON	HELIO	ARGON
0.040	50		65	MIN. 30	MIN. 40
1/16	50-125	10- 20	65-150	20-115	20- 60
3/32	125-225	20- 35	140-280	100-185	50-100
1/8	200-300	25- 50	250-375	150-225	75-175
3/16	250-350	30- 75	300-475	200-340	150-240
1/4	300-475	40-125	375	300-445	175-375

Fig. _____ dimensiones de electrodos de Tungsteno y las capacidades e corriente sugeridas para cada una, basada en el diámetro, el tipo de gas usado y el tipo de corriente usada.

DIAMETRO DEL ELECTRODO DE TUNGSTENO	DIAMETRO DE LA BOQUILLA DIAM. INTERIOR	FLUJO DE GAS HELIO PIES CUBICOS POR HORA
0.040	5/32 - 3/8	11
1/16	5/16 - 3/8	15
3/32	3/8 - 1/2	18
1/8	3/8 - 1/2	25
5/32	1/2 - 5/8	32
3/16	5/8	40

Fig. _____ diámetro interior de boquilla (copa) aproximado y valores del -- flujo de gas en relación al diámetro del electrodo de Tungsteno.

M E T A L		FLUJO DE GAS PIES³/HR.	
TIPO	ESPESOR	ARGON	HELIO
ACERO	0.35-3/32	8-10	20-30
HIERRO FORJADO	1/4	16	40
ACERO INOXIDABLE	1/16-1/8	11	30
ACERO INOXIDABLE	3/16-1/4	13	32
COBRE	1/16-1/4	15	38
MAGNESIO	1/16-1/8	10	25

Fig. _____ tabla de velocidades de flujo sugeridas para metales diferentes.

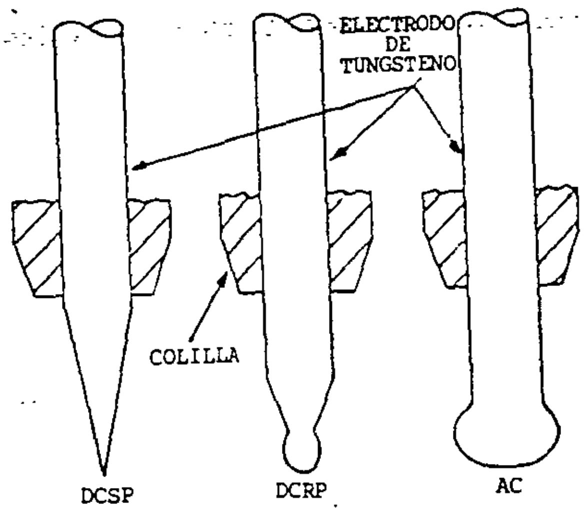
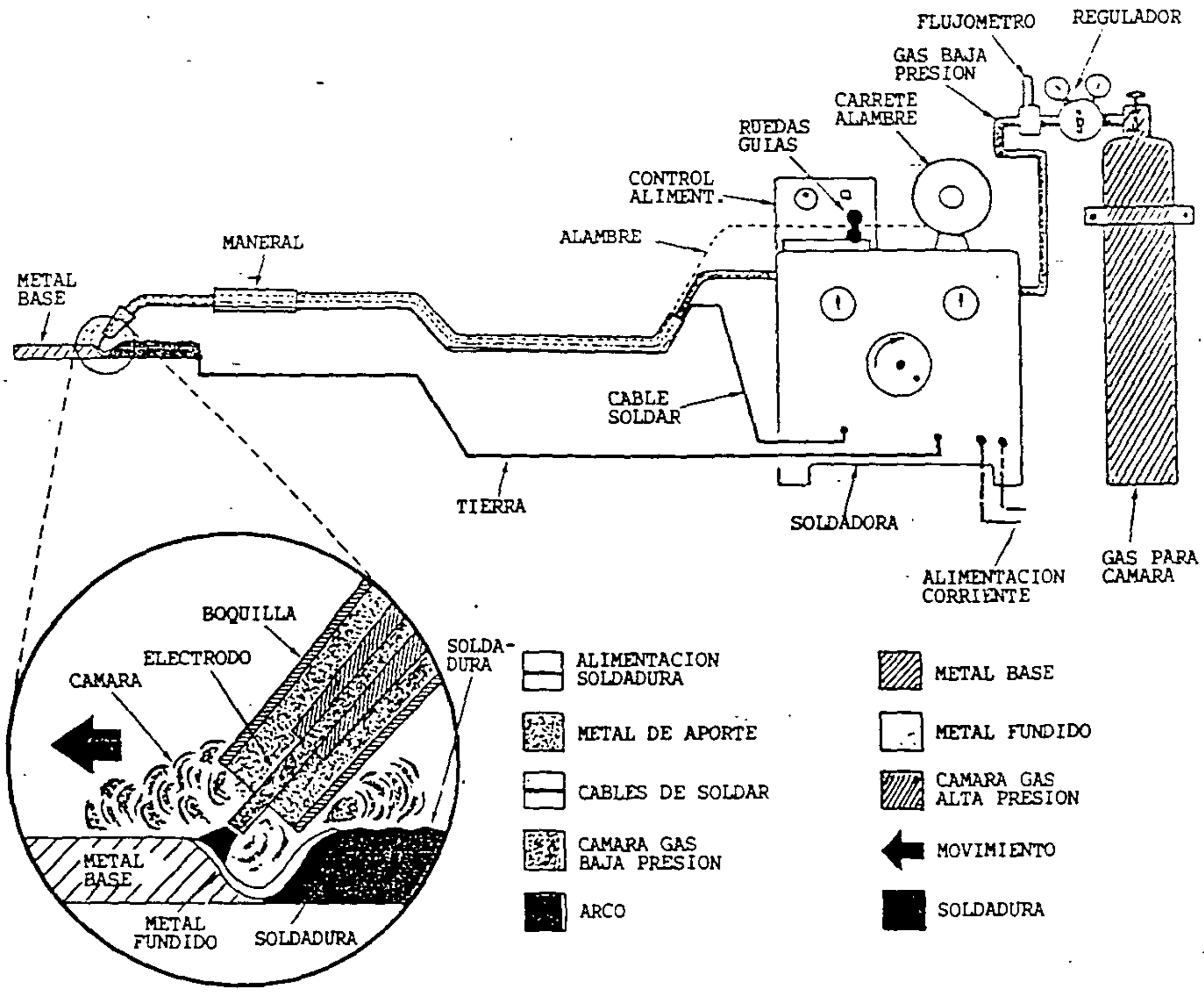
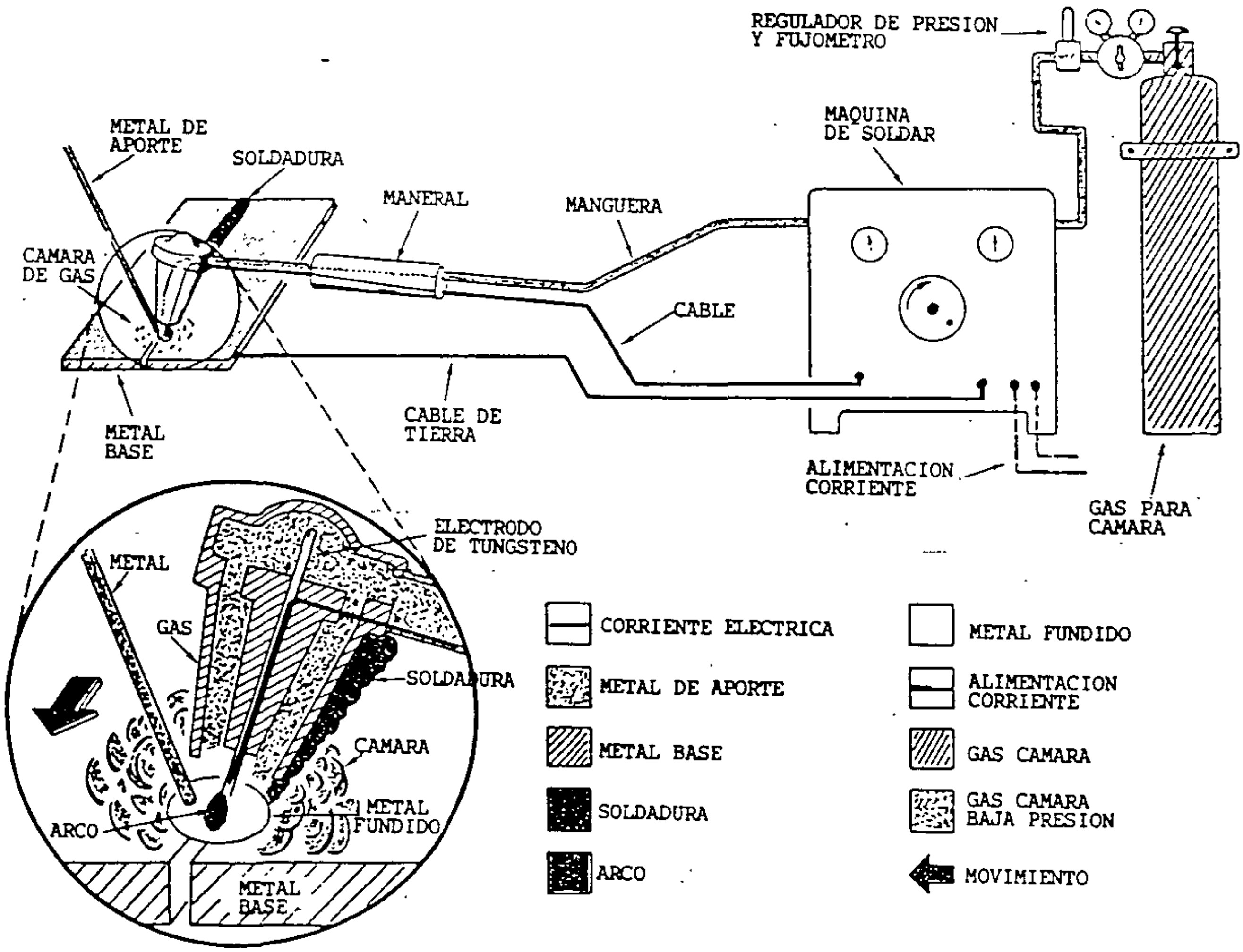


Fig. _____ forma correcta de los electrodos de Tungsteno para corriente directa polaridad directa, polaridad invertida y corriente alterna.



SOLDADURA CON ARCO-GAS METAL (GMAW)



SOLDADURA ARCO-GAS TUNGSTENO (GTAW)

SOLDADURA POR RESISTENCIA ELECTRICA

Toda la soldadura por resistencia esta basada sobre el principio fundamental que cuando una corriente eléctrica es enviada a través del metal, la resistencia de el metal a este flujo eléctrico, calienta este mismo. Por medio de la aplicacíón de suficiente corriente, la alta temperatura resultante puede producir temperaturas de fusión y hacer posible la soldadura.

El término soldadura por resistencia eléctrica incluye una variedad de aplicaciones de soldadura y es descrito bajo una variedad de nombres tales como soldadura de puntos, soldadura de disparo, soldadura con pistola, soldadura por centelleo, soldadura de perno, soldadura de espiga, soldadura por presión y otras.

Anteriormente se observó que si se utilizaba suficiente, corriente, los metales se volverían plásticos y después se fundirian. Si las dos piezas son prensadas juntas cuando sus superficies estan plásticas ó fundidas, las piezas se fusionarán en una misma pieza. Una máquina de soldadura por resistencia eléctrica es fundamentalmente un transformador eléctrico, operando a partir de un circuito de corriente alterna.

Para que la unidad soldadora pueda ejecutar la operación de soldadura, debe producir una corriente muy alta con un relativo bajo voltaje. Este requerimiento significa que el circuito primario tendrá muchas vueltas en el transformador, mientras que el arrollamiento secundario tendrá ordinariamente una sola vuelta.

La aplicación correcta de la soldadura por resistencia depende de la aplicación adecuada y control de las variables siguientes:

1) Corriente

2) Presión

3) Tiempo

4) Area de contacto del electrodo

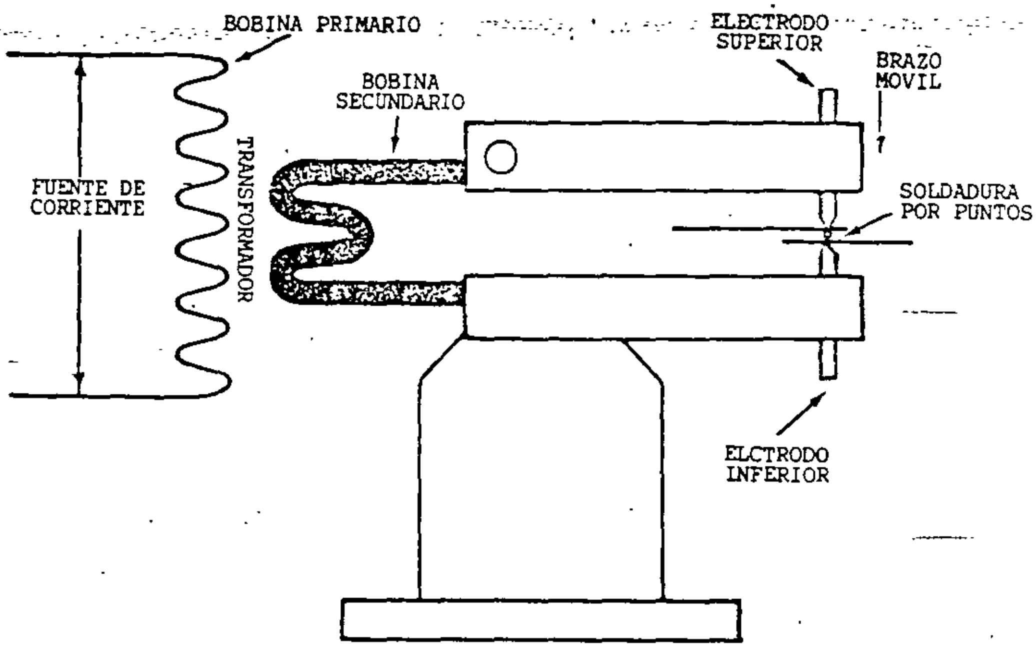


Fig. partes de una típica soldadora de resistencia por puntos.

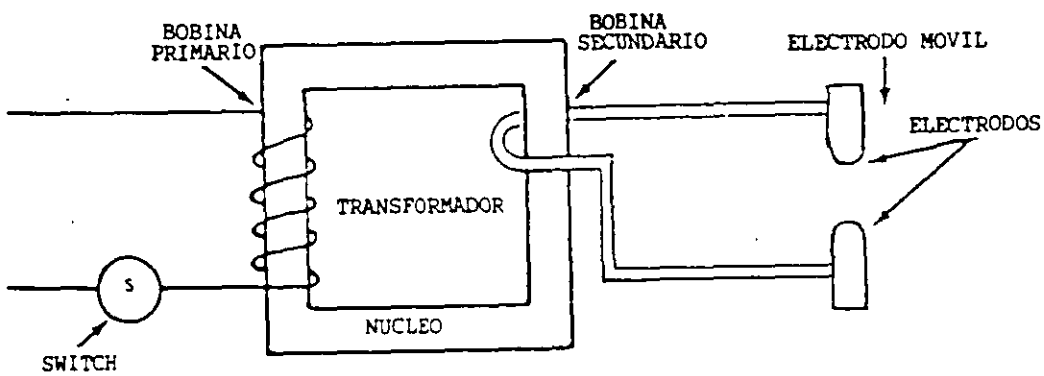
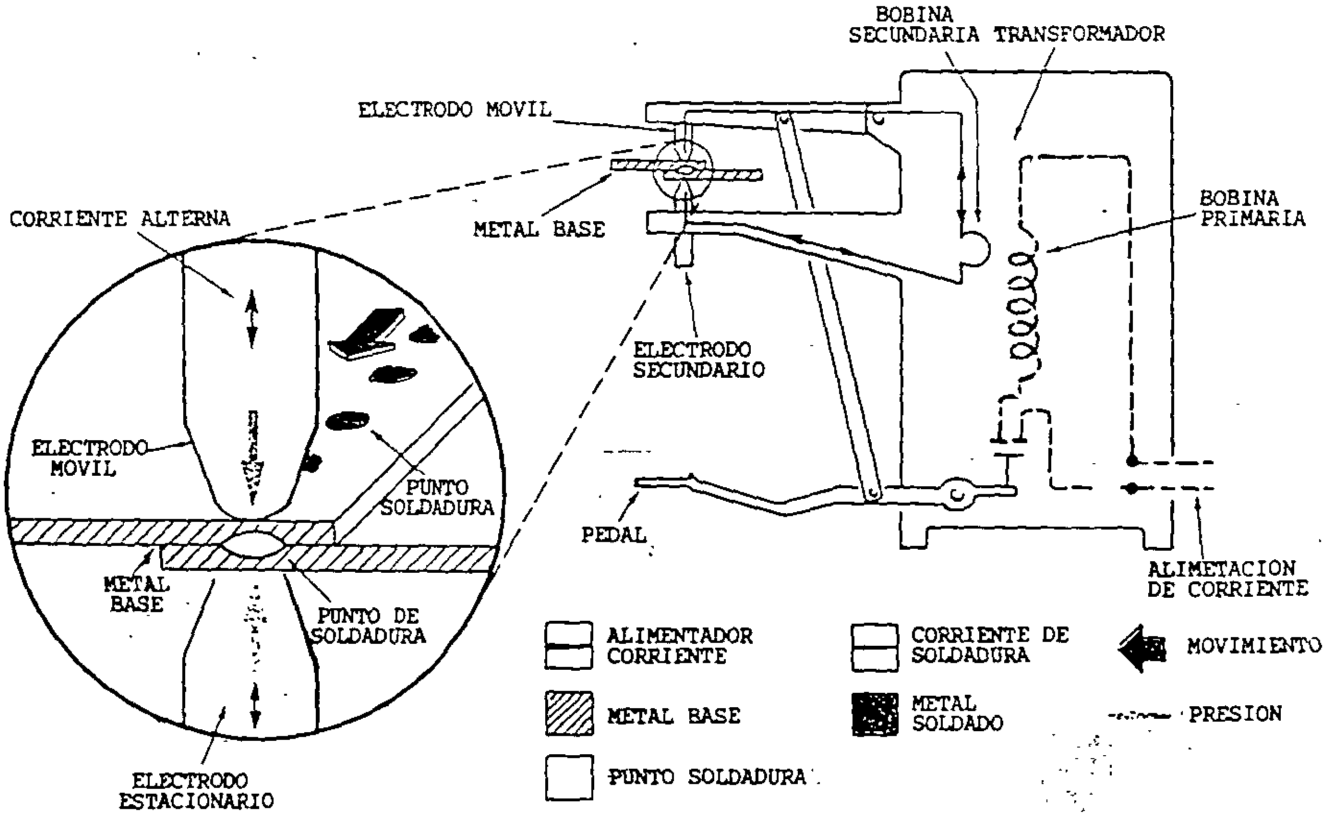


Fig. circuito básico de soldadura eléctrica por resistencia. Note que el devanado primario tiene mucho más vueltas que el devanado secundario. Este es un transformador reducido ó escalonado.



SOLDADURA POR RESISTENCIA (RSW)

SELECCION Y CLASIFICACION DE ELECTRODOS

La selección de electrodo apropiado para un trabajo dado es una de las más importantes decisiones que debe tomar el soldador.

Los electrodos se pueden diferenciar de las siguientes maneras:

1) Con recubrimientos ligeros y revestimientos pesados.

2) La composición química de la cubierta puede variar para dar los resultados deseados cuando se sueldan diferentes metales y aleaciones o para mejorar las características en posiciones diferentes:

3) El recubrimiento del electrodo puede ser diseñado para usarse en corriente directa (polaridad directa o invertida) o en corriente alterna.

4) La composición del metal del electrodo.

5) Los diámetros deseados de los electrodos.

Los electrodos con revestimiento pesado normalmente produciran soldaduras de resistencias superior y buena apariencia, pero son más caros.

Los electrodos estan diseñados y designados por códigos de color y números, tales como E6010, E6011, etc., cada electrodo tiene cualidades las cuales puede hacerlo más deseable que algún otro para un trabajo en particular.

ELECTRODOS METALICOS

Existen muchos tipos de electrodos metálicos. Una de las formas más comunes de clasificar los electrodos es por el recubrimiento sobre el electrodo.

Esto incluye:

1) Electrodos desnudos

2) Electrodos polveados

3) Electrodos sumergidos en flux

4) Eletrodos extruidos y cubiertos

De estos tipos, el electrodo con una cubierta extruida o sumergidos en flux, es el más popular. El electrodo desnudo es el menos caro. Para aplicación de soldadura en aceros para alta temperatura, aceros para herramientas, aceros al Molibdeno y para soldadura resistente en aceros suaves, los electrodos recubiertos son comunmente utilizados.

Los electrodos más comunes en relación al tamaño de la varilla (dimensiones) son: 1/8, 5/32, 3/16, 7/32, 1/4, 5/16 y 3/8 en diámetro. Estas varillas vienen en una longitud de 14 pulgadas para todos los tamaños y pueden también en algunos diámetros, conseguirse en 18 pulgadas.

La mayoría de los electrodos son fabricados de acero suave pero también se fabrican en metales aleados:

1) Acero suave

2) Acero de baja aleación

3) Acero al niquel

4) Acero al Cromo-Molibdeno

5) Acero Molibdeno-Manganeso

6) Acero Molibdeno-Niquel-Manganeso

7) Acero Niquel-Molibdeno-Vanadio

8) Aluminio

9) Cobre-Aluminio

10) Bronce-Plomo

11) Bronce-Fósforo

CLASIFICACION DE LOS ELECTRODOS

La American Welding Society (AWS) ha desarrollado unas series de clasificaciones de números de identificación.

La letra E precediendo los cuatro ó cinco números dígitos (EXXXXX) indica - un electrodo para utilizarse en soldadura de arco. Esto es en contraste con las letras RG, las cuales indican una varilla de soldadura usada para soldadura con gas. El significado de los números dígitos en la AWS es como sigue: Los primeros dos ó tres dígitos de los cuatro ó cinco dígitos (E60XX) ó (E-100XX) representan el esfuerzo a la tensión. Esto es, 60 significa ---

60,000 libras por pulgada cuadrada y 100 significa 100,000 libras por pulgada cuadrada. El esfuerzo a la tensión puede estar dado en la condición — "Tal como se soldo" ó "Relevada de esfuerzos". Deberán consultarse las especificacones de los fabricantes para determinar bajo que condiciones esta indicado el esfuerzo a la tensión. "Tal como se soldo" significa sin postcalentamiento. "Relevada de esfuerzos" significa que la soldadura deberá llevar un tratamiento térmico después de terminada para aliviar los esfuerzos causados durante la soldadura.

El segundo dígito de la derecha indica la posición recomendada de la junta para la cual el electrodo esta diseñado para soldar. Por ejemplo EXX1X; --- Este electrodo soldará en todas las posiciones; EXX2X significa que este — electrodo debe utilizarse en plano ó en posición horizontal; el EXX3X indica que el electrodo es recomendado para soldaduras en posición plana unicamente.

Los dígitos de más a la derecha, indican el tipo de suministro de energía - (corriente DIRECTA POLARIDAD DIRECTA O INVERTIDA, CORRIENTE ALTERNA), el -- tipo de recubrimiento y presencia de polvo de hierro o características de - bajo hidrógeno o ambos.

Los últimos dígitos deberán ser observados juntos para determinar la aplicación adecuada y la composición del recubrimiento para un electrodo.

Por ejemplo:

NUMERO DE ELECTRODO	COMPOSICION DEL RECUBRIMIENTO
EXX10	Celulosa alta, Sodio
EXX11	Celulosa alta, Potasio
EXX12	Titanio alto, ó Rutilio, Sodio
EXX13	Titanio alto, ó Rutilio, Potasio
EXX14	Polvo de hierro, Titanio
EXX15	Bajo hidrógeno, Sodio
EXX16	Bajo hidrógeno, Potasio
EXX18	Polvo de Hierro, bajo Hidrógeno
EXX20	Alto oxído de Hierro
EXX24	Polvo de Hierro, Titanio
EXX27	Polvo de Hierro, Oxído de Hierro
EXX28	Polvo de Hierro, bajo hidrógeno

Ocasionalmente, un número de electrodo puede tener una letra y número después de los cuatro números normales tales como E-7010-A1 ó E-8016-B2. Esta combinación ó sufijo de letra y número se utiliza para los electrodos con acero de baja aleación. El sufijo indica la composición del metal depositado.

A1	1/2% Molibdeno
B1	1/2% Cromo, 1/2% Molibdeno
B2	1 1/4% Cromo, 1/2% Molibdeno
B3	2 1/4% Cromo, 1% Molibdeno
C1	2 1/2% Niquel
C2	3 1/4% Niquel
C3	1% Niquel, .35% Molibdeno, .15% Cromo
D1 y D2	.25 a .45% Molibdeno, 1.25 a 2.0% Manganeso
G	.50 min. de Niquel, .30 min. de Cromo, .20 min. de Molibdeno, .10 min. de Vanadio

La letra A indica un acero al carbón Molibdeno. La letra B designa a un electrodo al Cromo-Molibdeno. La letra C es para un electrodo al Niquel y la letra D es para electrodos al Manganeso-Molibdeno. El dígito final indica en el sufijo, determina la composición química bajo una de estas clasificaciones químicas. La composición química exacta se puede obtener del Fabricante del electrodo.

Un ejemplo de una clasificación completa de un electrodo es el E-8016-B2.

1) E indica electrodo para arco eléctrico.

2) 80 indica que su esfuerzo a la tensión es de 80,000 libras por pulgada cuadrada.

3) 16 indica que puede utilizarse en todas las posiciones; que el recubrimiento contiene bajo Hidrógeno y Potasio.

4) El 1 indica que es un electrodo para todas las posiciones con corriente alterna ó corriente directa en polaridad invertida.

5) El sufijo B-2 indica que la composición química del metal depositado es un acero de baja aleación al Cromo-Molibdeno, con 1 1/4% de Cromo y 1/2% de Molibdeno.

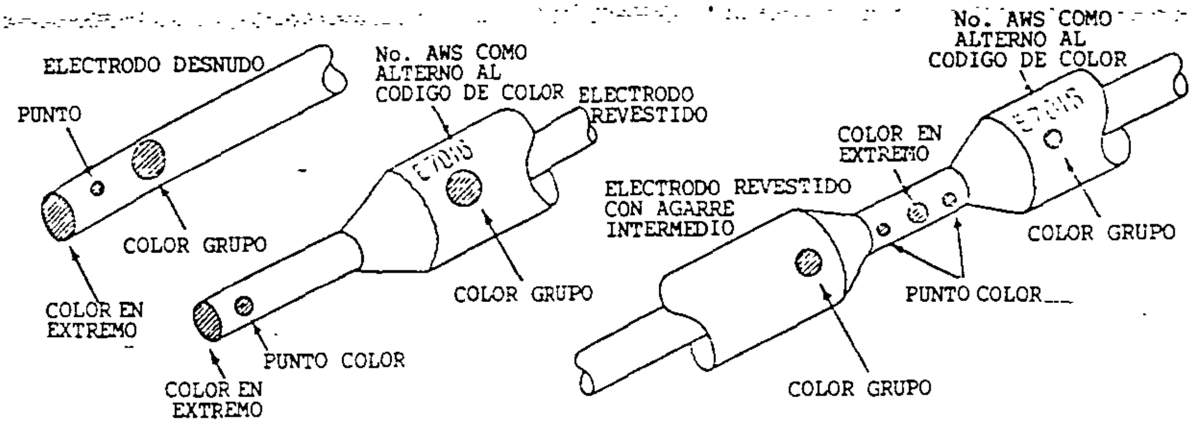


Fig. ______ la AMERICAN WELDING SOCIETY (AWS) ha estandarizado un sistema de numeración para identificar los electrodos para soldadura. Este número de electrodo esta colocado sobre la cubierta cerca de la punta del electrodo. Algunas compañías aún utilizan el código de color para identificarlos:

ELECTRODE AWS NUMBERS	POSITION	USE	TYPE CURRENT USED
E 4520	all		DCSP
E 6010	all	Penetration	DCRP
6011	all	Penetration	DCRP or AC
6012	all	Production	DCSP or AC
6013	all	Sheet Metal and Fillers	DCSP, DCRP or AC
6020 (iron oxide)	H (Fillets)		DCSP or AC
	F		DCSP, DCRP or AC
6027 (iron powder)	H (Fillets)		DCSP or AC
	F		DCSP, DCRP or AC
E 7010	all		DCRP
7011	all		DCRP or AC
7014 (iron powder)	all		DCSP DCRP or AC
7015 (low hydrogen)	all		DCRP
7016 (low hydrogen)	all		DCRP or AC
7018 (iron powder, low hydrogen)	all		DCRP or AC
7020	H (Fillets)	Chrome-Moly Steel	DCSP or AC
	F		DCSP, DCRP or AC
7024 (iron powder)	H (Fillets)		
	F		DCSP DCRP or AC
7027 (iron powder)	H (Fillets)		DCSP or AC
	F		DCSP, DCRP or AC
7028 (iron powder low hydrogen)	H (Fillets)		
	F		DCRP or AC
E 7010-X	all		DCRP
7011-X	all		DCRP or AC
7014-X (iron powder)	all		DCSP, DCRP or AC
7015-X (low hydrogen)	all		DCRP
7016-X (low hydrogen)	all		DCRP or AC
7018-X (iron powder low hydrogen)	all		DCRP or AC
7020-X	H (Fillets)	Chrome-Moly Steel	DCSP or AC
	F		DCSP, DCRP or AC
7024-X (iron powder)	H (Fillets)		
	F		DCSP DCRP or AC
7027 X (iron powder)	H (Fillets)		DCSP or AC
	F		DCSP DCRP or AC

ELECTRODE AWS NUMBERS	POSITION	USE	TYPE CURRENT USED
7028-X (iron powder, low hydrogen)	H (Fillets)		
	F		DCRP or AC
E 8010-X	all	Chrome-Moly Steel	DCRP
8011-X	all	Chrome-Moly Steel	DCRP or AC
8013-X	all		DCSP, DCRP or AC
8015-X (low hydrogen)	all		DCRP
8016-X (low hydrogen)	all	Nickel Alloy	DCRP or AC
8018-X (iron powder, low hydrogen)	all		DCRP or AC
E 9010-X	all	Chrome-Moly Steel	DCRP
9011-X	all	Chrome-Moly Steel	DCRP or AC
9013-X	all		DCSP, DCRP or AC
9015-X (low hydrogen)	all		DCRP
9016-X (low hydrogen)	all		DCRP or AC
9018-X (iron powder low hydrogen)	all		DCRP or AC
E 10010-X	all	Chrome-Moly Steel	DCRP
10011-X	all		DCRP or AC
10013-X	all		DCSP, DCRP or AC
10015-X (low hydrogen)	all		DCRP
10016-X (low hydrogen)	all	Nickel Alloy	DCRP or AC
10018-X (iron powder, low hydrogen)	all		DCRP or AC
E 11015-X (low hydrogen)	all		DCRP
11016-X (low hydrogen)	all		DCRP or AC
11018-X (iron powder, low hydrogen)	all		DCRP or AC
E 12015-X (low hydrogen)	all		DCRP
12016-X (low hydrogen)	all	Nickel Alloy	DCRP or AC
12018-X (iron powder low hydrogen)	all		DCRP or AC

NOTE: The suffix X stands for the weld metal chemical composition. See Fig. 6-31

Fig. _____ clasificación de varios electrodos AWS y sus posiciones recomendadas, aplicaciones y polaridad para utilizar cada uno de ellos.

CLASIFICACION DEL ELECTRODO	COLOR DE PUNTA	COLOR DE PUNTO	COLOR DE GRUPO
E 6010			
E 6011		AZUL	
E 6012		BLANCO	
E 6013		CAFE	
E 6020		VERDE	
E 7010 A-2	AZUL	BLANCO	
E 7016	AZUL	NARANJA	VERDE
E 7018	NEGRO	NARANJA	VERDE
E 8016 B-2	BLANCO	NEGRO	VERDE
E 9016 B-3	CAFE	AZUL	VERDE
E 10016	VERDE	NARANJA	VERDE

Fig. _____ tabla que muestre el código de color usado en la punta de los -- electrodos metálicos.

ELECTRODOS DE BAJO HIDROGENO

El hidrógeno tiene efectos dañinos sobre los aceros aleados, que causan --- fracturas intergranulares llamadas EMBRITTLEMENT de hidrógeno, lo anterior disminuye la resistencia a la fatiga y al esfuerzo.

Los electrodos de bajo hidrógeno depositan un mínimo de hidrógeno en la soldadura. Estos electrodos pueden ser usados con corriente directa polaridad invertida o con corriente alterna. Estos electrodos deberán ser horneados a 250° F antes de usarlos.

LECTURAS DE CORRIENTE PARA ELECTRODOS DE BAJO HIDROGENO

DIAMETRO DEL ELECTRODO	AMPERES EN POSICION PLANA	AMPERES SOBRE CABEZA	VOLTAJE
1/8	140 - 150	120 - 140	22 - 26
5/32	170 - 190	160 - 180	22 - 26
3/16	190 - 250	200 - 220	22 = 26
7/32	260 - 320		24 - 27
1/4	280 - 350		24 - 27
5/16	360 - 450		26 - 29

ELECTRODOS CON POLVO DE HIERRO

La adición con polvo de hierro a las cubiertas del recubrimiento de los --- electrodos de arco, cambia el arco favorablemente. Se incrementa en buena - cantidad el metal depositado. Corrientes mucho más altas pueden ser usadas para producir soldaduras más rápidas, soldaduras más faciles de limpiar, -- menor chisporroteo, y mejor forma de las capas de soldadura.

	E7024	E7024	E7018	E7016
PORCENTAJE DE POLVO DE HIERRO	65	50	33	0
CANTIDAD DEPOSITADA LB/HR	14	6	3	2
PORCENTAJE DE EFICIENCIA	190	160	130	80

ELECTRODOS DE CARBON

Los electrodos de carbón se usan para corte. Estos electrodos vienen en un rango de dimensiones desde 1/16" hasta 1" de diámetro. Las varillas pueden ser obtenidas en 12", 18" y 24" de longitud. La calidad de la varilla debe ser extremadamente alta, así como la estructura del carbón debe estar uniforme. Los dos tipos de electrodos obtenibles son los electrodos de carbón y los de grafito. El grafito tiene mejor conductividad y es usualmente de calidad más uniforme.

La varilla debe ser insertada en el porta electrodo con la punta del carbón aproximadamente 10 veces el diámetro de la varilla alejado del porta electrodo.

En la siguiente tabla se dan algunos requerimientos de corriente para electrodos de carbón.

DIAMETRO DEL ELECTRODO PULGADAS	CORRIENTE DE SOLDADURA MIN.	MAX.	MAXIMA DENSIDAD DE CORRIENTE AMPERES POR PULG.	LIBRAS DEPOSITADAS POR HORA
1/8	0	35	2980	
3/16	25	60	2200	
1/4	50	90	1855	
5/16	80	125	1650	
3/8	110	165	1510	
7/16	140	210	1420	1.5
1/2	170	260	1340	2.5
5/8	230	370	1220	4.5
3/4	290	490	1125	6.0
7/8	350	615	1035	
1	400	750	965	

TENSIONES Y DEFORMACIONES EN LAS SOLDADURAS

PRINCIPIO DE LAS TENSIONES DE SOLDADURA

La dilatación lineal de los cuerpos se rige por la fórmula $L = L_o \propto AT$; $\propto$ es el coeficiente de dilatación y es específico para cada material. Asi por ejemplo el coeficiente de dilatación entre 0 y 200°C es de 0,012 mm/m y °C, siendo la del aluminio aproximadamente el dobre y la del cobre una vez y media.

Al soldar habremos de tener en cuenta otros parámetros específicos como son el punto de fusión, calor específico, conductividad, límite elástico y módulo elástico que influyen de igual manera en las tensiones y encogimiento.

Un caso similar se produce en las tensiones transversales de la soldadura. Mientras el material de aportación se contrae, los flancos de soldadura tratan de separarse por la dilatación. Al estar fuertemente unidos los biseles con el material depositado, se contraen hacia el centro teórico de la soldadura.

El ancho total de las piezas unidas en frio es menor que antes de soldar
La contratación será mayor cuando mayor sea el calor introducido, menor sea la velocidad de la soldadura y mayor la zona afectada por el calor.

Tensiones en la soldadura.

TENSIONES Y DEFORMACIONES

Las tensiones y deformaciones por soldadura son debidas a la introducción de calor de un material.

El calor introducido por la soldadura no se puede utilizar totalmente para la fusión del material sino que parte se transmite al medio ambiente por conducción.

De esta forma, al introducir calor en un material por soldadura, parte de él sirve para fundir pero una gran parte se transmite al resto del material y otra se disipa en el aire.

Al soldar dos placas con biseles rectos entre sí, los puntos de igual temperatura en un momento determinado forman elipses (isotermos).

Cada punto afectado por el calor sufre una subida y una bajada de temperatura alcanzando las temperaturas máximas las zonas mas cercanas a la soldadura.
El incremento de la velocidad de soldadura supone un estrechamiento de los elipses isotérmicos, siendo menor la zona afectada por el calor y por lo tanto el enfriamiento más rápido y la diferencia de temperaturas mas abrupta.

Actuando por una parte la dilatación y por otra la contracción, las zonas afectadas por el calor quedan sometidas a un movimiento mecánico.

Se producen por lo tanto, esfuerzos de tracción y compresión que llegan a un equilibrio en el momento en que el material se enfria.

Las tensiones interiores resultantes son mayores cuando mayor sea la fuerza que impida estos movimientos.

Terminada la soldadura, quedan en la pieza soldada tensiones ya que las contracciones no se pueden liberar nunca totalmente.

Si la pieza a soldar no se puede dilatar, se suelen producir deformaciones plasticas pero donde se producen la mayoria de las tensiones residuales es cuando los esfuerzos de contracción de la soldadura no pueden liberarse.

Se puede afirmar que es casi imposible soldar sin tensiones residuales.

En general, podemos decir que cuando mayor sean las deformaciones menor serán las tensiones residuales y viceversa.

Por este motivo, antes de comenzar la soldadura, tendremos que optar por una de las soluciones o deformaciones grandes con tensiones pequeñas o bien tensiones grandes y deformaciones pequeñas

CONTRACCION DE LA SOLDADURA:

Las contracciones en la soldadura se clasifican en :

contracciones transversales, contracciones longitudinales, contracciones axiales y contracciones angulares.

Aunque surgen al mismo tiempo, tienen forma y reacciones diferentes.

Se denominan contracciones transversales las resultantes en el sentido perpendicular al eje de la soldadura.

El factor de mayor influencia es sin duda el calor introducido en relación con el espesor a soldar. Como quiera que a espesores iguales la cantidad de calor a introducir es mayor cuando mas ancho sea el bisel de soldadura, habra que tener en cuenta la forma de éste.

El bisel ha de ser por lo tanto lo mas estrecho posible, por otra parte, también por razones de economía.

Las contracciones transversales originan las contracciones angulares debido a la forma de los biseles.

Las contracciones longitudinales se producen en el mismo eje de la soldadura.

La contracción axial se produce en el espesor de la soldadura y puede dar problemas en la soldadura de placas gruesas en relación con la rotura frágil.

MEDIDAS PARA MINIMIZAR LA TENSION Y DEFORMACION

Sabemos que las tensiones y contracciones de la soldadura son inevitables. Sin embargo, es posible minimizarlas con diseño y ejecución planificada al efecto. Para ello habremos de tener en cuenta los siguientes detalles :

a) Mínimo de soldadura
b) Reducir la introducción de calor
c) Reducir el material de aportación
d) Subdividir la construcción soldada (diseño)
e) Fijar las secuencias de soldadura
f) Precalentamiento (carbono equivalente)

a) Mínimo de soldadura

La menor construcción soldada es sin duda la que reduce a un mínimo la cantidad de soldaduras y consta de un mínimo de piezas.

b) Reducción de la introducción del calor

Para cada soldadura dependiendo del material, espesor, etc., habrá que elegir el procedimiento de soldadura adecuado para introducir el calor mínimo por unidad de tiempo.

c) Reducción del material de aportación

La elección del tipo de bisel para cada soldadura es de gran importancia.
En las soldaduras a tope elegiremos un bisel de poca abertura que puede ser de 60° para soldadura manual y menor para soldadura automática o semiautomática (arco sumergido, MIG, MAG).
La separación entre labios (GAP) será mínima con objeto de que la sección de soldadura se reduzca.

En las soldaduras a solape no se rebasará la medida de cálculo indicada en los planos, sino que se mantendrá estrictamente a lo indicado.

d) Subdividir la construcción en subconjuntos cuando se trata de soldar construcciones grandes; en el diseño ya se indicarán los subconjuntos a soldar con lo cual ahorraremos tiempo en el manejo y reduciremos a un mínimo las tensiones.

Se soldará desde dentro a afuera, primero las soldaduras a tope y después a solape, primero las cortas y después las largas, primero las transversales y luego las longitudinales.
En depósitos se soldarán primero las longitudinales y luego las circunferenciales.

e) En construcciones soldadas críticas, hemos de fijar la secuencia de cada cordón con objeto de reducir las tensiones o deformaciones.

f) Precalentamiento (carbono-equivalente)

Se denomina precalentamiento al calentamiento previo a la soldadura.

El precalentamiento no solo se recomienda sino que es imprescindible en muchos casos. Cuando soldamos una placa gruesa el calor introducido por la soldadura es absorbido por la masa de la placa rapidamente enfriando a velocidad crítica la zona afectada por el calor, pudiendo incluso formarse martensita.
En estas zonas de dureza extrema (400 - 750 HB) se forman grandes tensiones al actuar la contracción y no poder deformarse plásticamente.

Con un precalentamiento se consigue sobre todo :

- reducir la velocidad del enfriamiento
- reducir la temperatura diferencial entre material y base y soldadura.
- la posibilidad de fisuración en frio (300C) se reduce ya que se forman estructuras mas ductiles (menos martensita)
- las cargas de tracción y compresión transcurren más suaves y en una zona mas amplia
- el hidrógeno tiene mas tiempo para difundirse

La tendencia a formar zonas duras durante la soldadura depende de la composición química del acero, siendo el carbono el elemento que mas influencia tiene.

La dureza en la zona afectada por el calor no solo depende del porcentaje de carbono sino además de la velocidad de enfriamiento.

Con objeto de unificar de algún modo la influencia del carbono y otros elementos en la estructura de la soldadura y la zona afectada por el calor, se dedujo una fórmula que se denomina equivalente (E C = equivalente carbón; valor K).

$$EC = C + \frac{Mn}{6} + \frac{Ni}{15} + \frac{Mo}{4} + \frac{Cr}{5} + \frac{Cu}{13} + \frac{Si}{4} + \frac{P}{2} + \frac{V}{5}$$

El porcentaje de cada elemento resultante del análisis químico se introduce en la fórmula resultando un valor.

Las temperaturas de precalentamiento se rigen por el resultado de este carbono equivalente.

EC	Precalentamiento recomendado °C
$< 0,45$	100°C
0,45 - 0,60	100 - 200°C
$> 0,60$	250 - 350°C (o superior)

TRATAMIENTOS TERMICOS DE LAS SOLDADURAS

En muchos casos es necesario efectuar un tratamiento después de la soldadura con objeto de conseguir las propiedades óptimas de los aceros empleados o para el alivio de las tensiones que se han formado durante el proceso de soldadura.

En el sinóptico anexo tenemos un resumen de los tratamientos después de la soldadura.

De todos los tratamientos térmicos, los más importantes son sin duda, el normalizado y el distensionado.

El normalizado es un tratamiento térmico con un calentamiento un poco por encima (20 - 50°C).

El material sufre una transformación doble que consigue una recristalización completa. Se consigue pues, un grano homogéneo en todas las direcciones.

El distensionado es un tratamiento térmico por debajo del punto de transformación, inferior con un enfriamiento lento con objeto de que puedan aliviarse las tensiones.
No se produce ninguna transformación en la estructura del grano.

El temple es un tratamiento térmico con enfriamiento rápido de temperaturas superiores al punto de transformación superior. Este tratamiento térmico se emplea para conseguir durezas superficiales o totales altas.

Para el enfriamiento se utiliza agua, aceite, sales o aire.

El revenido es un tratamiento térmico para conseguir tenacidades altas con cargas de rotura constantes.

Se efectua a temperaturas inferiores al punto de tranformación inferior variando la velocidad de enfriamiento para conseguir los resultados óptimos.

Se recomienda efectuar todos los tratamientos térmicos de la pieza completa aunque hay excepciones en las que se puede efectuar un tratamiento local.

Tratamientos después de la Soldadura

- Térmicos
  - Recocido
    - Normalizado (20-50°C A3)
    - Distensionado (< A1)
  - Templado > Ac3 - agua, aceite, aire
  - Revenido después del temple < Ac
  - Bonificado > lim. elástico, estricción, resilencia (temple + revenido)
- Termomecánicos
  - Martilleado en caliente (Cu)
  - Distensionado a baja temperatura 230°C (tanques)
- Quimicomecánicos
  - Decapado
- Mecánicos
  - Martilleado
  - Por vibración
  - Prueba hidráulica

INSPECCION Y PRUEBAS DE SOLDADURAS

Una soldadura terminada no es siempre tan buena o tan mala como pueda parecer de acuerdo a su superficie. Debido a el incremento de altos estandares de producción, se requieren adecuados métodos de inspección y pruebas en las soldaduras.

Los métodos usados para determinar la calidad de una soldadura pueden ser divididos en dos clasificaciones generales :

1. Pruebas no destructivas
2. Pruebas destructivas

El método de soldadura utilizada, la forma del artículo y el tipo de metal, influyen el tipo de prueba o inspección requerido.

PRUEBAS NO DESTRUCTIVAS

Los métodos que pueden caer bajo esta clasificación incluyen :

1. Inspección visual
2. Inspección de partículas magnéticas
3. Inspección de líquidos penetrantes
4. Inspección de ultra sonido
5. Inspección de rayos X
6. Inspección de corriente de Eddy
7. Detección por espectógrafo de masas
8. Prueba de fuga de presión de aire
9. Prueba de fuga de gas halógeno

PRUEBAS DESTRUCTIVAS

Ciertos tipos de elementos soldados deben ser cortados y preparados mediante esmeril para determinar las diferentes propiedades físicas. Cuando la soldadura es destruída o dañada después de su uso, la prueba es denominada como destructiva.

Algunas pruebas destructivas son :

1. La prueba de tensión
2. Análisis químicos
3. Prueba de doblés
4. Prueba microscópica
5. Prueba macroscópica
6. Prueba de dureza
7. Prueba charpy
8. Prueba hidrostática para destrucción
9. Prueba de corteza

INSPECCION VISUAL

Una soldadura la cual no requiere tener una alta resistencia física puede ser inspeccionada para observar fracturas, inclusiones, contornos y otras cualidades visuales.
Este tipo de inspección es subjetivo por naturaleza y usualmente no es definitivo en sus límites de aceptabilidad.
Una plantilla puede ser usada para checar el contorno de la capa de soldadura. Utilizando el método de inspección visual, una inspección puede comparar una soldadura terminada con un estándar aceptado y pasar o rechazar una soldadura por el método de comparación únicamente.

INSPECCION DE PARTICULAS MAGNETICAS

Este método es el más efectivo en el chequeo de una soldadura cercana a una superficie. Es utilizado unicamente en materiales que pueden ser magnetizados.
Una solución líquida que contiene pequeñas partículas magnéticas, se rocia sobre la superficie que se va a checar y entonces el metal es sometido a un fuerte campo magnético. Estas partículas estan pintadas de rojo o negro y estan suspendidas en un fino vehículo de aceite. Cualquier falta de continuidad en o cerca de la superficie del metal cuando esta magnetizado crea

un polo magnético local norte y sur y atrae las partículas metálicas en la solución usada. Cuando el campo magnético es retirado, el inspector encontrará una concentración de partículas magnéticas en el área de cada defecto. Si las imperfecciones son encontradas estas son esmeriladas, la parte es nuevamente soldada y de nuevo se prueba.

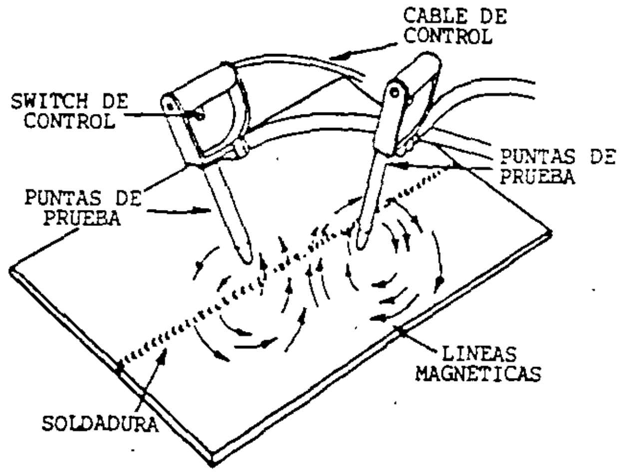


FIG. CAMPO MAGNETICO CREADO ALREDEDOR DE UNA SOLDADURA. LA CORRIENTE PASA A TRAVES DE LA SOLDADURA ENTRE LAS DOS PINZAS DE PRUEBA.

INSPECCION DE LIQUIDOS PENETRANTES

El método de inspección de líquidos penetrantes utiliza líquidos coloreados y líquidos flourescentes para checar fallas en la superficie. Este sistema puede ser utilizado para detectar fallas en la superficie de los metales, plásticos, cerámicas y vidrio. Este método no detectará defectos bajo la superficie.

El líquido penetrante es rociado sobre la superficie limpia que va a ser inspeccionada. Después de esperar un tiempo corto para que el líquido penetre. La cantidad excedente se limpia con un buen limpiador y se seca. Después de que la superficie está completamente seca, un revelador se rocia sobre la superficie el cual regresa el color del líquido penetrante que ha penetrado dentro de alguna fisura ó poro.

INSPECCION DE ULTRASONIDO

Un relativamente nuevo método de inspección de soldaduras es utilizar ondas de sonido de alta frecuencia. Esta técnica de prueba puede detectar defectos internos así como en la superficie. Una onda de sonido de alta frecuencia (onda de ultrasonido) es enviada dentro del metal por muy cortos periodos (1 a 3 microsegundos). Entonces la onda es detenida. La misma unidad la cual fué usada para enviar la onda de sonido actúa como receptor para escuchar la onda de ultrasonido tal como se refleja a través del metal.
El sonido nuevamente para, y su onda reflejada es recogida por el transreceptor.
Este ciclo es repetido de 1/2 a 5 millones de veces por segundo. Cada onda es visualmente representada por un osciloscopio. El osciloscopio está calibrado para recoger únicamente defectos de un tamaño los cuales pudieran considerarse dañinos. El patrón de onda del osciloscopio está también calibrado para mostrar la distancia entre la unidad rastreadora y cualquier defecto encontrado.

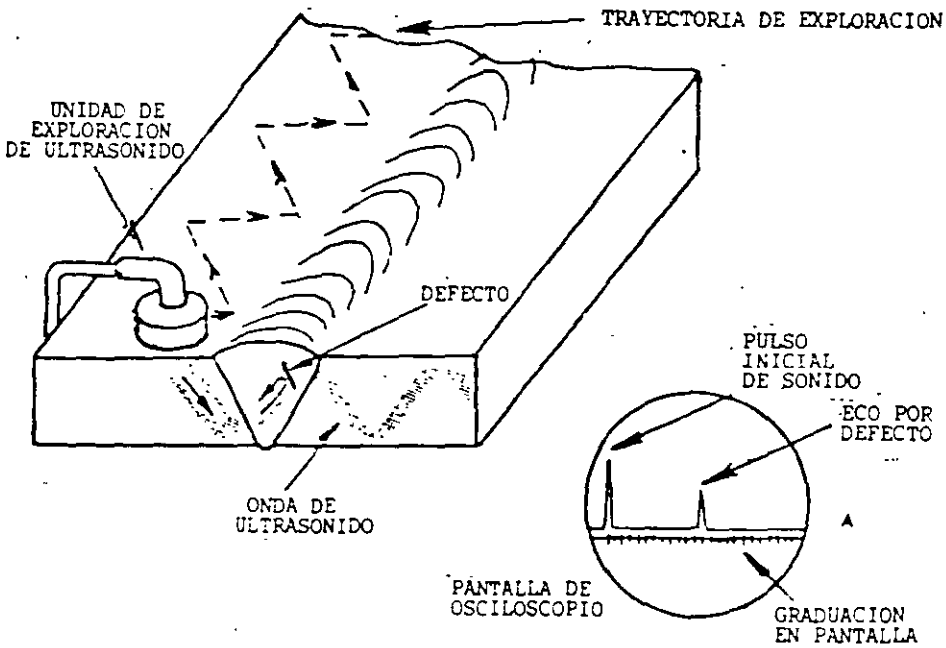


FIG. DIBUJO ESQUEMATICO MOSTRANDO EL CAMINO SEGUIDO POR LA UNIDAD DE RASTREO Y EL CAMINO DE LAS ONDAS DE SONIDO TAL COMO SE MUEVEN A TRAVES DEL METAL QUE ESTA SIENDO PROBADO. EL SONIDO INICIAL, TAMBIEN EL SONI-

DO DEL ECO, SE MUESTRAN EN EL OSCILOSCOPIO DE SONIDO EN UNA MANERA SIMILAR A COMO SE MUESTRA EN A. LA DISTANCIA ENTRE PICOS ES LA INDICACION DE CUAN LEJOS ESTA EL DEFECTO DE LA CABEZA DE LA UNIDAD DE RASTREO.

INSPECCION POR RAYOS X

El Rayo X es una onda de energía el cual pasa a través de la mayoría de los materiales y reproduce su imagen sobre una película (radiografía), sobre una pantalla fluorescente (fluoroscopia) o sobre una pantalla de televisión para ver alguna mancha remota.
La energía radioactiva puede ser producida electrónicamente en una máquina de Rayos X o por medio de isotopos radioactivos. El equipo que utiliza isotopos radioactivos es portátil y puede ser usado para checar soldaduras hechas en campo.
Algunos isotopos radioactivos populares son :

Cesio	137
Cobalto	60
Iridio	192
Samario	153
Talio	70

Las energías necesitadas para Rayos X varian considerablemente. El equipo está disponible desde 50 KV hasta 24000 KV, 140 KV radiografiarian 2 pulgadas de acero mientras que 24000 KV radiografiarian 20 pulgadas de acero.
Los defectos en una soldadura usualmente son facilmente vistos en una radiografía. Como quiera que sea, la profundidad a la cual el defecto está presente no puede ser determinada con unos Rayos X hechos a partir de una sola dirección.
El equipo requerido cuando se inspecciona por medio de Rayos X depende de :

1. Clase de material
2. Espesor por material
3. Accesibilidad de la parte a ser probada
4. La geometría de la parte a ser probada
   (Por ejemplo, una placa plana es más fácil de inspeccionar que un grupo de tubería).

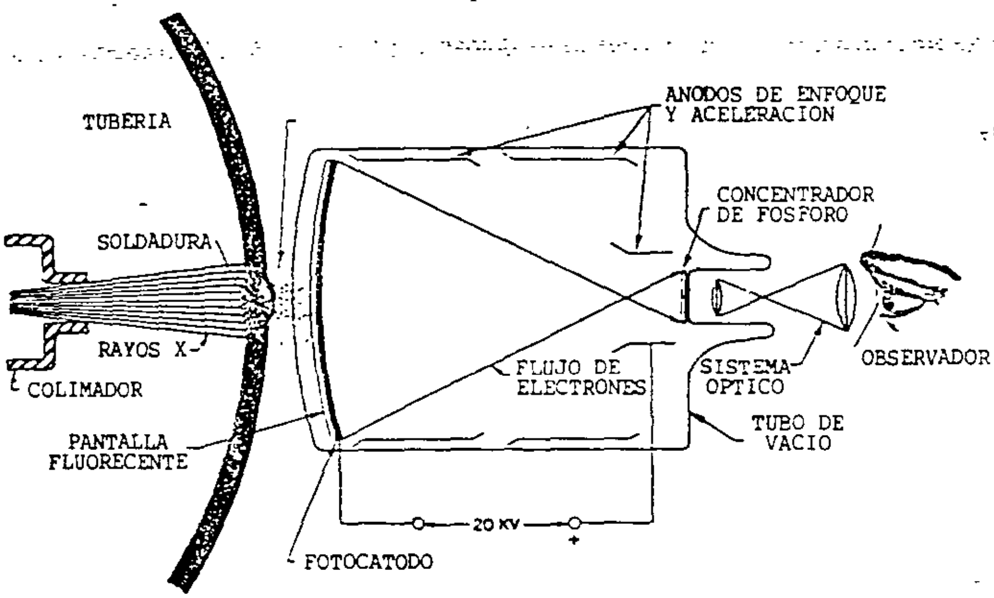


FIG. EXPLICACION DIAGRAMATICA DE UNA EXAMINACION FLUOROSCOPICA A UNA SOLDADURA DE TUBERIA.

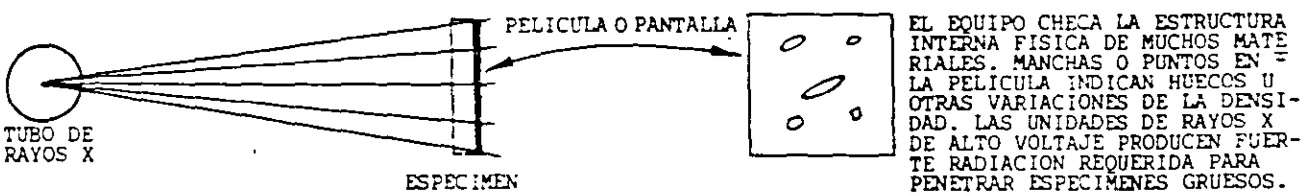


FIG. ESQUEMA DE UNA FOTOGRAFIA DE RAYOS X A UNA SOLDADURA.

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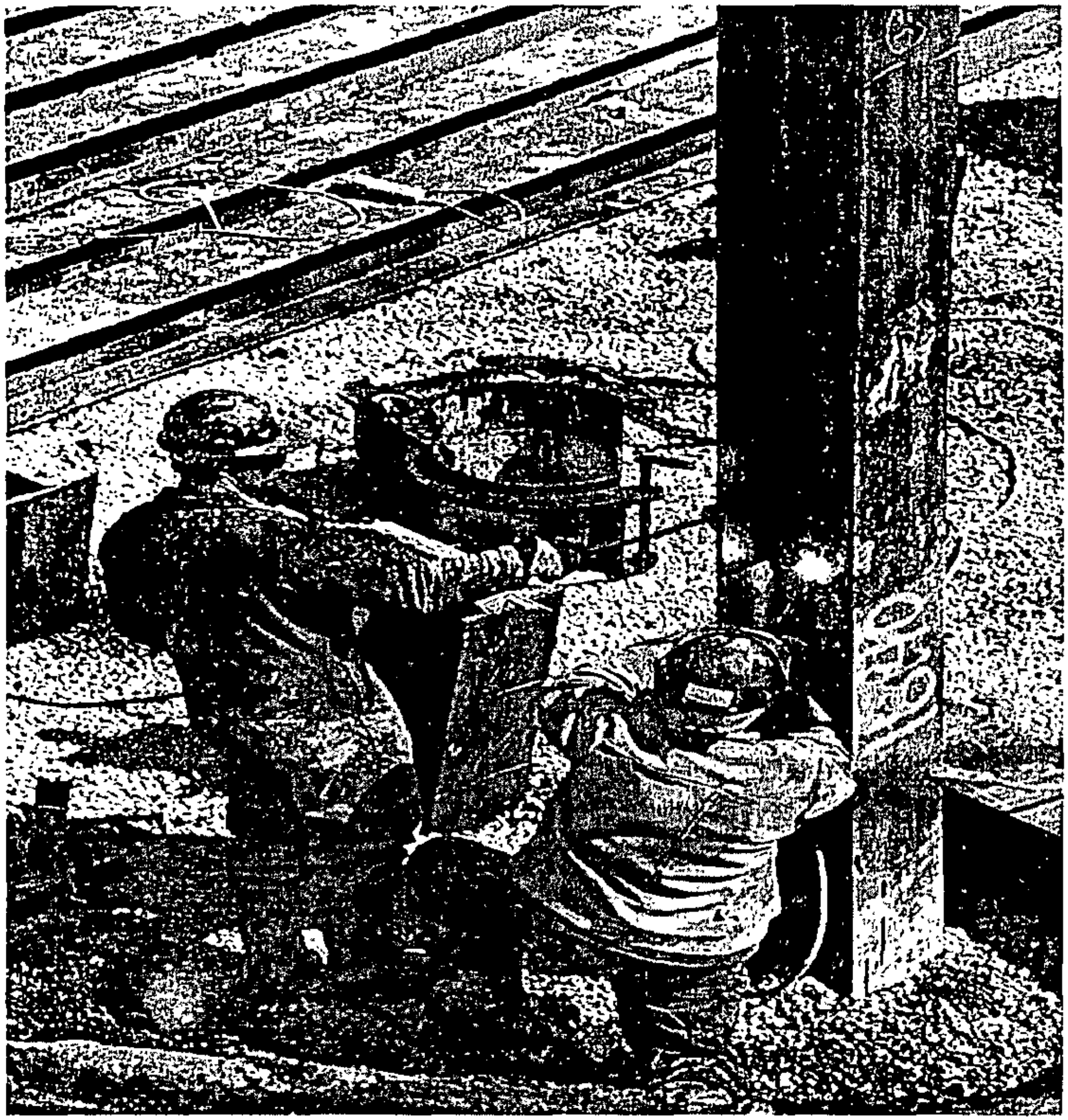
DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCION DE ESTRUCTURAS
MODULO IV "CONSTRUCCION DE ESTRUCTURAS DE ACERO"



OBJETIVO

CONOCER LA TERMINOLOGIA, BASES Y FUNDAMENTOS QUE APLICAN AL CONCEPTO DE LA SOLDADURA EN LA FABRICACION, ARMADO Y MONTAJE DE ESTRUCTURAS DE ACERO.

INDICE

SOLDADURA

EN LA TERMINOLOGIA METALMECANICA ES LA ACCION DE UNIR MEDIANTE FUSION TERMICA DOS O MAS ELEMENTOS METALICOS DE UNA COSA, PARA FORMAR UN TODO INTEGRADO. GENERALMENTE FUNDIENDO SU PROPIO MATERIAL O POR MEDIO DEL APORTE DE UN MATERIAL COMPATIBLE.

SOLDADURA

BASICAMENTE SE CONOCE DOS TIPOS DE SOLDADURA DENTRO DE LA TERMINOLOGIA DE LAS UNIONES METALICAS QUE SON:

- **UNIONES FUERTES TAMBIEN CONOCIDAS COMO SOLDADURAS DE PLATA YA QUE DENTRO DE SUS COMPONENTES SE ENCUENTRA ESTE METAL. (Plata, Cobre y Zinc)**

- **UNIONES BLANDAS O MALEABLES EN LAS QUE VARIANDO LOS COMPONENTES (Plomo, Estano, Bismuto, Cadmio, Antimonio, Fierro, etc) DEL APORTE DE LA SOLDADURA, PUEDEN LOGRARSE ALEACIONES QUE SE FUNDEN A TEMPERATURAS MAS CONVENIENTES, SEGUN EL OBJETO A QUE SE DESTINE.**

SOLDADURA

EN TODA SOLDADURA CUALQUIERA QUE SEA SU TIPO O FORMA DEBERA CUIDARSE QUE ESTAS SEAN HOMOGENEAS, EVITANDO PERDIDAS POR VOLATILIZACION Y OXIDACION.

ASI MISMO DEBERA VIGILARSE QUE LA FUSION DE LOS METALES BASE SEA HOMOGENEA EN EL CASO DE LA SOLDADURA SIN APORTE.

PARA LOS CASOS DE LA SOLDADURAS CON APORTE ES DE VITAL IMPORTANCIA, LA CORRECTA SELECCION DEL MATERIAL DE APORTE, EL CUAL DEBERA SER COMPATIBLE Y HOMOGENEIZAR CON EL O LOS ELEMENTOS DEL METAL BASE.

TEORIA DE LA SOLDADURA

CON OXIACETILENO

PROCESO TAMBIEN LLAMADO DE SOLDADURA AUTOGENA, BASADO EN LA COMBINACION DE UN GAS COMBUSTIBLE (Acetileno) Y UN GAS COMBURENTE (Oxigeno) LOS CUALES EN CONDICIONES Y CON EL EQUIPO ADECUADO, PRODUCIRA UNA FLAMA CON EL PODER CALORIFICO CAPAZ DE PRECALENTAR, SOLDAR Y CORTAR DIVERSOS METALES.

COMPONENTES DEL EQUIPO DE SOLDADURA AUTOGENA

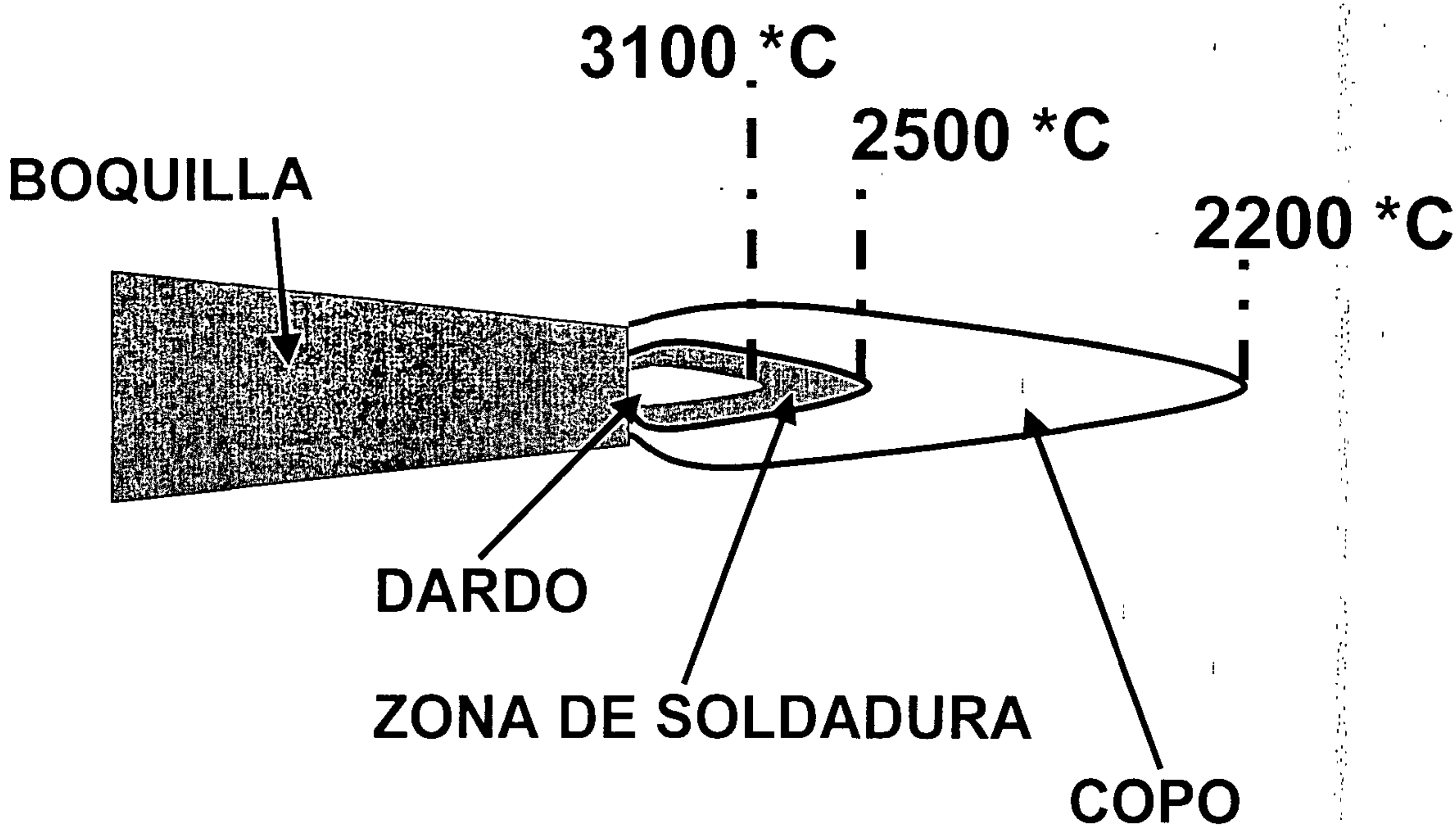
- CILINDOS (OXIGENO ACETILENO)
- REGULADORES (OXIGENO, ACETILENO)
- BLOQUEADORES DE RETROCESO
- MANGUERAS DE CAUCHO (C/Conectores)
- VALVULAS CHECK
- SOPLETE O MANERAL PARA SOLDAR
- MEZCLADOR
- BOQUILLA PARA SOLDAR

LA LLAMA OXIACETILENICA SE COMPONE PRINCIPALMENTE DE DOS ZONAS: EL DARDO Y EL COPO Y SE OBTIENE DE LA COMBUSTION DE OXIGENO Y ACETILENO EN PARTES CASI IGUALES.

EN LA TEORIA PARA QUEMAR UN LITRO DE ACETILENO SE REQUIEREN DOS LITROS Y MEDIO DE OXIGENO.

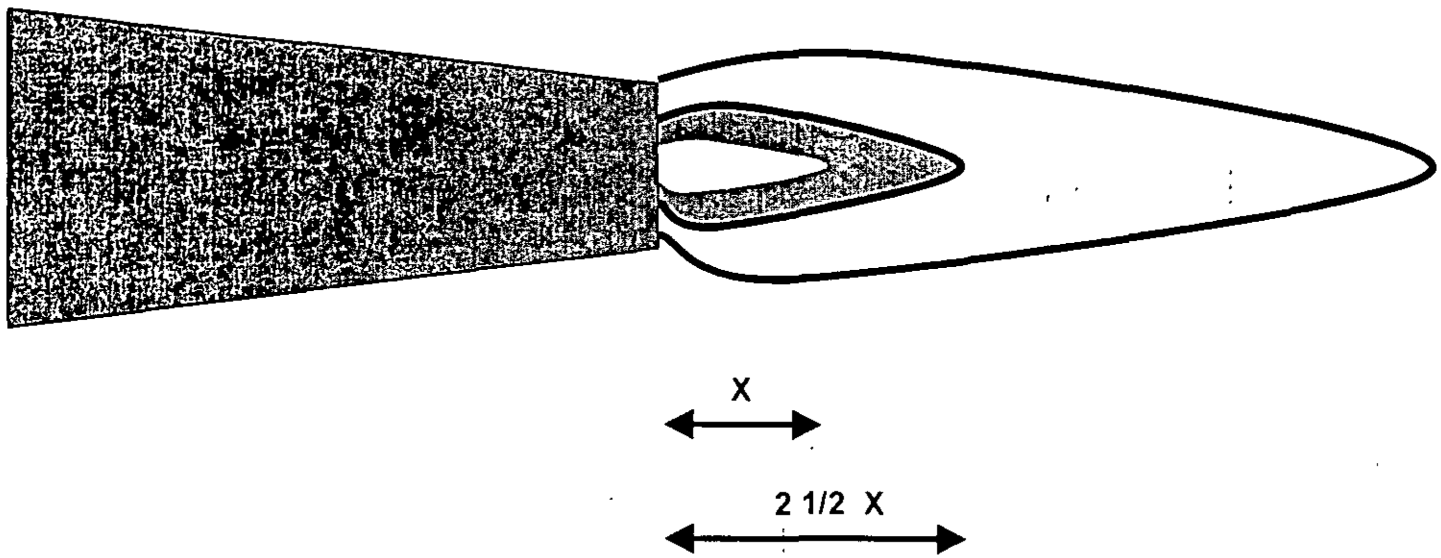
EN LA PRACTICA UNICAMENTE SE EXTRAERA UN LITRO DE OXIGENO DEL CILINDRO A PRESION YA QUE EL FALTANTE SERA TOMADO DEL MEDIO AMBIENTE CON LO CUAL SE COMPLEMENTA LA FORMACION DEL COPO.

TIPOS DE FLAMA



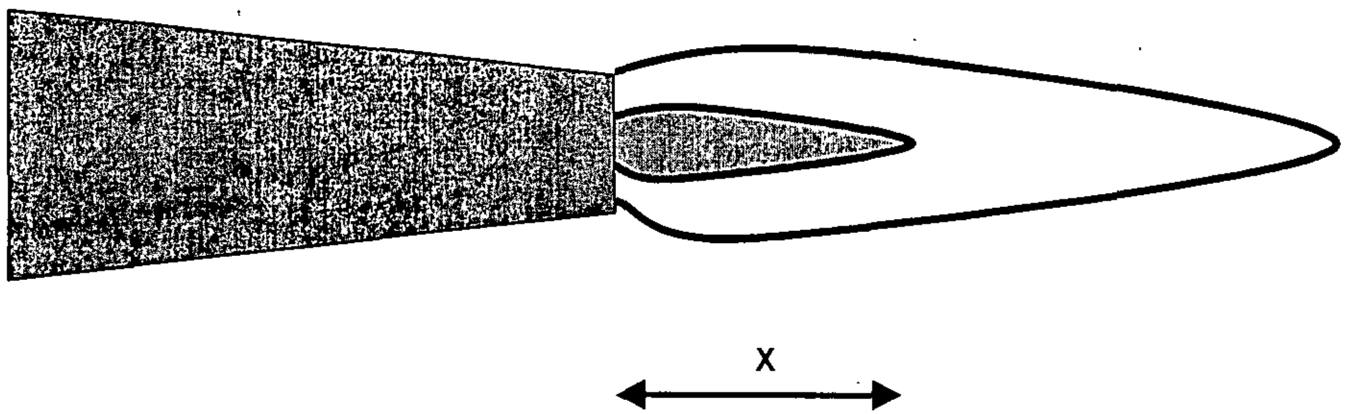
TIPOS DE FLAMA

FLAMA CARBURANTE
(Exceso de Acetileno)



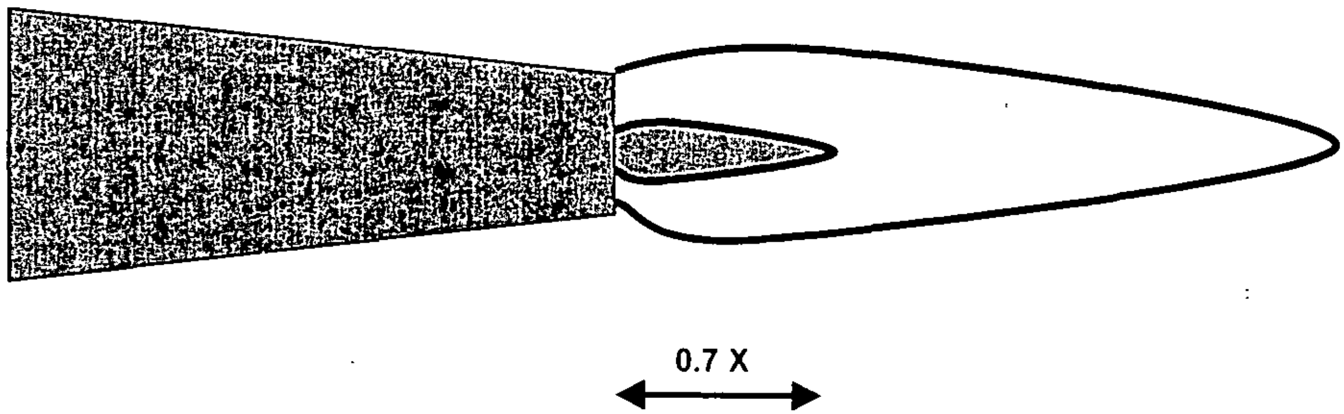
TIPOS DE FLAMA

FLAMA NORMAL
(Gastos Iguales de Oxigeno y Acetileno)



TIPOS DE FLAMA

FLAMA OXIDANTE
(Exceso de Oxigeno)



LA PRESION A LA QUE SE RECOMIENDA ALIMENTAR LOS GASES DE LOS CILINDROS HACIA EL SOPLETE, DEBERAN SER COMO MINIMO DE 1 lb/pulg.^{2} Y EN CIERTAS OCACIONES EL OXIGENO PODRA ALIMENTARSE HASTA A 25 lb/pulg.^{2} ACORDE A LOS REQUERIMIENTOS DE LAS BOQUILLAS.

CON LA FINALIDAD DE SUPERAR LA PROPAGACION DE LA LLAMA Y EVITAR QUE LA MEZCLA SE ENCIENDA EN EL INTERIOR DEL SOPLETE.

" LA VELOCIDAD DE LA MEZCLA DE GASES A LA SALIDA DE LA BOQUILLA DEL SOPLETE DEBERA DE SER POR LO MENOS DE 150 m/seg. "

PROCESO DE SOLDADURA AUTOGENA

• PREVIO A LA REALIZACION DE CUALQUIER SOLDADURA SE DEBERA VIGILAR QUE SE CUENTE CON LAS HERRAMIENTAS Y MATERIALES NECESARIOS PARA LA CORRECTA EJECUSION DE LOS TRABAJOS.

• SE DEBERA VIGILAR QUE EN LO POSIBLE LAS PARTES A SOLDAR COINSIDAN EN LO POSIBLE A LO LARGO DE LA JUNTA, PARA ELEMENTOS O PLACAS CON ESPESORES SUPERIORES A 1/4” ESTOS ELEMENTOS DEBERAN SER VISELADOS Y LA SOLDADURA EJECUTADA CON VARILLA DE APORTE.

• UNA VEZ FIJAS LAS PIEZAS POR MEDIOS MECANICOS (Tornillos o Soportes) SE PROCEDERA A ENCENDER EL SOPLETE REGULANDO LA FLAMA HASTA LA LLAMA NORMAL VIGILANDO QUE LAS PRESIONES EN EL MANÓMETRO A LA SALIDA DE LOS REGULADORES NO EXCEDAN LAS RECOMENDADAS CON ANTERIORIDAD.

PROCESO DE SOLDADURA AUTOGENA (cont. 2)

- **INICIAR EL PROCESO DE LA SOLDADURA COLOCANDO EL MANERAL EN UN ANGULO DE 30 A 45 GRADOS CON RELACION AL PLANO DE TRABAJO. COLOCANDO LA FLAMA EN DIRECCION DE LA SOLDADURA PRECALENTANDO EL METAL ANTES DE EXPONERLO A LA FLAMA DE ALTA TEMPERATURA.**

- **<u>FUNDICION DEL METAL BASE (Torcheo o Caldeo)</u> ESTE PASO ES FUNDAMENTAL EN LOS PROCESOS DE SOLDADURA BASADOS EN LA FUNDICION DE METALES, ESTA FUNDICION SE PRESENTA EN LA MAYORIA DE LAS FORMAS DE SOLDADURAS TANTO AUTOGENA COMO DE ARCO ELECTRICO. LAS CARACTERISTICAS Y DIMENSIONES (Diametro) DEL CALDEO DEL METAL FUNDIDO, ESTARA EN PROPORCION A LA PROFUNDIDAD O PENETRACION DE LA SOLDADURA. EL CALDEO DEBERA HACERSE SIEMPRE CON LA FLAMA NEUTRAL, PUDIENDOSE OBSERVAR UN CHARCO DE METAL FUNDIDO ESTABLE, SI EL CHARCO HIERVE O SALPICA EN EXCESO DEBERA HACERSE UN AJUSTE A LA FLAMA PARA MEJORAR LA CALIDAD DE LA SOLDADURA.**

PROCESO DE SOLDADURA AUTOGENA (cont. 3)

- EL AREA DE ALTA TEMPERATURA DE LA FLAMA NORMAL DEBERA ESTAR SIEMPRE EN CONTACTO CON EL CHARCO DE METAL FUNDIDO Y MANTENER UN MOVIMIENTO DE GIRO ATRAVEZ DE LA SUPERFICIE DEL CHARCO PARA MANTENER LA TEMPERATURA DE LOS METALES BASE Y GENERAR UNA FUSION UNIFORME. PARA EL CASO DE SOLDADURAS CON APORTE A ESTA ACCION SE DEBERA DE INCLUIR EL ACERCAMIENTO DE LA VARILLA DE APORTE A LA FLAMA DE SOLDEO PARA SU FUSION E INTEGRACION AL METAL BASE.

- UNA VEZ CONTROLADO EL CHARCO DE METAL FUNDIDO SE PROCEDE A AVANZAR EN DIRECCION A LA JUNTA DE SOLDADURA A UNA VELOCIDAD QUE COMUNMENTE ESTARA BASADA EN LA HABILIDAD DEL SOLDADOR, ESPESOR Y TIPO DE LOS MATERIALES.

- EL ADECUADO AJUSTE DE LA FLAMA DURANTE EL PROCESO DE SOLDADURA EVITARA QUE EL OXIGENO DE LA ATMOSFERA ENTRE EN CONTACTO CON LA SUPERFICIE DE LA ZONA DE CALDEO EVITANDO CON ELLO CONDICIONES DE OXIDACION EN LA JUNTA SOLDADA.

CORTE CON GAS Y

ARCO ELECTRICO=GAS

DENTRO DE LA INDUSTRIA METALMECANICA, EXISTEN VARIOS PROCESOS PARA EL CORTE DE METALES Y ELEMENTOS ESTRUCTURALES PARA LA FABRICACION DE ESTRUCTURAS METALICAS.

ENTRE LOS MAS USUALES ENCONTRAMOS LOS SIGUIENTES:

CORTE CON OXIGENO ACETILENO
ARCO ELECTRICO METAL
ARCO ELECTRICO AIRE
ARCO ELECTRICO PLASMA
LASSER

CORTE CON OXIGENO ACETILENO

- EL CORTE OXIACETILENICO SE PUEDE DIVIDIR EN DOS GRANDES GRUPOS.

  - METALES CON OXIDOS QUE TIENEN TEMPERATURA DE FUNDICION MAS BAJO AL DEL METAL BASE.

  - METALES CON OXIDOS QUE TIENEN TEMPERATURA DE FUNDICION MAS ALTO AL DEL METAL BASE.

- PRACTICAMENTE TODOS LOS ACEROS CAEN DENTRO DE LA PRIMERA CLASIFICACION, EXCEPTO EL FIERRO FUNDIDO, ACERO INOXIDABLE Y METALES NO FERROSOS.

CORTE CON OXIGENO ACETILENO

• EL CORTE CON OXIACETILENO ES EL MAS UTILIZADO EN EL PROCESO DE OBTENCION DE LOS ELEMENTOS ESTRUCTURALES PARA LA FABRICACION DE ESTRUCTURAS METALICAS A PARTIR DE PLACAS METALICAS O ELEMENTOS PREFABRICADOS.

• ESTE ES EL PROCESO MAS USUAL TANTO EN TALLER COMO EN EL CAMPO DE LA CONSTRUCCION YA QUE RESULTA SER EL DE MENOR NECESIDAD DE CAPACITACION Y SE CONJUGA CON OTRAS ESPECIALIDADES DEL OPERARIO.

• DURANTE ESTE PROCESO ES UTILIZADA UNA FLAMA DE OXIACETILENO PARA EL CALENTAMIENTO DEL METAL Y UN CHORRO DE OXIGENO ADICIONAL PARA REALIZAR EL CORTE.

• LA FLAMA DE OXIACETILENO, EN EL SOPLETE DE CORTE SERA SIMILAR A LA EMPLEADA PARA SOLDAR, CALENTANDO CON ESTA EL METAL HASTA LLEGAR A UNA TEMPERATURA APROXIMADA DE 1800 GRADOS FARENHEIT (Color rojo cereza).

CORTE CON OXIGENO ACETILENO

- **LOGRADO EL CALENTAMIENTO NECESARIO ES ABIERTA LA VALVULA DE PASO DE OXIGENO A PRESION LOCALIZADA EN EL SOPLETE DE CORTE, INICIADO ESTE PROCESO SE MOVERA EL SOPLETE HACIA EL EL OPERARIO REALIZANDO CON ESTE LA OPERACION DE CORTE.**

- **EL MANERAL DE CORTE ES MUY SIMILAR AL MANERAL DE SOLDAR SOLO QUE EL MANERAL DE CORTE TIENE UN CONDUCTO PARA EL CHORRO DE OXIGENO, MISMO QUE ESTA CONECTADO DIRECTAMENTE A LA BOQUILLA SIN MEZCLARSE CON EL ACETILENO.**

- **LA VARIACION EN LA PRESION DE ALIMENTACION DEL OXIGENO ESTA RELACIONADA CON EL ESPESOR DEL MATERIAL A CORTAR LA CUAL PODRA FLUCTUAR DESDE 20 A 50 lb/pulg^2 .**

CORTE CON ARCO ELECTRICO METAL

ESTE CORTE ES POCO USUAL YA QUE REQUIERE DE UNA MAQUINA SOLDADORA, ALTO AMPERAJE Y CONSUMO DE VARILLA ELECTRODO. ASIMISMO EL CORTE LOGRADO ES MUY IMPRECISO Y NO SE PRESTA PARA PLACAS DE ESPESORES SUPERIORES A 3/8" .

ESTE CORTE ES FACTIBLE DE SER UTILIZADO EN LA REMOCION DE PERNOS O HERRAJES AUXILIARES PARA ENSAMBLADO DE ALGUNAS ESTRUCTURAS O RECIPIENTES.

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CORTE ARCO ELECTRICO AIRE (ARC-AIR)

PARA ESTE PROCESO ES NECESARIO LA UTILIZACION DE UNA MAQUINA SOLDADORA, CABLES CON MANERAL PORTAELECTRODO PARA ARCO-AIRE Y UNA LINEA DE SUMINISTRO DE AIRE CON UNA PRESION MINIMA DE 30 lb/pulg.^{2}

ESTE SISTEMA SE UTILIZA MAS COMUNMENTE EN EL VACIADO DE CORDONES DE SOLDADURA EN REPARACIONES O CONSTRUCCION DE ALGUNOS RECIPIENTES EL LOS CUALES SE TIENEN JUNTAS SOLDADAS CON DOBLE BISEL Y A LOS CUALES SE TIENE QUE ELIMINAR LA RAIZ DE LA PRIMERA SOLDADURA.

CORTE ARCO ELECTRICO PLASMA

ESTE PROCESO ES COMUNMENTE UTILIZADO PARA CORTE DE LOS METALES DEL GRUPO DOS (Inoxidables y Fierro fundido) ASI COMO PLACAS DE METAL DE GRANDES ESPESORES (Hasta 7"). Y ESTA BASADO EN LOS SIGUIENTES PUNTOS:

• EL ARCO ELECTRICO ES GENERADO CON UNA BARRA DE TUNGSTENO Y LLEGA HASTA EL INTERIOR DE LA ANTORCHA, MISMA QUE ESTA INTEGRADA AL MANERAL DE CORTE POR PLASMA.

• CREADO EL ARCO SE PROCEDERA A PASAR UN FLUJO DE GAS INERTE A TRAVEZ DE LA ANTORCHA, MISMO QUE AL SER SOBRECALENTADO POR EL CONTACTO CON EL ARCO ELECTRICO SALE EXPULSADO A ALTAS TEMPERATURAS EN FORMA DE PLASMA LA CUAL ENTRA EN CONTACTO CON EL METAL A CORTAR Y GENERA UN CORTE LIMPIO Y DE GRAN PRESICION.

• ESTE PROCESO ES COMUNMENTE UTILIZADO EN TALLER POR REQUERIR INSTALACIONES DE SUMINISTRO DE AGUA Y MESAS DE TRABAJO ADECUADAS AL TRABAJO DE PRESICION.

CORTE CON RAYO LASSER

-101-

EL PROCESO DE CORTE DE METALES CON RAYO LASSER NO ES MUY COMUN EN EL AREA DE FABRICACION DE ESTRUCTURAS METALICAS, YA QUE POR SUS CARACTERISTICAS DE PRESICION Y SUS REQUERIMIENTOS DE ALTA TECNOLOGIA LO COLOCA AL ALCANCE DE LOS GRANDES FABRICANTES DE LOS EQUIPOS DE GUERRA O DEL PROGRAMA ESPACIAL DE LOS EEUU.

SEGURIDAD EN LA MEJORA DE LOS PROCESOS

- SIEMPRE QUE ESTE USTED A CARGO DE UNA ACTIVIDAD DE SOLDADURA O CORTE ASEGURESE DE QUE LAS CONDICIONES DEL MEDIO AMBIENTE, OPERACION DE EQUIPOS Y EJECUCION DE LOS TRABAJOS SERAN REALIZADOS BAJO CONDICIONES SEGURAS.

- ASEGURESE DE QUE AL REALIZAR CUALQUIER ACTIVIDAD DE CORTE O SOLDADURA SE CUENTE CON LAS HERRAMIENTAS ADECUADAS PARA ESAS ACTIVIDADES.

- EL PERSONAL DEDICADO A LAS ACTIVIDADES DE SOLDADURA DEBERA UTILIZAR INVARIABLEMENTE EL EQUIPO DE PROTECCION Y SEGURIDAD MARCADO EN LOS REGLAMENTOS Y MANUALES DEL CASO.

- RECUERDE SIEMPRE, QUE LA MEJOR INVERSION EN SU NEGOCIO, SERA LA SEGURIDAD DE SUS RECURSOS HUMANOS YA QUE LA VIDA NO ES UN PRODUCTO RENOVABLE.

SOLDADURA CON ARCO ELECTRICO

A ESTE PROCESO TAMBIEN SE LE DENOMINA SOLDADURA ELECTRICA, Y SE FUNDAMENTA EN LA GENERACION DE UN ARCO ELECTRICO, BASADO EN EL SALTO DE CORRIENTE PROVENIENTE DE UNA FUENTE DE PODER O MAQUINA SOLDADORA. EL ARCO FORMADO ES CAPAZ DE GENERAR UN PODER CALORIFICO CAPAZ DE FUNDIR LOS METALES Y GENERAR UNA SOLDADURA ENTRE LOS ELEMENTOS.

SOLDADURA CON ARCO ELECTRICO

DE ACUERDO A LA DEFINICION GENERICA DEL AWS LA SOLDADURA CON ARCO ELECTRICO "ES UN GRUPO DE PROCESOS DENTRO DE LAS CUALES LA UNION SE PRODUCE POR CALENTAMIENTO ELECTRICO, CON UN ARCO ELECTRICO O ARCOS, CON O SIN LA APLICACION DE PRESION, CON O SIN METAL DE RELLENO.

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LAS MAQUINAS SOLDADORAS O FUENTES DE PODER SE PUDEN CLASIFICAR EN TRES TIPOS BASICOS.

• SOLDADORA TRANSFORMADOR. (Corriente Alterna)
Maquina de reducidas dimenciones, sencillas de operar y de bajo precio. Limitado a cierto tipo de electrodos.

• SOLDADORA RECTIFICADOR. (Corriente Directa)
Maquina de dimenciones regulares, de tipo trifasico, basadas en bobinas por lo que ofrece altas capacidades de amperaje y polaridad. Amplia cobertura en los procesos de soldadura.

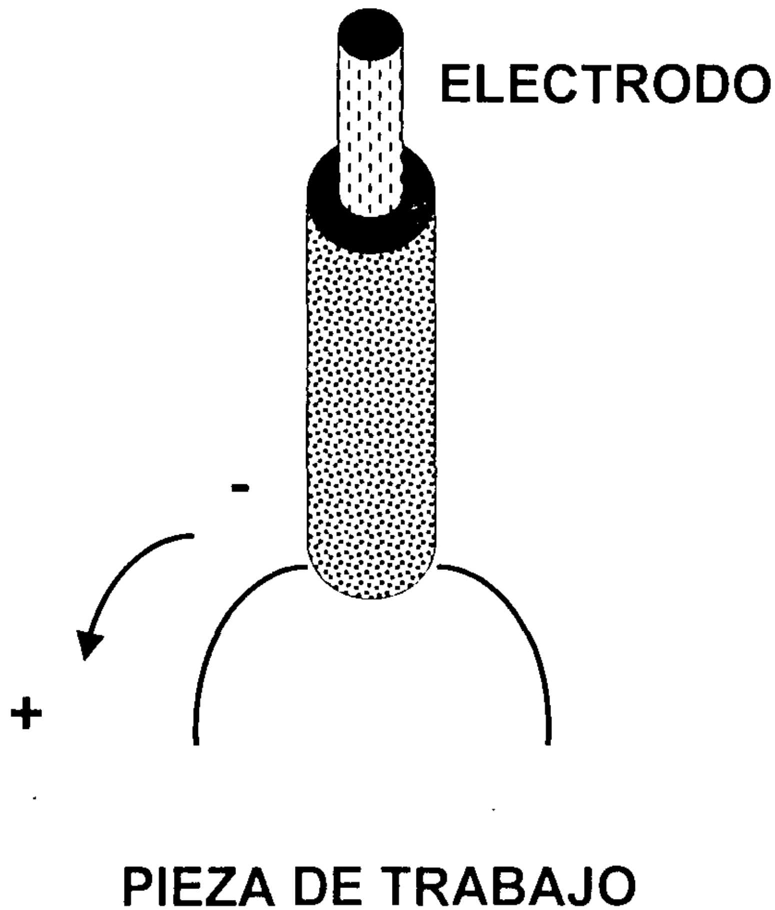
• SOLDADORA DE GENERADOR. (Corriente Continua)
Es una maquina que se basa en un motor electrico trifasico o motor de combustion diesel, acoplado a un generador de corriente continua que brinda amplio rango de amperaje y polaridad, brinda amplia cobertura en los procesos de soldadura. Es ideal para areas donde no existen lineas de corriente electrica disponibles.

POLARIDAD

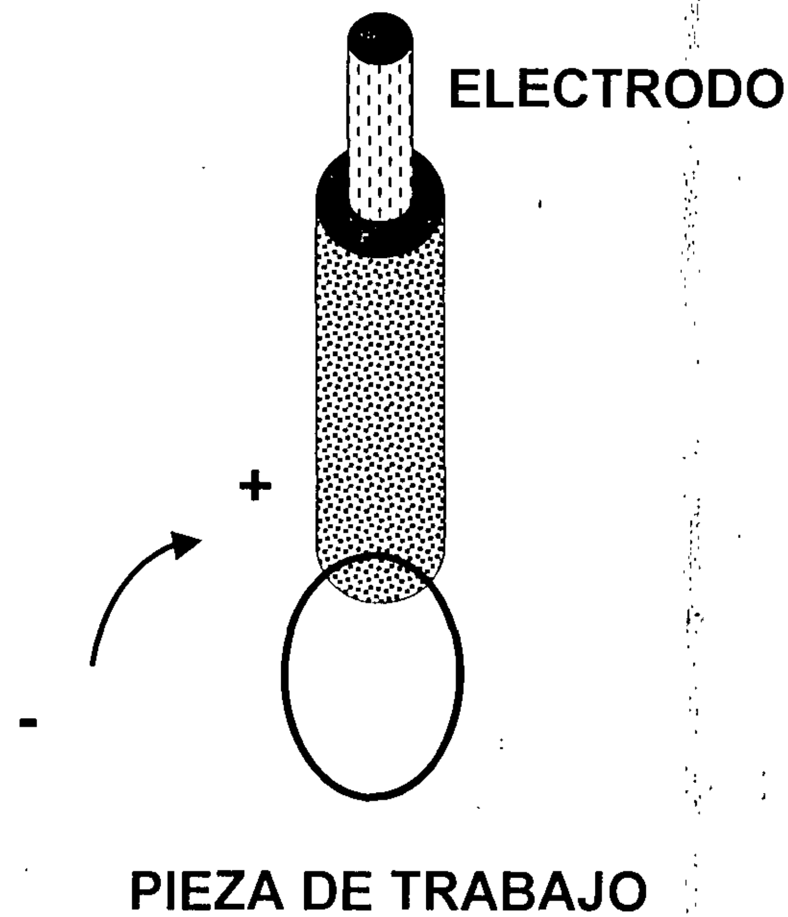
COMO SE PODRA NOTAR LA VENTAJA DE LOS GENERADORES Y RECTIFICADORES ES SU CAPACIDAD DE PROPORCIONAR LA CONEXION EN DIFERENTE POLARIDAD, MISMA QUE ES LA PROPIEDAD DE ESTABLECER EL VIAJE DE LOS ELECTRONES DE LA CORRIENTE, SOBRE EL METAL BASE Y EL ELECTRODO EN UN SENTIDO PREFERENCIAL QUE SIEMPRE SE LLEVARA A CABO DEL POLO NEGATIVO AL POLO POSITIVO DE LA MAQUINA.

SI CONECTAMOS EL CABLE DEL PORTAELECTRODO AL NEGATIVO DE LA MAQUINA, SE DICE QUE LA <u>POLARIDAD ES DIRECTA</u>.
SI POR EL CONTRARIO EL CABLE DEL PORTAELECTRODO SE CONECTA AL POLO POSITIVO, LA <u>POLARIDAD ES INVERTIDA</u>.

LA POLARIDAD MAS COMUNMENTE USADA ES LA POLARIDAD INVERTIDA LLAMADA POR LOS SOLDADORES "POLARIDAD NORMAL" YA QUE SU PRINCIPAL CARACTERISTICA ES CONCENTRAR EL CALOR DEL ARCO EN EL PUNTO DE APLICACION PRODUCIENDO UNA FUSION CONSIDERABLE DEL METAL BASE Y POR LO TANTO UNA GRAN PENETRACION.



POLARIDAD DIRECTA
O NEGATIVA

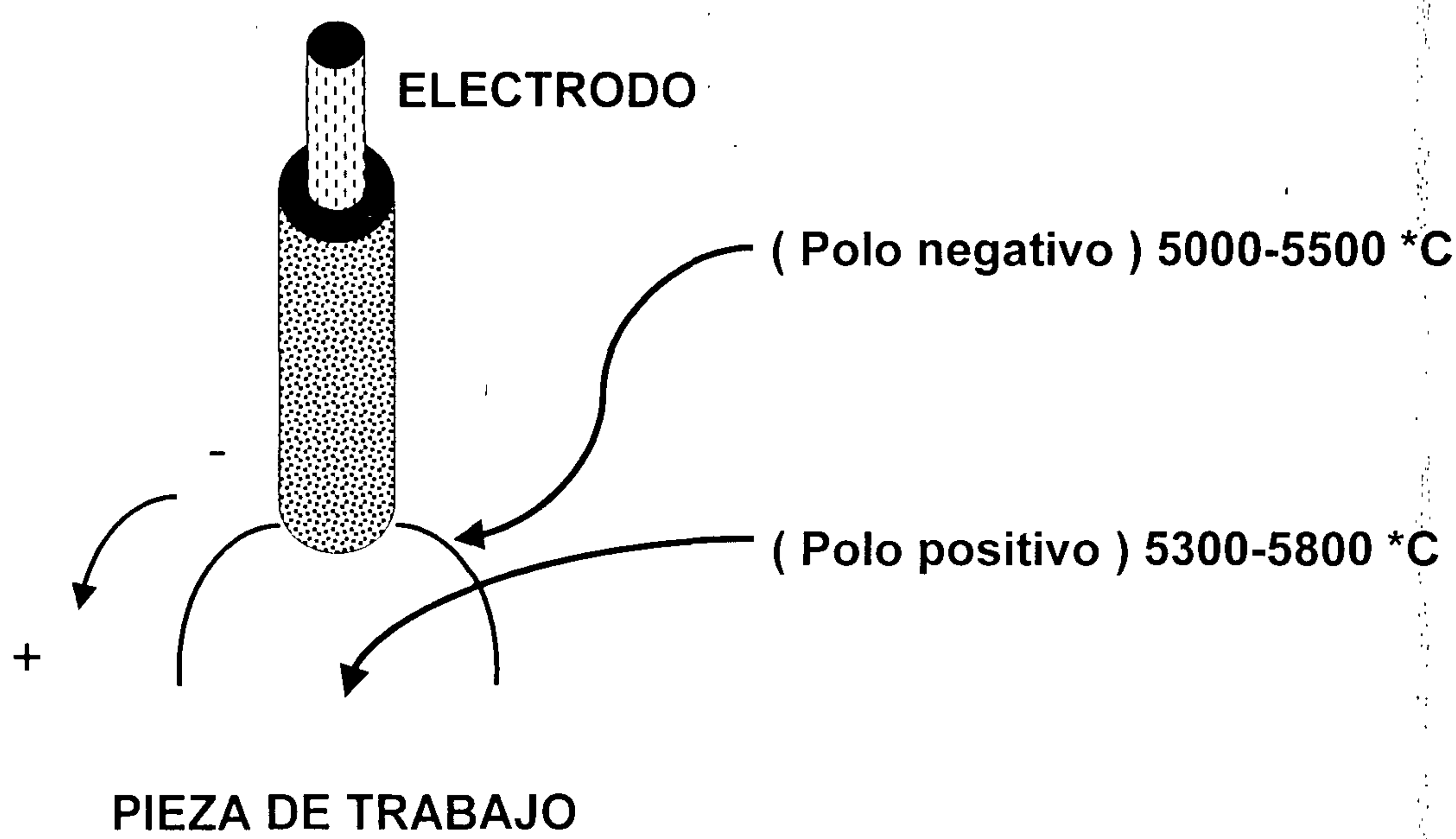


POLARIDAD INVERTIDA
O POSITIVA

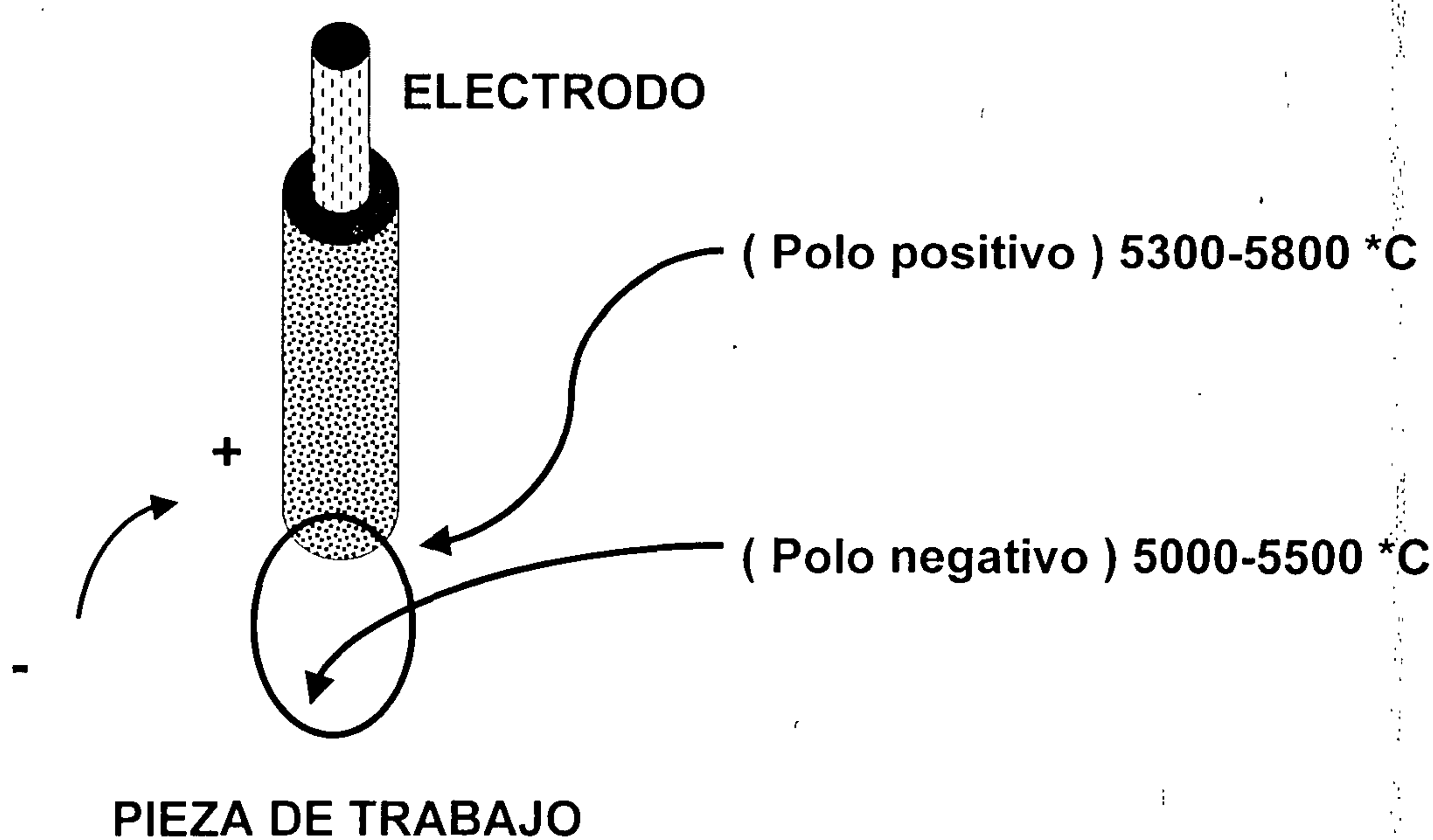
POLARIDAD

AL UTILIZAR LA POLARIDAD INVERTIDA UN TERCIO DEL CALOR GENERADO EN EL ARCO ES LIBERADO EN EL METAL BASE Y DOS TERCIOS SON LIBERADOS EN EL ELECTRODO.

CON DOS TERCIOS DE CALOR LIBERADOS, EL ELECTRODO METALICO Y LOS GASES DE PROTECCION DEL ARCO, SON SOBRECALENTADOS. ESTE SOBRECALENTAMIENTO GENERARA QUE EL METAL FUNDIDO DEL ELECTRODO VIAJE A TRAVEZ DEL ARCO A UNA ALTA VELOCIDAD, LA FUERZA DE ESTA VELOCIDAD GENERARA UNA PENETRACION PROFUNDA DE LA SOLDADURA EN EL METAL BASE.



**POLARIDAD DIRECTA
O NEGATIVA**



**POLARIDAD INVERTIDA
O POSITIVA**

SOLDADURA CON ARCO ELECTRICO

LA SOLDADURAS MAS COMUNES EN EL AREA DE FABRICACION Y MONTAJE DE ESTRUCTURAS METALICAS, SON REALIZADAS A BASE DEL PROCESO CON ARCO ELECTRICO Y ELECTRODO DE APORTE YA SEA DESNUDO O RECUBIERTO. LOS ELECTRODOS (varilla de soldadura) FUERON FABRICADOS INICIALMENTE DEL TIPO DESNUDOS SIMILARES A LOS UTILIZADOS EN SOLDADURA AUTOGENA, MISMOS QUE PRESENTABAN GRAN DIFICULTAD PARA MANTENER EL ARCO, YA QUE LAS ATMOSFERAS ABIERTAS NO PERMITEN LA ESTABILIDAD DEL MISMO DEBIDO A LA PRESENCIA DEL FENOMENO DE OXIDACION, ESTAS DIFICULTADES GENERARON DOS IMPORTANTES PROCESOS BASICOS DE SOLDADURA.

- LOS DE ATMOSFERAS PROTEGIDAS CON GASES INERTES Y ELECTRODO DESNUDO.
- LOS DE ELECTRODOS RECUBIERTOS O CON FUNDENTE INTEGRADO.

SOLDADURA CON ARCO ELECTRICO (cont.2)

EN ESTE CASO EXPONDREMOS EL PROCESO GENERICO DE SOLDADURA DE ARCO CON APORTE A BASE DE ELECTRODOS REVESTIDOS.

LOS ELECTRODOS REVESTIDOS O CONFUNDENTE INTEGRADO PERMITEN AL ARCO MAYOR ESTABILIDAD AL CREARSE UNA ATMOSFERA DE PROTECCION QUE COLABORA A LA EXPULSION DE IMPUREZAS EN EL METAL FUNDIDO, DESARROLLANDO GASES INERTES MISMOS QUE COLABORAN A MANTENER LA SUPERFICIE EXTERIOR DEL METAL FUNDIDO LIBRE DE OXIDACION. EL PRODUCTO DE LA COMBUSTION DEL REVESTIMIENTO, FORMARA UNA SUPERFICIE DE ESCORIA QUE PROTEGE A LA SOLDADURA DE LA INTEPERIZACION A LA VEZ QUE PERMITE SU ENFRIAMIENTO GRADUAL.

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SOLDADURA CON ARCO ELECTRICO (cont. 3)

PROCESO DE SOLDADURA:

- ANTES DE INICIAR LAS ACTIVIDADES DE SOLDADURA SE DEBERA VERIFICAR LA DISPONIBILIDAD DE LAS HERRAMIENTAS Y EQUIPO NECESARIO PARA LA CORRECTA Y SEGURA EJECUCION DE LOS TRABAJOS.

- AL IGUAL QUE EN TODO PROCESO DE SOLDADURA SE DEBERA PROCURAR TENER EN LO POSIBLE LOS ELEMENTOS FIJOS EN BASE A TORNILLERIA O SOPORTERIA PROVISIONAL, Y DE ACUERDO A COMO LO MARQUE EL PROCEDIMIENTO Y EL CODIGO APLICABLE AWS .

- PARA INICIAR EL PROCESO DE SOLDADURA LOS ELEMENTOS DEBERAN ESTAR LIMPIOS LIBRES DE GRASA, OXIDOS Y HUMEDAD, ASI MISMO ES RECOMENDABLE PRECALENTAR LAS PIEZAS A SOLDAR, Y SERA OBLIGATORIA ESTA ACTIVIDAD, SI ASI LO MARCA EL CODIGO DE REFERENCIA.

SOLDADURA CON ARCO ELECTRICO (cont. 4)

- SE COLOCARA LA VARILLA DE SOLDADURA (Electrodo) POR LA SECCION DESNUDA, DENTRO DE LA PINZA PORTAELECTRODO. VERIFICANDO QUE LA MAQUINA SOLDADORA SE ENCUENTRE EN CONDICIONES Y PARAMETROS (Encendida, Amperaje, Voltaje, Conexion a tierra y Conexion de los cables de tierra y portaelectrodo a los bornes de la maquina).

- PARA INICIAR EL ARCO DEBERA TOCAR CON LA PUNTA DEL ELECTRODO EL METAL BASE, LO MAS CERCANO POSIBLE AL PUNTO DEL INICIO DEL TRABAJO A REALIZAR UNA VEZ LOGRADO EL ARCO ESTE DEBERA SER MANTENIDO TENIENDO LA PUNTA DE LA VARILLA ELECTRODO A UNA DISTANCIA APROXIMADAMENTE IGUAL AL DIAMETRO DEL ELECTRODO. EJEMPLO: UN ELECTRODO DE 1/8 DEBERA TENER UNA LONGITUD DE ARCO DE 1/8"

SOLDADURA CON ARCO ELECTRICO (cont. 5)

- SERA IMPORTANTE MANTENER EL ARCO EN LA DISTANCIA RECOMENDADA YA QUE UN ARCO MUY CORTO NO BRINDARA EL CALOR SUFICIENTE PARA OBTENER UNA FUSION HOMOGENEA ENTRE EL METAL BASE Y EL METAL QUE SE DEPOSITA Y UN ARCO MAS LARGO UNICAMENTE FUNDIRA LA PUNTA DEL ELECTRODO GENERANDO GRANDES GOTAS QUE SE DEPOSITARAN EN FORMA IRREGULAR Y CON MALA ADERENCIA AL METAL BASE.

- CORRIENTE APROPIADA. EN SOLDADURA ELECTRICA ES MUY IMPORTANTE EL AMPERAJE Y ESTE ESTARA BASADO AL TIPO DE JUNTA, ESPESOR DEL MATERIAL BASE, POSICION DE LA JUNTA A SOLDAR Y DIAMETRO DEL ELECTRODO.

- PARA EL AJUSTE DEL AMPERAJE SE CONSIDERAN TANTOS AMPERES COMO MILESIMAS DE PULGADA TENGA EL DIAMETRO DEL ELECTRODO. Ejemplo. PARA UN ELECTRODO DE 1/8 DE DIAM. (125 milesimas de pulgada) SE PODRAN UTILIZAR 125 AMPERES.

CORRELACION DE ESPESOR DEL METAL BASE, ELECTRODO Y AMPERAJE A UTILIZAR

ESPESOR DEL METAL	DIAMETRO DEL ELECTRODO	AMPERES PARA SOLDAR	VOLTAJE
1/16 - 1/8	3/32	50 - 90	15 -17
1/8 - 1/4	1/8	90 - 140	17 - 20
1/4 - 3/8	5/32	120 - 180	18 - 21
3/8 - 1/2	3/16	150 - 230	21 - 22
1/2 - 3/4	7/32	190 - 240	22
3/4 - 1	1/4	200 - 300	22

SOLDADURA CON ARCO ELECTRICO (cont. 6)

- CREADO EL ARCO EN LAS CONDICIONES ANTES DESCRITAS SE NOTARA UN CHARCO DE METAL FUNDIDO, MUY SIMILAR AL LOGRADO CON LA SOLDADURA AUTOGENA. ESTE CHARCO DEBERA SER MANTENIDO DURANTE TODO EL PROCESO DE SOLDADURA HASTA LOGRAR CONSUMIR LA TOTALIDAD DE LA VARILLA ELECTRODO.

- PARA LOGRAR UN CORDON DE SOLDADURA UNIFORME EN ANCHO PENETRACION Y ESPESOR SERA NECESARIO MANTENER UNA VELOCIDAD DE AVANCE A LO LARGO DEL CORDON DE SOLDADURA AL MISMO TIEMPO QUE SE MANTIENE UN MOVIMIENTO OSCILATORIO QUE MANTENDRA EL CHARCO EN DIMENCIONES Y TEMPERATUR.

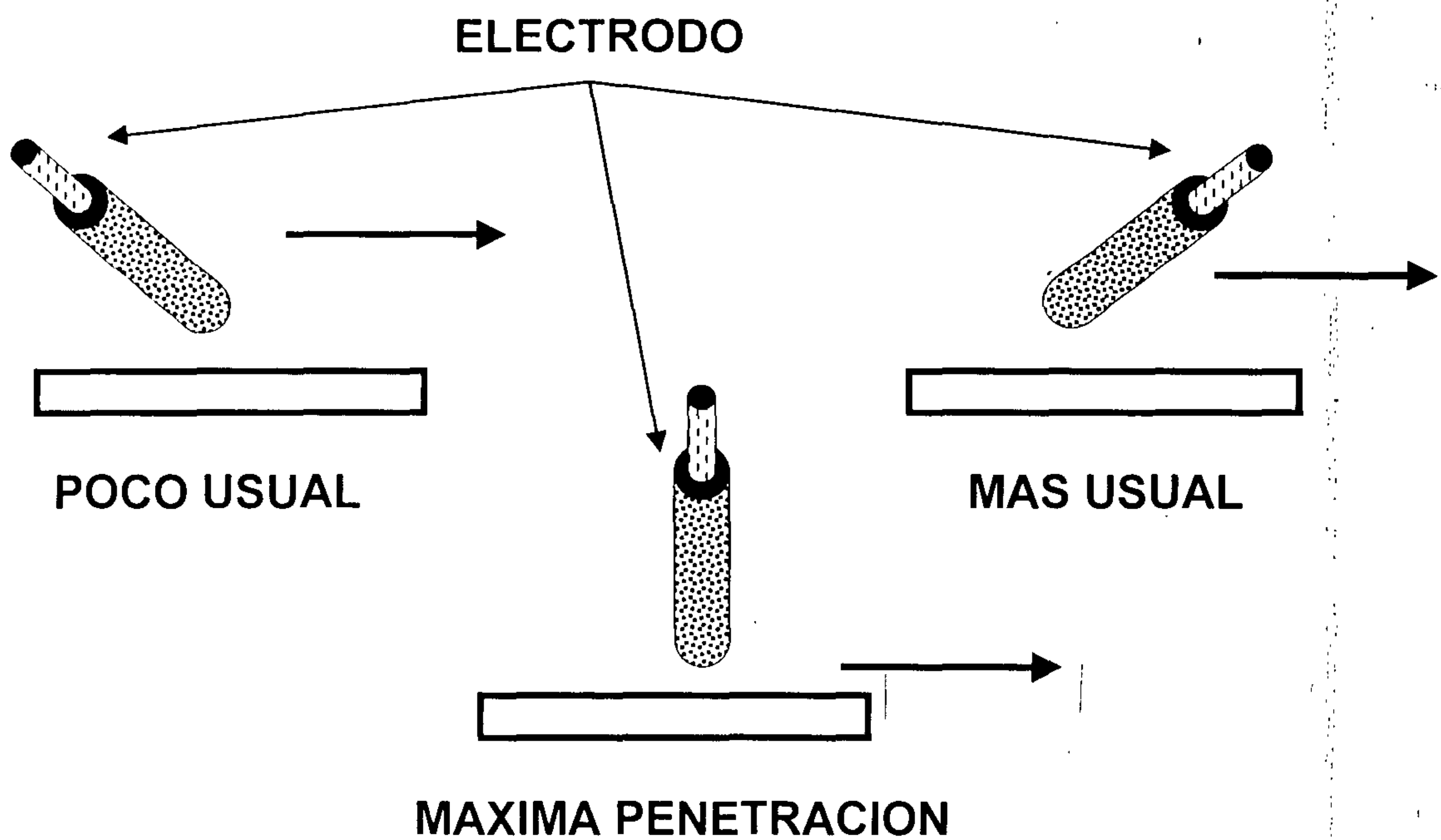
- PARA EL CASO DE SOLDADURAS DE PLACAS DE GRAN ESPESOR SERA NECESARIO LA APLICACION DE VARIOS CORDONES DE SOLDADURA, SIENDO NECESARIO REALIZAR LIMPIEZAS CON CEPILLO DE ALAMBRE O CARDA ENTRE CADA UNO DE LOS PASOS.

SOLDADURA CON ARCO ELECTRICO (cont. 7)

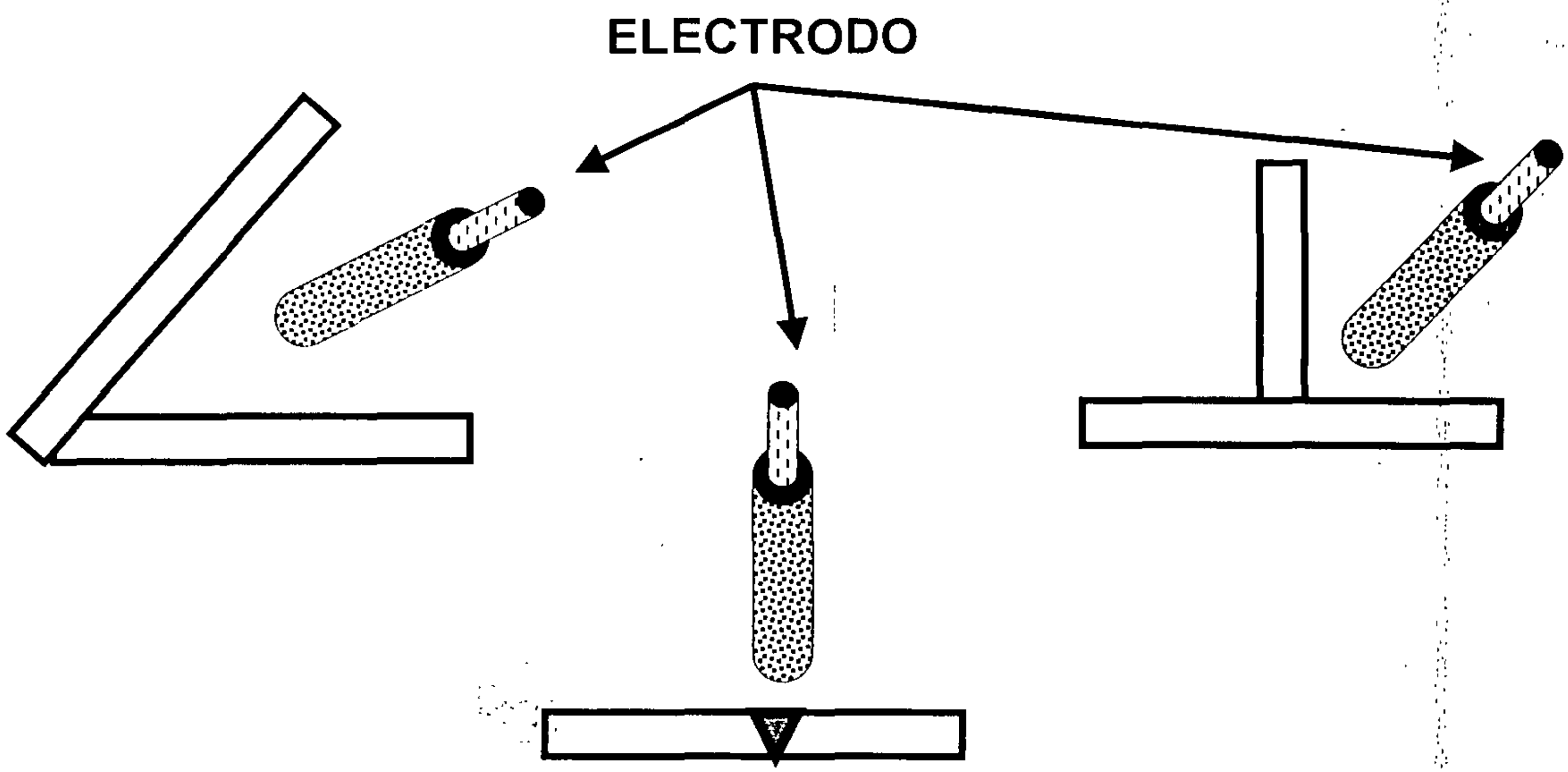
• <u>VELOCIDAD DE AVANCE.</u> NORMALMENTE CONVIENE AVANZAR A LA MISMA VELOCIDAD DE MANERA QUE EL CORDON DE SOLDADURA TENGA EL DOBLE DE ANCHO QUE EL DIAMETRO DEL ELECTRODO. NATURALMENTE, ESTAS CONDICIONES PUEDEN VARIAR SEGUN LAS NECESIDADES O POSICIONES DE LA SOLDADURA.

• <u>ANGULO DEL ELECTRODO.</u> OTRA NORMA MUY IMPORTANTE QUE DEBERA RESPETARSE EN SOLDADURA, PARTICULARMENTE EN SOLDADURA DE ANGULO Y JUNTAS CON BISELES, ES MANTENER EL "ANGULO DEL ELECTRODO" . ESTE ANGULO SE DEBE COLOCAR EN EL BISECTOR CON UNA INCLINACION DE 45 GRADOS CON RELACION A LA VERTICAL; EN UNIONES DE JUNTAS A TOPE, CON O SIN BISEL, EL ELECTRODO SE DEBE MANTENER PERPENDICULAR A LA LINEA DE SOLDADURA FORMANDO UN ANGULO DE 90 GRADOS.

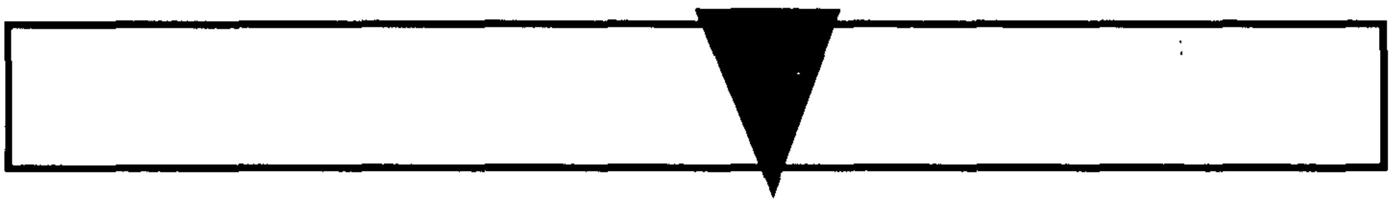
ANGULO DEL ELECTRODO EN EL SENTIDO DEL AVANCE PERPENDICULAR



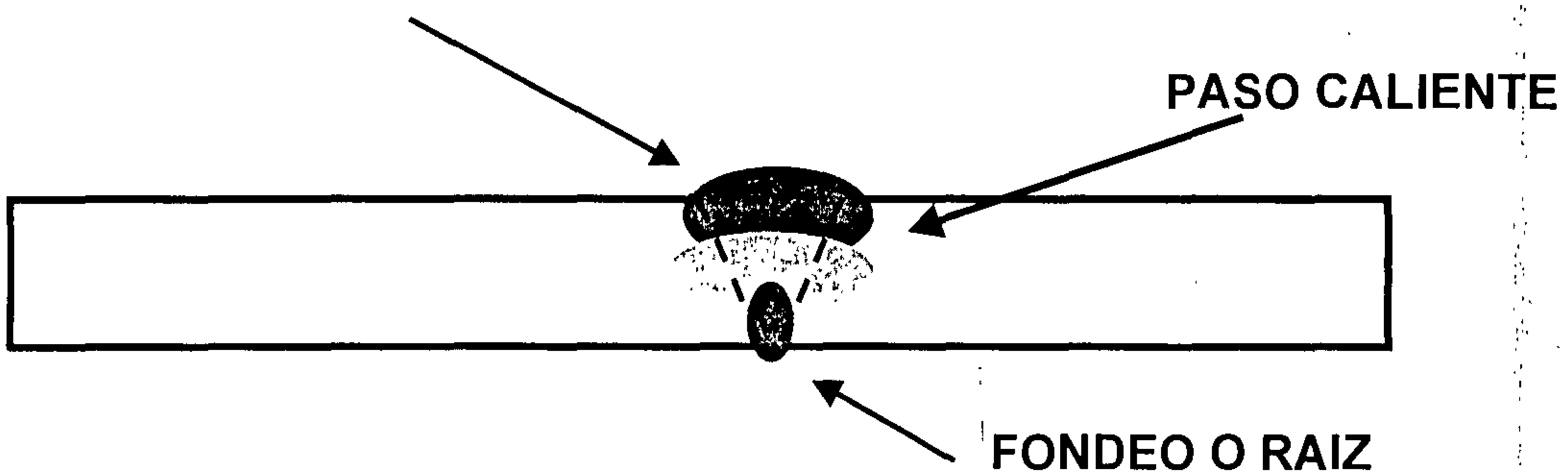
ANGULO DEL ELECTRODO TRANSVERSAL AL CORDON



TRES PASOS COMUNES DE UNA JUNTA SOLDADA



PREPARACION DE LA JUNTA



FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

PUENTES

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PALACIO DE MINERÍA
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CONSTRUCCION DE PUENTES DE ACERO

I.- INTRODUCCION

Los principales tipos de puentes con Estructura de Acero - son los siguientes:

1.- Con Viguetas o Vigas simplemente apoyados.

2.- Vigas Continuas.

3.- Vigas Gerber.

4.- Vigas Presforzadas.

5.- Armaduras

a).- De Paso Inferior

b).- De Paso Superior

c).- De Paso a Traves

6.- Cajones

7.- Estructuras Espaciales

8.- Puentes Colgantes

9.- Puentes Atirantados

1.- Los puentes con Viguetas o Vigas simplemente apoyados consis-- ten en un conjunto de viguetas del tipo comercial producidas en li nea por las laminadoras sobre las cuales se cuela una losa de con- creto armado.

Cuando se usan los perfiles comerciales regularmente se trata de puentes de claros pequeños.

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Para cubrir claros mas grandes se usan viguetas de tres placas soldadas de peralte mayor al de las viguetas comerciales y sobre - las mismas se cuela la losa de concreto armado.

Antiguamente estas viguetas se hacían con remaches en vez de - soldadura y se formaban con canales y placas,o bien con ángulos y placas.

2.- Puentes de Vigas Continuas es una variación de los anteriores, y se utilizan vigas de acero que pasan continuas a través de los - apoyos de la Subestructura con objeto de cubrir claros mayores que los cubiertos con vigas simplemente apoyadas sin aumentar despro-- porcionadamente el peralte.

3.- Vigas Gerber o articuladas constituyen una variedad de los - puentes con vigas continuas, siendo su mayor utilización cuando se tienen claros contiguos de diferentes dimensiones.

Para cubrir el claro mayor se prolongan las viguetas que cu--- bren el claro menor y se liga una vigueta a través de una articula ción para terminar de cubrir el claro mayor.

4.- Vigas Presforzadas; son vigas de acero soldadas a las cuales - se les presfuerza para aprovechar mejor la sección y poder reducir peralte. No es una práctica muy usual, pues con los procedimientos usados hasta la fecha han resultado antieconómicos.

5.- Armaduras; podemos distinguir fundamentalmente tres tipos de - puentes de armaduras en función de la colocación de la superficie de rodamiento con respecto a la armadura.

Cuando la losa o superficie de rodamiento se encuentra en la - parte inferior de la armadura, este puente se llama de Paso Infe-- rior.

Cuando la losa o superficie de rodamiento se encuentra sobre - la armadura este se llama de Paso Superior.

Cuando la superficie de rodamiento se encuentra entre la cuer- da superior e inferior de la armadura, se llama de Paso a Través.

Las armaduras son de diversos tipos como pueden ser entre otras la Warren, Pratt, etc.

6.- Cajones.- Estos puentes se forman con elementos delgados y - - atiesados formando una gran sección cerrada sobre la cual se cuela una losa de concreto, o bien se le coloca una superficie de roda-- miento del tipo asfáltico, en el caso de puentes carreteros, o bien la estructura de la vía en caso de puentes ferrocarrileros.

7.- Estructuras Espaciales.- Son armaduras tridimensionales de - - Paso Superior.

8.- Puentes Atirantados.- Regularmente se usan secciones cajón o - armaduras de paso superior, las cuales son suspendidas por siste-- mas de cables que se anclan a grandes pilas.

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Hay varios modelos de atirantamiento como son los sistemas de ti-- rantes paralelos y a los lados de la calzada, o bien con tirantes al centro de la superficie de rodamiento.

9.- Puentes Colgantes.- En la mayoría de estos casos se usan armaduras que son suspendidas de la catenaria que forman los cables.

II.- TECNICAS CONSTRUCTIVAS.

1.- Remaches y Tornillos.- Los primeros puentes de Acero se hacían utilizando remaches y tornillos para lograr la unión entre -- los diferentes elementos, en la actualidad los remaches practicamente han dejado de usarse y los tornillos han tenido usos muy específicos y restringidos.

2.- La soldadura ha logrado tener una importancia fundamental en la construcción moderna, pues aún en el acero de refuerzo de los puentes de concreto se utiliza para hacer uniones a tope de vari-- llas de diametro grande (#8 a #12) y cuando los traslapes de varillas menores son muy numerosos y bloquean la separación de las - mismas, se pueden usar soldaduras de filete para reducir la longitud de traslape.

En practicamente todos los tipos de puentes de acero que - se construyen en la actualidad todas las juntas o uniones son realizadas con soldadura.

Al realizar la soldadura para elementos de puentes debe considerarse el uso de soldadura de comportamiento dúctil, ya que

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puede estar sujeta a esfuerzos alternados de tensión y compresión. o por lo menos está sujeta a variaciones de magnitud de los esfuerzos a los que trabaja. Lo anterior es ocasionado por las cargas -- móviles.

Para soldaduras de campo que se deberán hacer con arco--eléctrico-manual (salvo que la importancia de la obra implique -- instalaciones especiales) deberá usarse electrodo de la serie - - AA18, teniendo especial cuidado de contar con hornos para almace--nar la soldadura una vez abiertos los depósitos de fábrica, ya que estas soldaduras son altamente higroscópicas y en muchas ocasiones la presencia de humedad en los puentes es grande y dañan la soldadura.

Es conveniente señalar que las máquinas para soldar conocidas como rectificadoras de corriente (máquinas eléctricas) son en términos generales mejores que las de generador, por lo cual es recomendable que si se requieren dos o más soldadoras en campo, se haga un equipo con un generador que alimente a las máquinas eléc--tricas.

3.- Se han descrito 9 tipos de puentes de estructura de acero. Ahora veremos el procedimiento general de construcción de cada uno de ellos, haciendo las acotaciones más importantes de su proceso.

<u>Puente con Viguetas o Vigas simplemente apoyadas.</u>

Este tipo de puente representa el procedimiento más sencillo y rápido de construir un puente, pues no requiere de una fabricación

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de los elementos metálicos, sino que se adquieren con los distri-- buidores o fabricantes de perfiles laminados; tratándose de elemen tos de producción en Línea; o bien se fabrican las vigas que seña- lan los planos del proyecto respectivo observando las especifica-- ciones de soldadura, calibre y calidad de las placas o elementos - metálicos. Cuando se trata de Vigas de más de 12 metros de longi-- tud debe preverse la forma de transporte con el fin de no compli- car el mismo. sin embargo debe considerarse que el transportar los tramos lo más grande posible representa economía y rapidez en la - ejecución de los trabajos, ya que puede evitar el realizar alguna unión de campo que implica costo, tiempo y una supervisión mas - - cuidadosa que la de taller ya que los trabajos de soldadura in - - situ no cuentan (regularmente) con todos los elementos que se -- tienen en un taller.

Hay pues necesidad de planear el trabajo desde un principio pa ra equilibrar en tiempo, costo y seguridad los trabajos de fabrica ción, transporte y ensamble en campo.

Precaución fundamental al cortar las vigas en taller es seña-- larlas con objeto de evitar confuciones en obra.

El montaje es de lo más sencillo y deberá hacerse con grúa si se encuentra una disponible cercana a la obra,en caso contrario - deberá hacerse utilizando plumas, polipastos y malacates manuales para cable de acero (tirfords).

Una vez realizado el montaje deberá colocarse la cimbra soste- nida en las mismas trabes o vigas de acero evitando totalmente la obra falsa. Regularmente el proyecto debe prever la condición de trabajo de cimbrado y colado sin obra falsa pero deberá asegurarse de esta situación, ya que las fallas del patín de compresión puede presentarse si no se tomaron las dimensiones necesarias durante el proyecto. En la generalidad de los casos es más económico, rápido y seguro hacer de las vigas elementos autoportantes para las condiciones de colado.

Regularmente la superficie de rodamiento, o bien la superficie para colocar babito, durmientes y rieles es de concreto pero en el caso de puentes carreteros puede presentarse la alternativa de pisos de madera y en algunas ocasiones de placa ó rejillas.

Puentes con Vigas Continuas.

En terminos generales la fabricación y transporte obedece a los mismos principios que los descritos en el caso de puentes con vigas simplemente apoyadas, sin embargo cuando se habla de puentes con vigas continuas se está tratando de un puente de varios claros y deberá preverse el orden en que se transporten y ensamblen los tramos del puente.

Un aspecto de importancia extrema es el orden en que deben ejecutarse los trabajos de colado y que deben estar claramente señalados en los planos de proyecto estructural y de taller, ya que en ocasiones puede presentarse inclusive la posibilidad de dar la continuidad después de todo o parte del colado.

Tratándose de puentes de varios claros debe suponerse que en muchos casos el acceso a las pilas o apoyos intermedios es difícil o costoso establecer la infraestructura para lograrlo, por lo que deberá preverse el lanzar los elementos utilizando para ello; en todo lo posible; los ya colocados.

Puentes con Articulaciones (Vigas Gerber).

De hecho estos casos nos presentan un caso particular de puentes con vigas continuas. Ya que se trata de vigas con uno o los dos extremos en voladizo donde se colocará una articulación (apoyo móvil que permite giro y desplazamiento) para recibir una viga que tendrá como apoyos las articulaciones mencionadas.

Deberá tenerse cuidado también en el orden de colado y necesariamente en el orden de montaje, ya que los tramos " suspendidos " tendrán que montarse posteriormente a los tramos con parte en voladizo.

Un recurso muy utilizado para el montaje de los tramos " suspendidos " es el de colocarlos abajo de su lugar definitivo e izarlos con plumas, poleas y malacates. En ríos muy caudalosos se usa un chalan para llevar al sitio de izaje las trabes correspondientes.

Vigas Presforzadas.

Se han utilizado con objeto de reducir en los puentes los esfuerzos producidos por la carga permanente y a través de un postensado se liberan para aumentar la capacidad de la sección para recibir los esfuerzos de la carga móvil. El postensado se realiza con cable y gatos iguales a los usados para concreto, variando unicamente los dispositivos de anclaje.

Armaduras

En la actualidad ha disminuido considerablemente el uso de armaduras para puentes, sin embargo, es una de las estructuras de -- acero más versátiles y con grandes ventajas sobre otros sistemas.

Debido a que las armaduras tienen un gran número de elementos, debe tenerse mucho cuidado desde la fabricación en taller pues de no hacerlo, se tendrán grandes problemas en la obra.

El proceso debe iniciarse con la ingeniería de detalle que nos produce finalmente los planos de taller que vienen siendo planos - constructivos donde se indica con toda exactitud y magnificando los detalles las secciones y ensambles de los mismos que proyectó el - calculista.

Posteriormente deben fabricarse; midiendo cuidadosamente; las piezas que constituyen la armadura.

Para efecto del montaje debe considerarse toda la maniobra pues hay varias alternativas para realizar la colocación de la armadura en su sitio final. Las principales formas para realizar el montaje son las siguientes:

Armado en Sitio.- A través de plumas y cables se van colocando cada uno de los elementos que conforman la armadura; una vez colocados dos o más elementos que integran un nudo, éste se sujeta al sistema de plumas a través de cables para que el nudo en cuestión ocupe el lugar en el espacio que será el definitivo y así en forma consecutiva se van colocando uno por uno de los elementos hasta --

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formar la armadura proyectada.

Lanzado.- Consiste en construir la armadura o conjunto de armaduras que formarán la estructura del puente antes de colocarla en su lugar. Esta construcción puede realizarse en la obra en la vecindad de uno de los apoyos, o bien parcialmente en taller y terminarla en la obra.

Una vez formada la armadura ésta se lanza en Cantiliver y aprovechando cables guía y plumas con cables y malacate, o bien grúas colocadas en el otro extremo del claro a cubrir se prosigue el lanzamiento hasta que la(s) armadura(s) ha(n) quedado en su sitio.

Esta opción implica el cálculo de las fuerzas en cada una de las barras de la armadura durante los diferentes pasos del proceso de montaje, ya que pueden presentarse condiciones críticas que hagan fallar la estructura.

Empujado.- Es una variante del proceso de lanzado con la diferencia que permite lanzar en Cantiliver toda la armadura. Regularmente se usan armaduras auxiliares provisionales que bien pueden ser delanteras (nariz) o bien posteriores que sirven en este último caso como anclas.

Izado.- Cuando se tienen grandes ríos un procedimiento recomendable es la utilización de un chalan sobre el cual se montan las armaduras y se colocan abajo de sus apoyos procediendo posteriormente a izarlas hasta colocarlas en su sitio.

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Para las maniobras de Izaje se pueden usar cables en polipas-- tos y plumas o bien mecanismos con gatos hidráulicos semejantes a los usados para los cables de presfuerzo.

Cajones.- Este tipo de Estructuras ha sido cada vez mas usado en virtud de las técnicas para soldar tan confiables con las que - contamos en la actualidad.

La fabricación de los cajones debe hacerse en taller; ya sea - en un taller colocado a pie de obra o bien en un taller remoto donde después de fabricar el cajón se desarme en las piezas adecuadas para poder transportarse al lugar de la obra donde se volverá a ensamblar.

Para el montaje los métodos usados son el de Lanzamiento, Empujado o el de Izado que se describieron para las armaduras.

Regularmente este tipo de Cajones lleva un sistema de piso de acero, recubierto posteriormente por una carpeta asfáltica para -- dar la superficie de rodamiento.

Estructuras Espaciales.- Este tipo de Estructuras para puentes ha sido desarrollado primero y fundamentalmente en México, aún cuando ya se trabaja en proyectos de este tipo en Japón, Francia, - - Estados Unidos e Inglaterra.

La estructura es una armadura Tridimensional de acero - con nudos soldados y una superficie de rodamiento regularmente de concreto en puentes definitivos y de madera ó rejilla en puentes -

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provisionales. En los puentes hechos en México la losa de concreto es un elemento estructural resistente como parte de la armadura.

Los elementos básicos que conforman la estructura tridimensional son pirámides de base rectángular.

El montaje regularmente se realiza con el método de lanzado, aún cuando puede hacerse con ventajas en el caso de grandes ríos con el de Izaje.

Puentes Atirantados.- Este tipo de puentes implica el cubrir grandes claros pues de lo contrario no resultaría económico. Es por lo tanto una obra de gran costo.

Regularmente se usan secciones cajón de acero con piso o superficie de rodamiento de acero también y recubierta con una carpeta asfáltica.

La forma del Cajón debe tener formas aerodinámica para amortiguar la acción del viento que es una solicitación importante que debe considerarse en el diseño.

Generalmente la magnitud de la obra implica la instalación de un Taller lo más cercano posible al sitio de la obra.

Las secciones cajón serán montadas en dovelas que una vez colocadas en su sitio se ligarán con soldadura y se les colocarán sus tirantes para sujetarlas en la forma más próxima a su sitio definitivo. La geometría del puente se va corrigiendo con retensado de los tirantes.

Los métodos para ir colocando las dovelas son el de Empujado o bien el de Izaje que tratándose de puentes sobre ríos es el más usual, siendo el de Empujado el más conveniente cuando se tra-

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ta de cubrir grandes barrancas con difícil acceso al fondo de las mismas.

Puentes Colgantes.- Fueron los precursores de los puentes atirantados pero aparentemente son más económicos los atirantados por lo que han caído en desuso los puentes colgantes.

Las estructuras más usuales en los puentes colgantes - son las armaduras que quedan suspendidas de los cables principales que son colocados entre apoyos formando (los cables) una Catenaria.

El procedimiento constructivo consiste en colocar los cables y posteriormente ir colocando las armaduras construídas en secciones (dovelas) y ligándose posteriormente entre ellas.

III.- CONSERVACION DE PUENTES DE ACERO.

La historia nos ha enseñado que dándole la debida conservación a un puente de Acero, éste se vuelve practicamente eterno ya que - si la conservación preventiva ha fallado es suceptible de ser corregida con reconstrucciones que implican un costo mínimo con relación al costo actualizado del puente en cuestión.

Los principales aspectos que debe atender la conservación son:

1.- Pintura o Protección Anti-Oxidante y Anticorrosiva.

2.- Conservación o Cambio de Apoyos.

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3.- Restitución de remaches o soldaduras falladas.

4.- Observación de las cargas soportadas para reforzar la es-- tructura oportunamente en caso de requerirlo.

5.- Mantenimiento y cambio si lo amerita de cables, contraven- teos, o piezas dañadas por la acción del tiempo o fuerzas físicas exteriores.

6.- En el caso de puentes atirantados y colgantes el retensado de los Tirantes o Cables Principales.

La conservación de los puentes de acero es fácil y económica - por lo que haciendose en forma adecuada y oportuna alarga la vida útil del puente.

IV.- MODERNIZACION DE PUENTES.- Esta actividad esta resultando una acción tan o más importante que la construcción de un puente nuevo.

En nuestro país se tienen recursos muy limitados debido a la - crisis en la que vivimos y sin embargo se han incrementado las car- gas que circulan por la red carretera de nuestro país, y el incre-- mento de las cargas ha sido en cantidad y magnitud.

Existen en nuestro país un gran número de puentes angostos y/o puentes diseñados para cargas mucho menores que las soportadas por ellos en la actualidad.

Se han desarrollado proyectos y equipos que han permitido hacer de los puentes antiguos puentes modernos; esto es; de antiguos puen

... #15

tes angostos se están haciendo puentes amplios y capacitados para - soportar las cargas actuales.

Los casos más frecuentes son:

1.- Puentes sobre Viguetas.- El procedimiento de reconstrucción ha consistido en reforzar con placas adosadas a las viguetas existentes, esto con el objeto de aumentar su capacidad de carga y si se requiere aumentar el ancho de la superficie de rodamiento, se agregan viguetas nuevas lateralmente a las existentes.

2.- Armaduras de Paso Inferiores.- Se han reforzado todos los elementos de la armadura que de acuerdo con el cálculo lo han requerido. Este refuerzo consiste en agregar área a - través de placas a los elementos que trabajan a tensión y para los elementos cuyo trabajo crítico es el de compresión. se han agregado placas ó perfiles que permiten modificar - el área y el momento de inercia de la sección.

Una vez reforzadas las armaduras se procede a cortar el puente a todo lo largo y se deslizan las armaduras lateralmente para dar el nuevo ancho complementando posteriormente las piezas de puente y reforzándolas para que -- sean capaces de resistir las nuevas cargas con su nueva -- longitud.

Debido a que las desviaciones son muy costosas se - han diseñado mecanismos que permiten CORTAR EL PUENTE A TODO LO LARGO SIN INTERRUMPIR EL TRANSITO.

El mecanismo más sencillo para lograr lo anterior, consiste en la colocación de armaduras provisionales que su̲jetan a las piezas de puente mientras se deslizan lateral-- mente las armaduras definitivas.

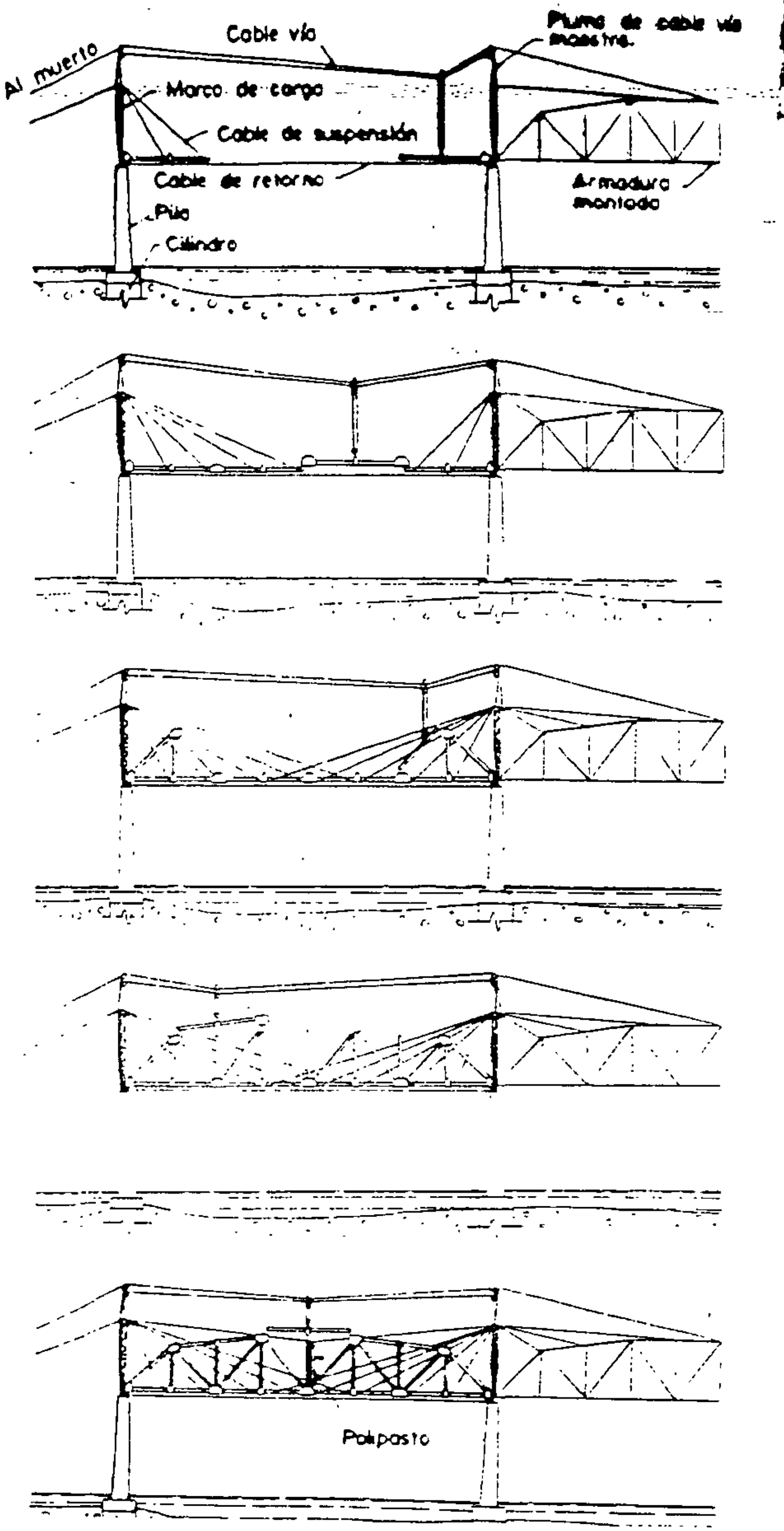
Por último se cuela la franja de concreto que se de̲molió mas la franja de concreto que hay que aumentar debido al nuevo ancho del puente.

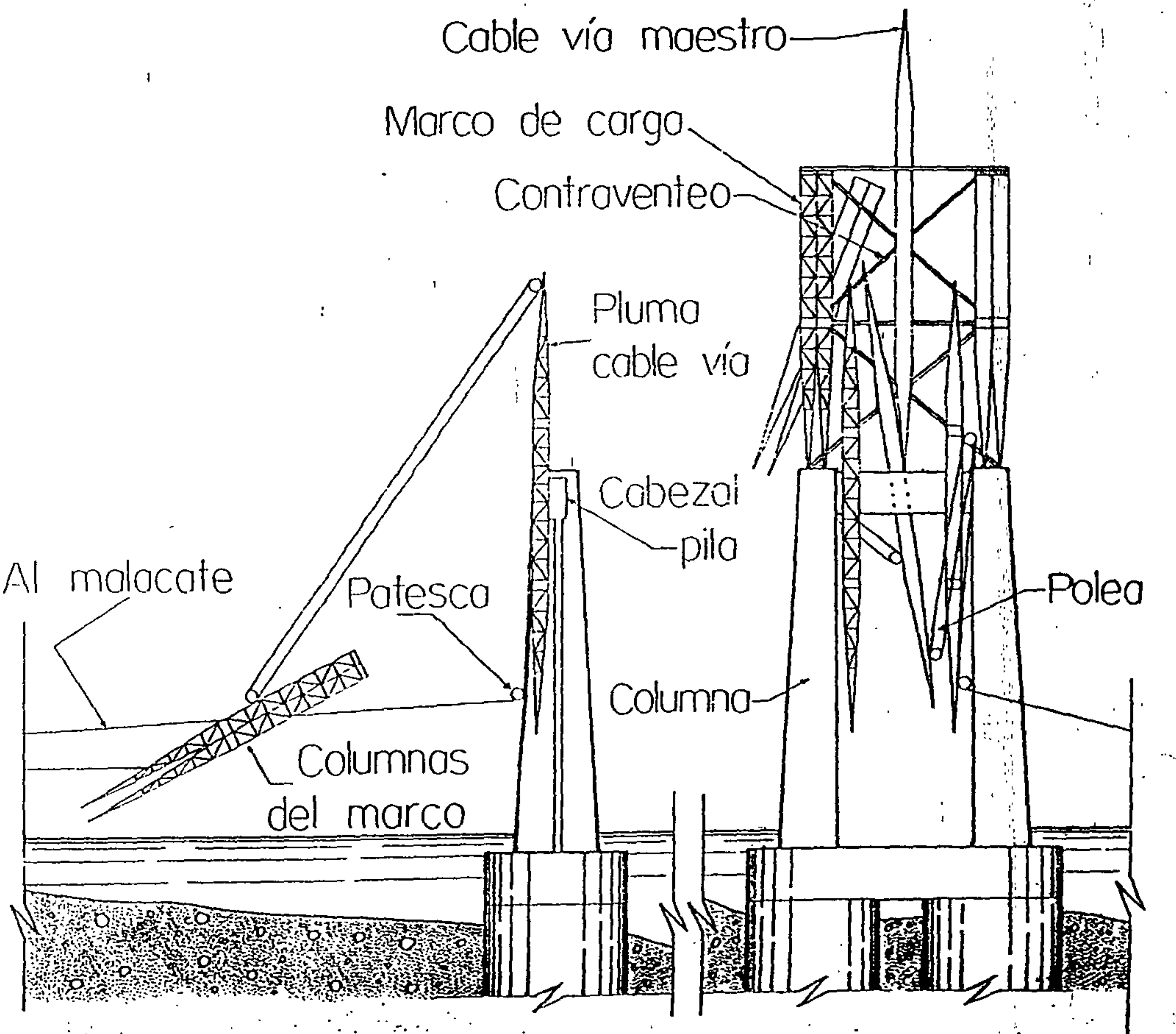
3.- Armaduras de Paso Superior.- Para modernizar este tipo de puentes, se han reforzado las armaduras existentes, y se han agregado nuevas armaduras inclinadas que coincidan en su - cuerda inferior con las armaduras existentes,estas nuevas - armaduras nos permiten ensanchar la calzada sin que los ele̲mentos soportantes de esta (Piezas de Puente y Largueros) estén trabajando en Cantiliver.

Una vez hecho el trabajo de reforzar las armaduras existentes y colocar las nuevas armaduras inclinadas, se -- agregan franjas laterales de losa de concreto armado para - dar un ancho nuevo a la superficie de rodamiento.

VIII MONTAJE DE ARMADURAS

PROCESO DE MONTAJE CON CABLE VIA

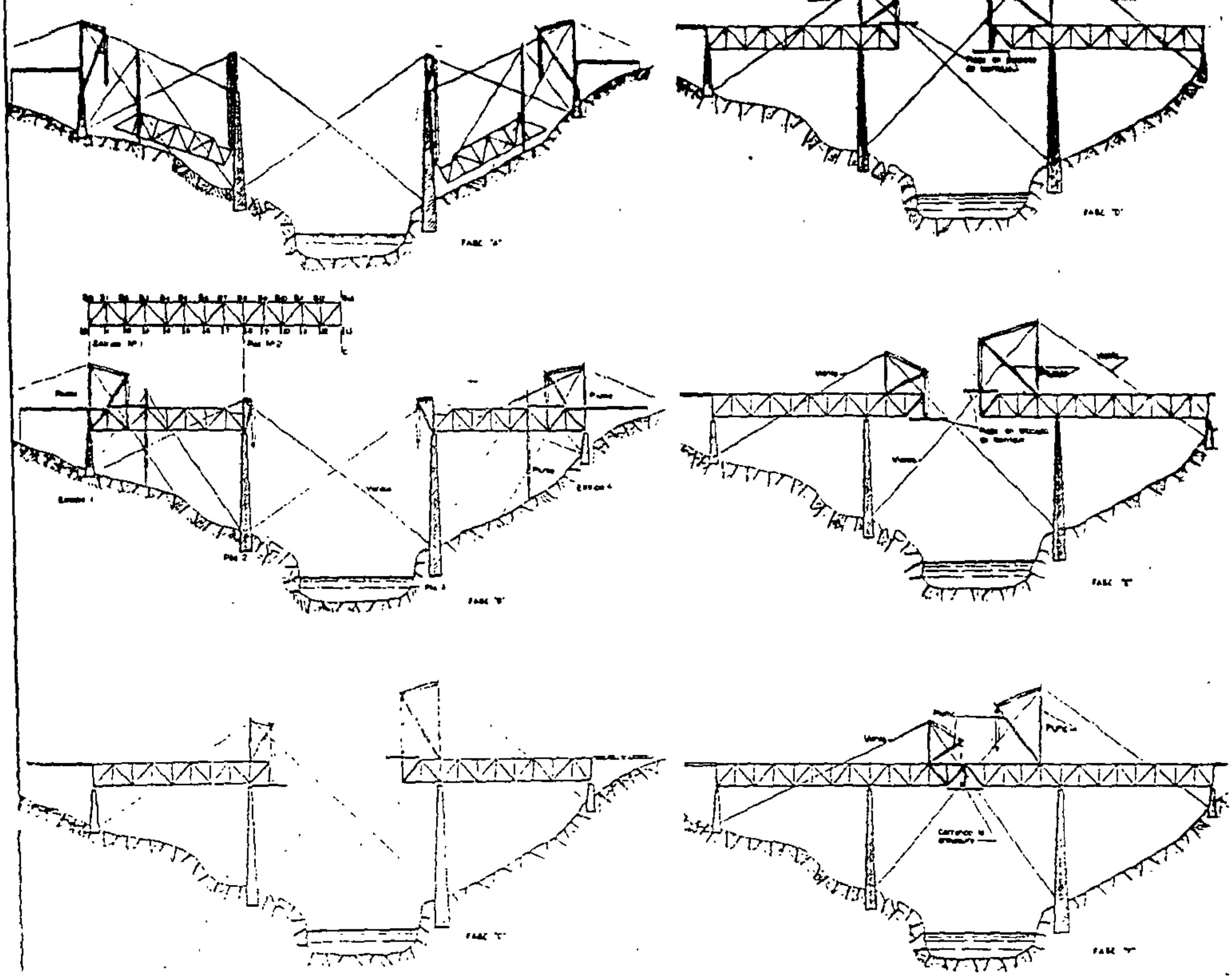


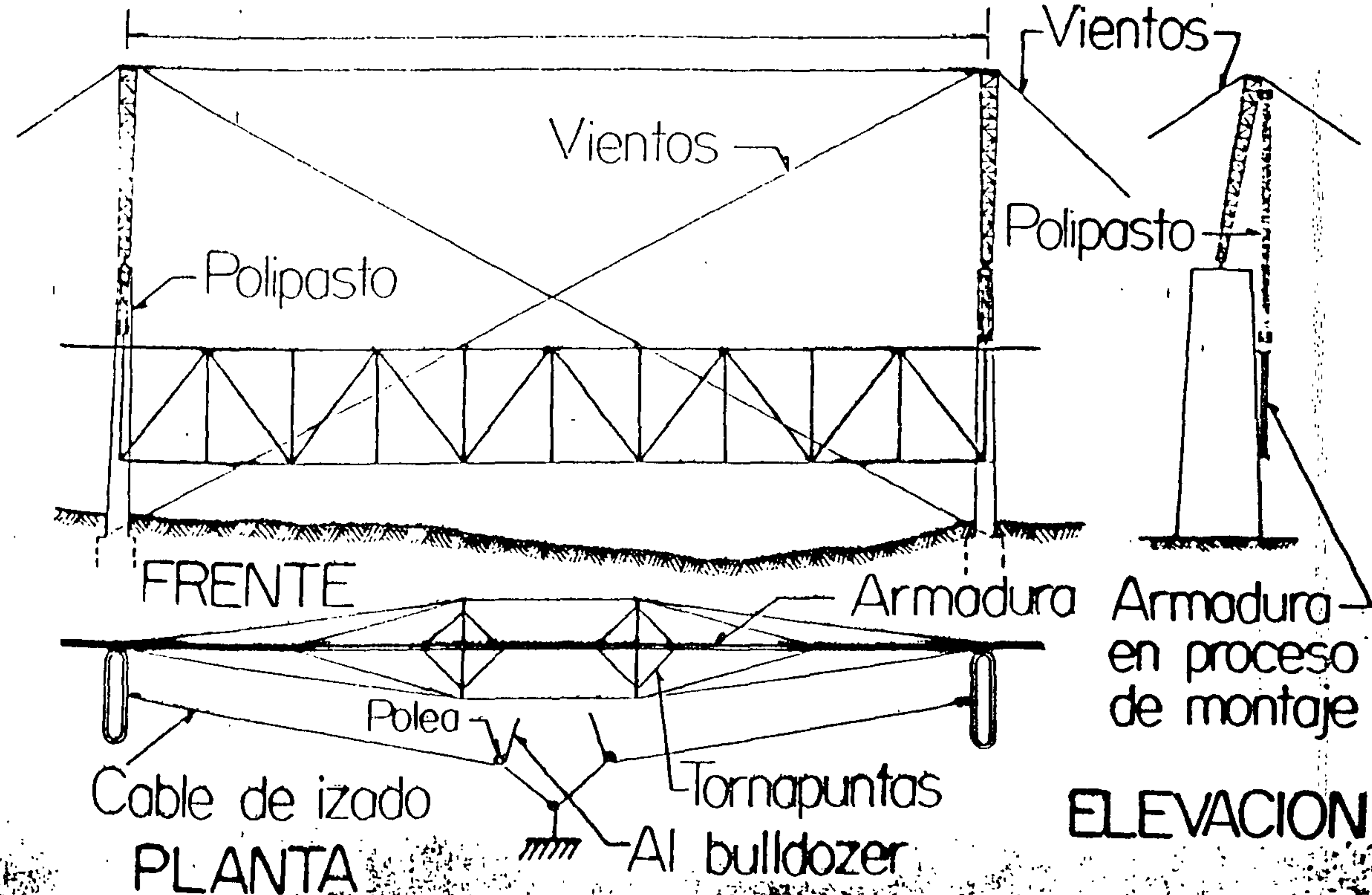


3-VIII

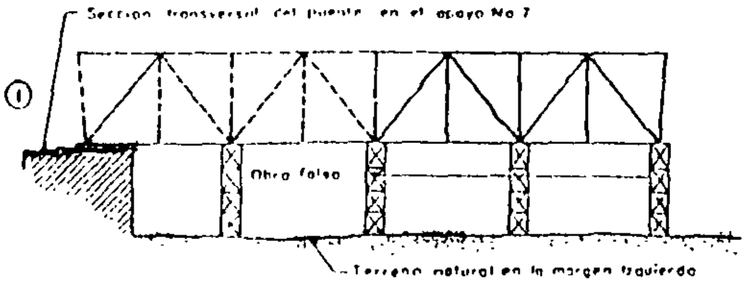
MONTAJE SISTEMA CABLES VIAS

PROCEDIMIENTO DE MONTAJE DE LOS TRAMOS LATERALES Y CENTRAL DE LA SUPERESTRUCTURA DEL PUENTE CHINIPAS

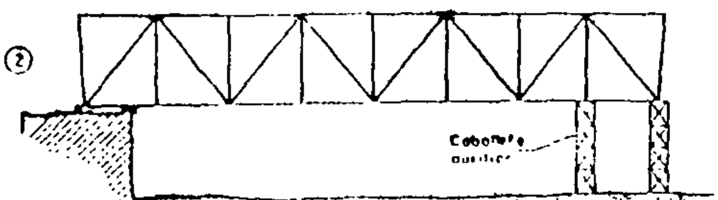




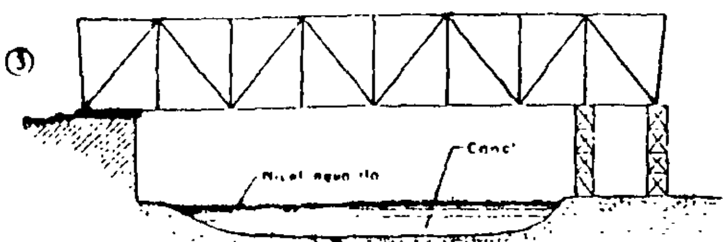
MONTAJE ARMADURA CENTRAL



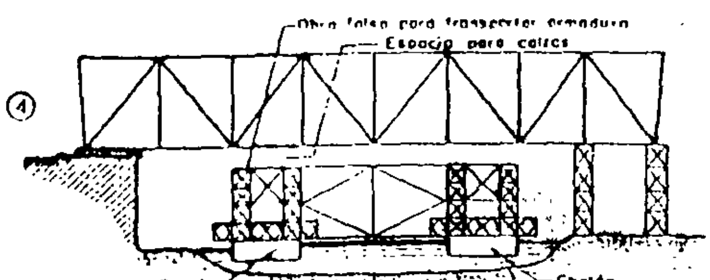
ARMADURA EN PROCESO DE ARMADO SOBRE CABALLETES PROVISIONALES



ARMADURA REACCIONANDO EN SUS EXTREMOS SOBRE APOYOS PROVISIONALES



CANAL CON SALIDA AL RIO CONSTRUIDO EN LA MARGEN BAJO LA ESTRUCTURA



INTRODUCIENDO EN MAREA BAJA UNA OBRA FALSA APOYADA SOBRE CHALANES PARA RECIBIR LA ARMADURA Y TRANSPORTARLA A SU LUGAR DEFINITIVO EN EL PUENTE

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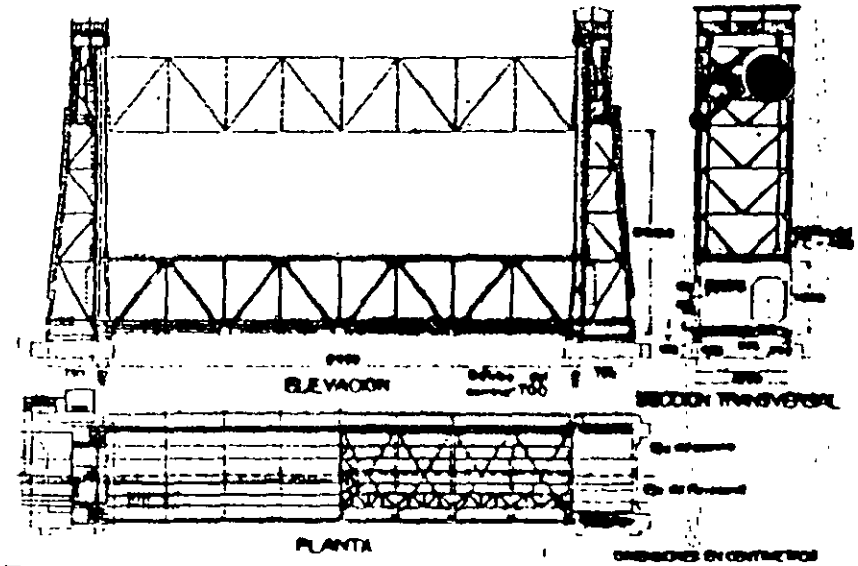

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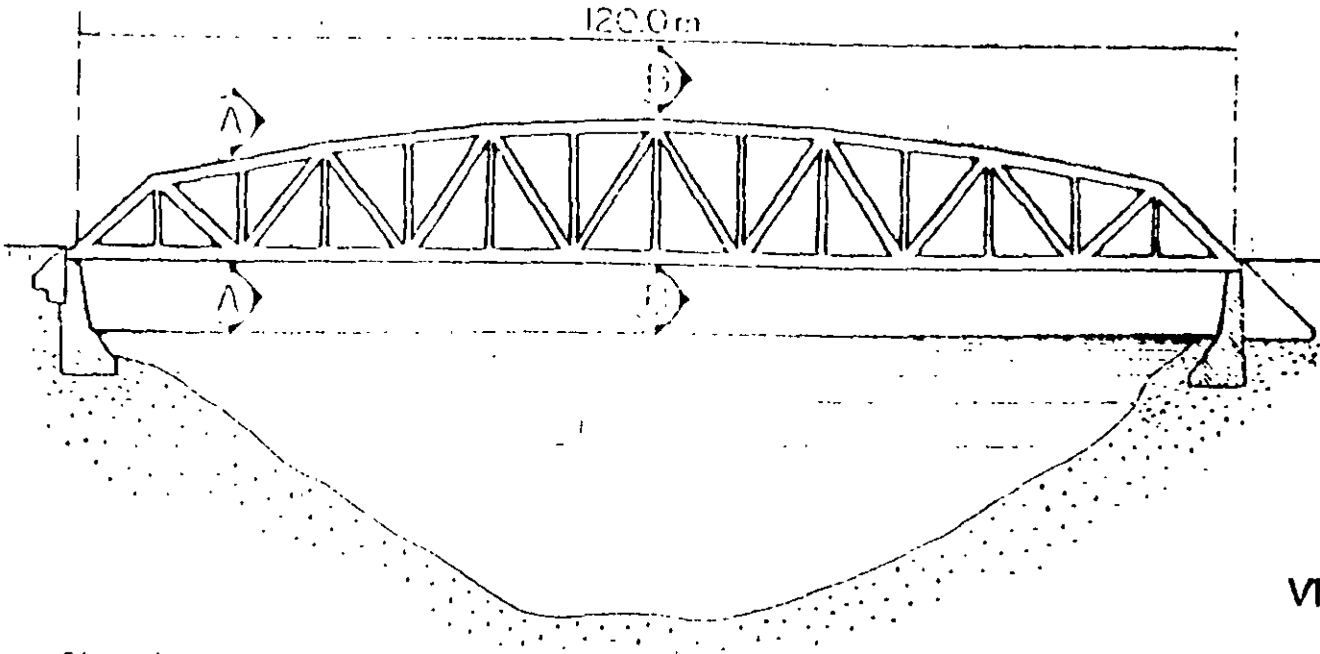
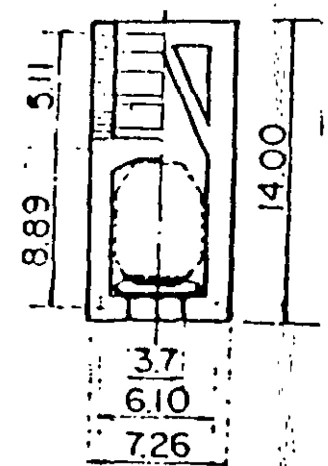
VISTA DE CONJUNTO DEL TRAMO LEVADIZO DEL PUENTE COATZACOALCOS

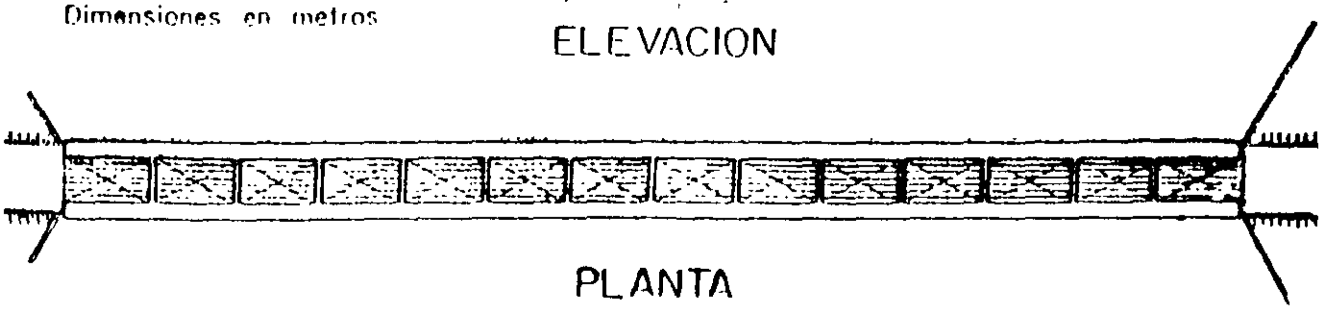




VISTA A-A CORTE B-B

Dimensiones en metros

ELEVACION



PLANTA

PUENTE SAN SALVADOR
FERROCARRIL CORONDIRO L. CARDENAS

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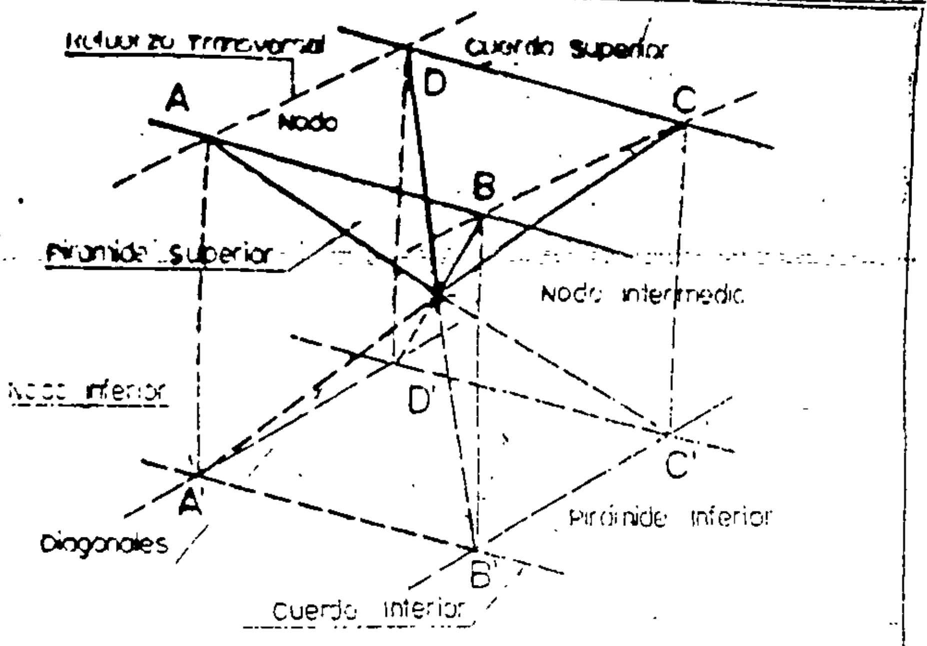
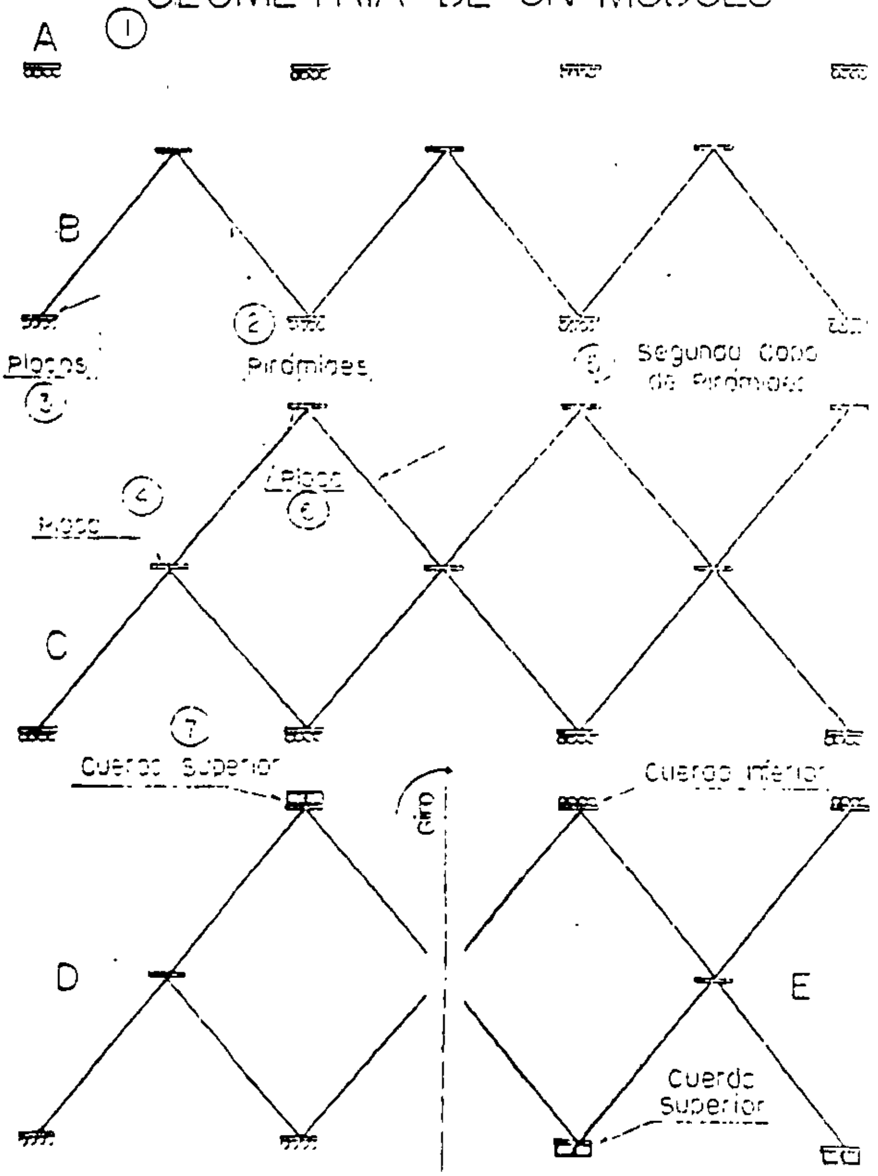


GEOMETRIA DE UN MODULO



MONTAJE PIRAMIDES

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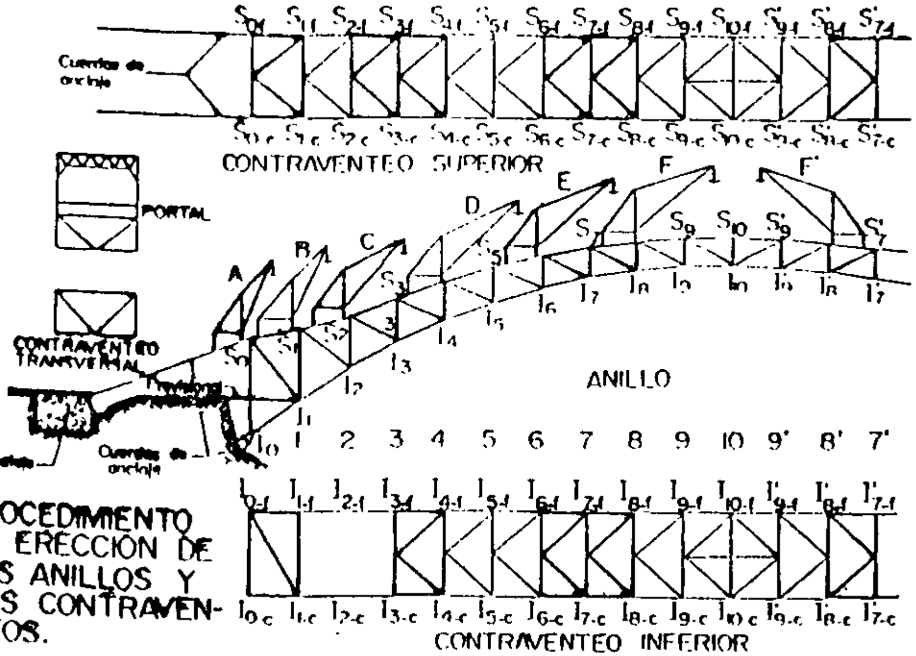
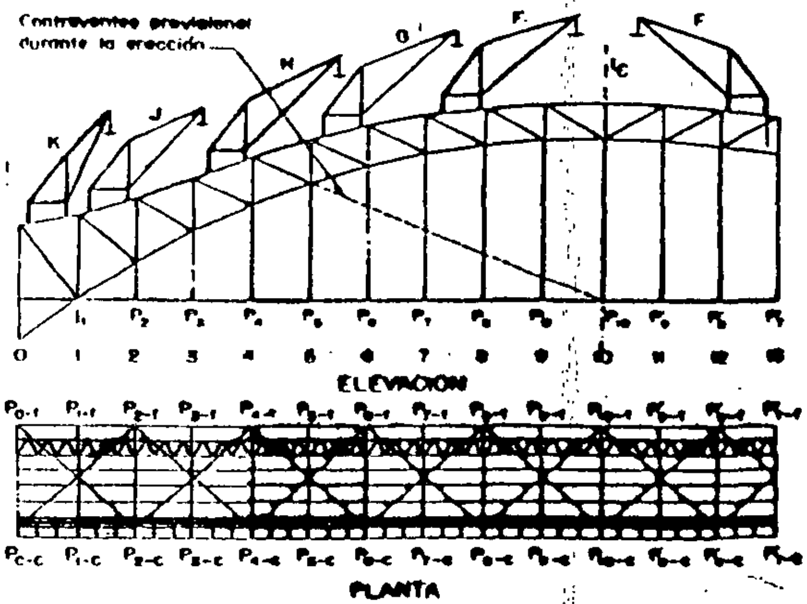
X ARCOS METALICOS



PROCEDIMIENTO DE ERECCION DE LOS ANILLOS Y SUS CONTRAVENTEOS.

X-1

PUENTE USUMACINTA DEL FERROCARRIL DEL SURESTE.



PROCEDIMIENTO DE ERECCION DE LAS PENDOLAS Y DEL SISTEMA DE PISO

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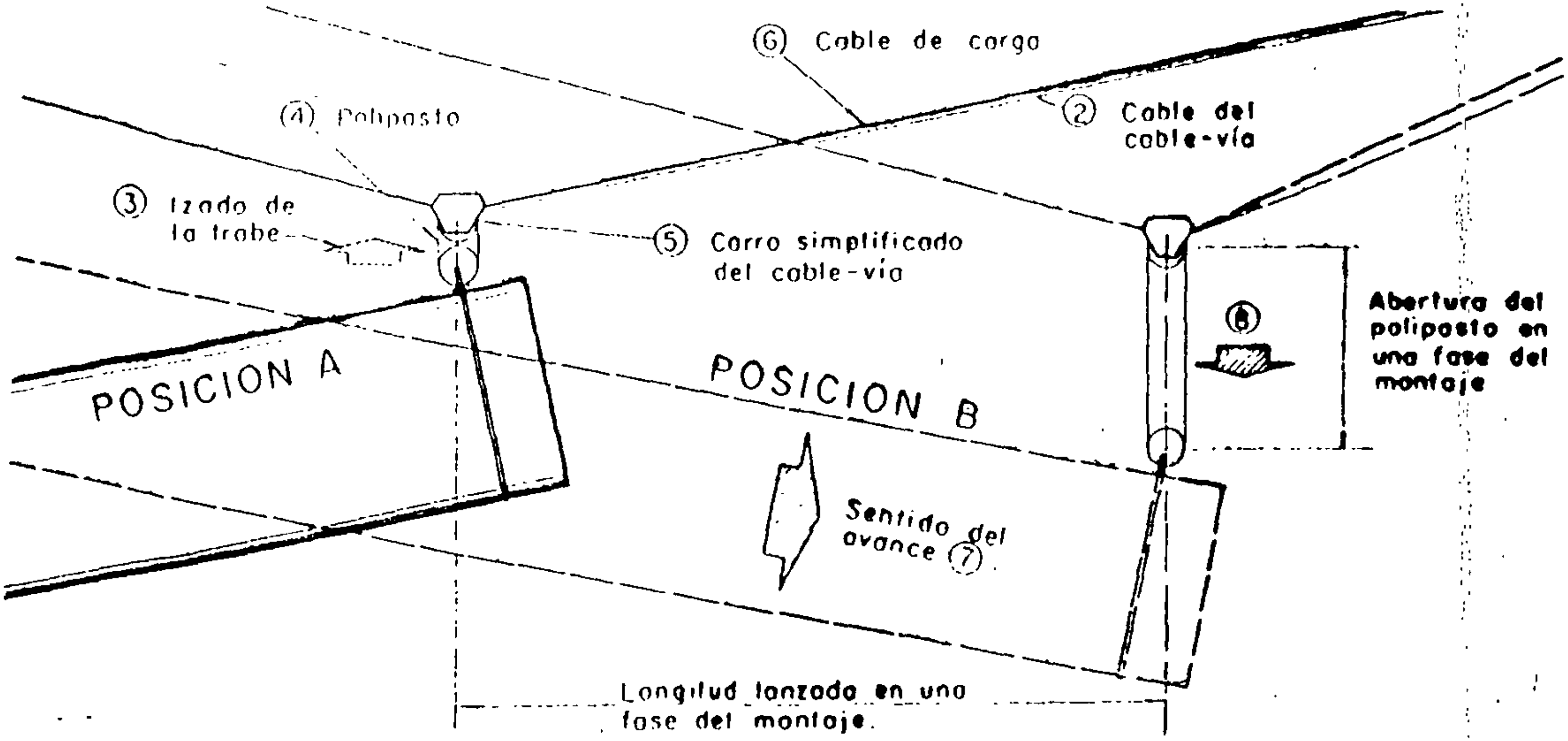
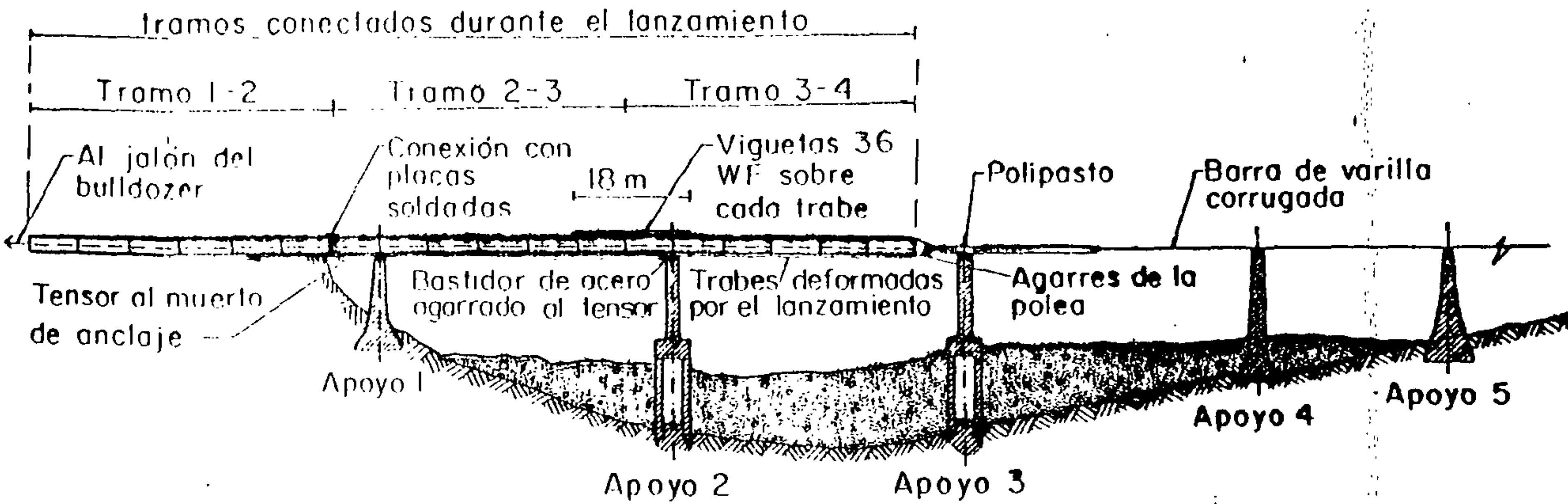
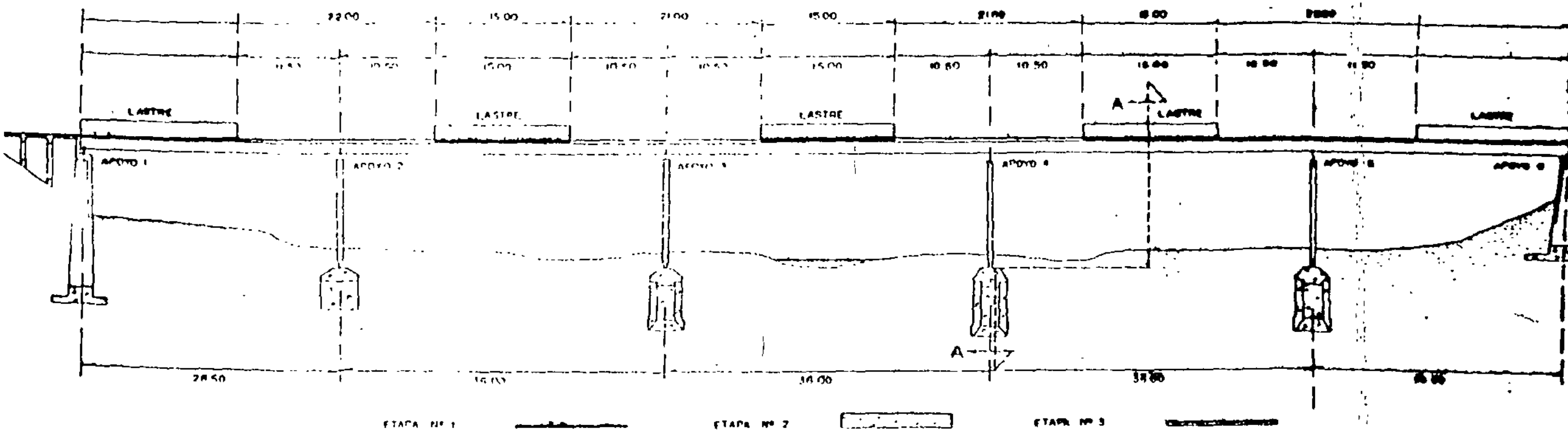
XI MONTAJE DE TRABES METALICAS



XI-1



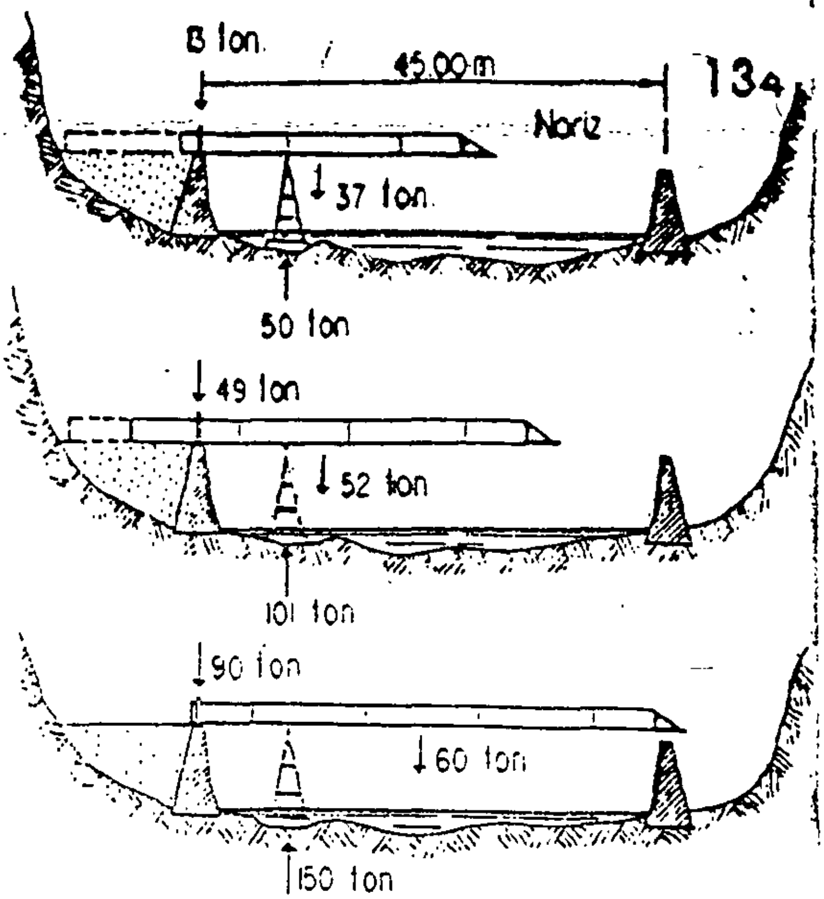
PROCEDIMIENTO DE MONTAJE POR LANZAMIENTO



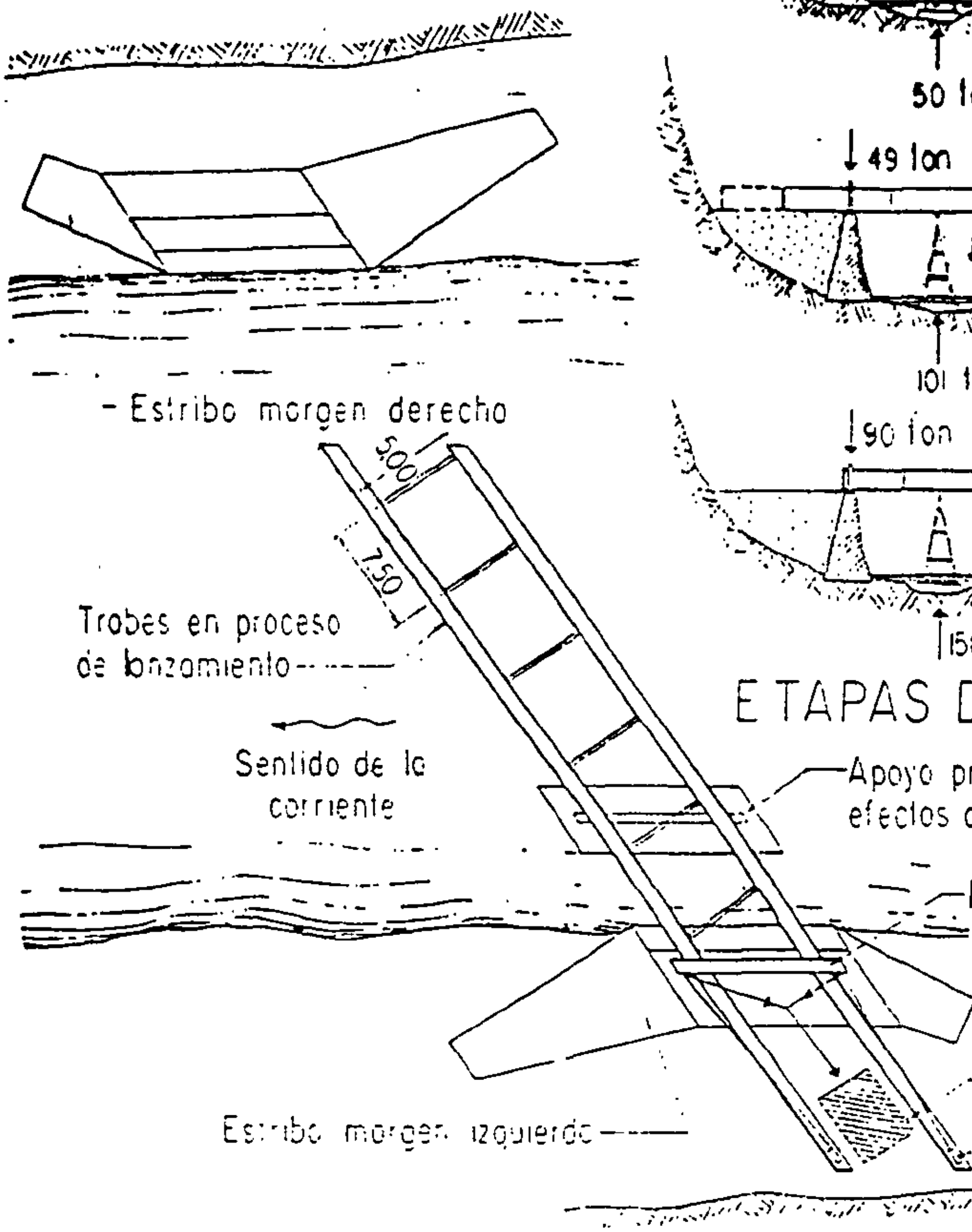
ORDEN DE COLADO DE LA LOSA SOBRE VIGUETAS Y LASTRES ESPECIFICADOS

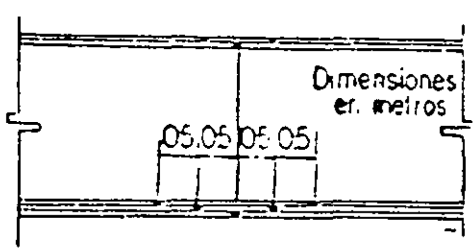
DIMENSIONES EN METROS

MANIOBRAS DE MONTAJE



ETAPAS DEL LANZAMIENTO





DETALLE UNION DE DOVELAS

PUENTE ROMULO O'FARRILL

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE Y TENDIDO DE ESTRUCTURAS DE LÍNEAS DE TRANSMISIÓN Y SU RELACIÓN CON EL DISEÑO

ING. DAMASO RÓLDAN FLORES
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

Standards

In April 1980, the Board of Direction approved ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by concensus standards process managed by the Management Group F on Codes and Standards. The consensus process includes balloting by the balanced standards committee made up of Society members and nonmembers, balloting by the membership of ASCE as a whole, and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding 5 years.

ANSI/ASCE 1-82 N-725 *Guideline for Design and Analysis of Nuclear Safety Related Earth Structures*

ANSI/ASCE 2-84 *Measurement of Oxygen Transfer in Clean Water*

ANSI/ASCE 3-84 *Specifications for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs*

ANSI/ASCE 4-86 *Seismic Analysis of Safety-Related Nuclear Structures*

Building Code Requirements for Masonry Structures (AC1530-88/ASCE5-88) and *Specifications for Masonry Structures* (AC1530.1-88/ASCE6-88)

Specifications for Masonry Structures (AC1530.1-88/ASCE6-88)

ANSI/ASCE 7-88 *Minimum Design Loads for Buildings and Other Structures*

ANSI/ASCE 8-90 *Specification for the Design of Cold-Formed Stainless Steel Structural Members*

ANSI/ASCE 10-90 *Design of Latticed Steel Transmission Structures*

ANSI/ASCE 11-90 *Guideline for Structural Condition Assessment of Existing Buildings*

Foreword

The material presented in this publication has been prepared in accordance with recognized engineering principles. This Standard and Commentary should not be used without first securing competent advice with respect to their suitability for any given application. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Society of Civil Engineers, or of any other person named herein, that this information is suitable for any general or particular use or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

Acknowledgments

To the memory of
GENE M. WILHOITE
Past Chairman of the Standards Committee
for Design of Steel Transmission Towers.
His leadership and his commitment to the advancement
of transmission line structural engineering
contributed in large measure
to the development of this Standard.

In 1971, the American Society of Civil Engineers (ASCE) published the *Guide for Design of Steel Transmission Towers, Manuals and Reports on Engineering Practice—No. 52*. Manual 52 has been used extensively in the United States and abroad as the basis for design specifications. In 1984, an ASCE task committee was established for updating Manual 52 to reflect new design procedures, availability of new shapes and materials, changes in loading criteria, and results of new test data. The second edition was published in 1988. In 1986, it was proposed that ASCE form a committee to develop a standard. The committee was established in 1987. The second edition of Manual 52 served as a resource in developing this Standard, although some of the formulas differ slightly from those in the Manual. The previous work of ASCE task committee on Manual 52 is greatly appreciated.

This Standard was prepared through the concensus standards process by balloting in compliance with procedures of ASCE's Management Group F on Codes and Standards. Those individuals who serve on the Design of Steel Transmission Towers Standards Committee are:

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Design of Latticed Steel Transmission Structures

1.0 SCOPE

Design of Latticed Steel Transmission Structures specifies requirements for the design, fabrication, and testing of members and connections for electrical transmission structures. These requirements are applicable to hot-rolled and cold-formed steel shapes. Structure components (members, connections, guys) are selected to resist design factored loads at stresses approaching yielding, buckling, fracture, or any other limiting condition specified in this Standard.

2.0 APPLICABLE DOCUMENTS

The following standards are referred to in the body of this document:

American Society for Testing Materials (ASTM) Standards:

A6/A6M REV A-90 *Standard Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use*
A36/A36M-90 *Standard Specifications for Structural Steel*
A123 REV A-89 *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*
A143-74 *Standard Practice for Safeguarding Against Embrittlement of Hot-Dip Galvanized Structural Steel Products and Procedure for Detecting Embrittlement*
A153-82 *Standard Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware*
A441-85 *High-Strength Low-Alloy Structural Manganese Vanadium Steel*
A242/A242M-89 *Standard Specification for High-Strength Low-Alloy Structural Steel*
A394-90 *Standard Specification for Zinc-Coated Steel Transmission Tower Bolts*
A529/A529M-89 *Standard Specification for Structural Steel with 42 ksi (290 MPa) Minimum Yield Point (1/2 in. [13mm] Maximum Thickness)*
A563-90 *Standard Specification for Carbon and Alloy Steel Nuts*
A563M-90 Standard Specification for Carbon and Alloy Steel Nuts (Metric)
A570/A570M-90 *Standard Specification for Steel, Sheet and Strip, Carbon, Hot-Rolled Structural Quality*
A572/A572M REV C-88 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality*
A588/A588M REV A-88 *Standard Specification for High-Strength Low-Alloy Structural steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100mm) Thick*
A606-90 *Standard Specification for Steel, Sheet and Strip, High-Strength Low-Alloy Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*
A607 REV A 90 *Standard Specification for Steel, Sheet and Strip, High-Strength Low-Alloy Columbium or Vanadium, or Both, Hot-Rolled and Cold-Rolled*
A715-90 *Standard Specification for Steel, Sheet and Strip, High-Strength Low-Alloy Hot-Rolled and Cold-Rolled, and Steel Sheet, Cold-Rolled, High-Strength Low-Alloy with Improved Formability*

American Welding Society Standard:

AWS D1.1-90 *Structural Welding Code Steel*

3.0 DEFINITIONS

BLOCK SHEAR: A combination of shear and tensile failure through the end connection of a member caused by high bolt forces acting on the material, also called rupture.

DEFORMED BARS: Steel bars meeting the requirements of ACI 318 for reinforcing bars.

DESIGN FACTORED LOAD: Unfactored load multiplied by a specified load factor to establish the design load on a structure.

DOWNTHRUST: The downward vertical component of the loads on a foundation.

LEG MEMBER: A primary member that serves as the main corner support member of a structure. Sometimes called a post member.

LINE SECURITY: Criteria established to prevent a progressive (cascade) failure of structures.

LOAD FACTOR: A multiplier used with the assumed loading condition, or unfactored load, to establish the design factored load.

PRIMARY MEMBERS: Tension or compression members which carry the loads on the structure to the foundation.

REDUNDANT MEMBERS: Members which reduce the unbraced length of primary members by providing intermediate support.

Para: Ing. Jesús Jonce
De: Ing. Francisco Alcalá

SHEAR FRICTION. For anchor bolts with the base assembly resting on concrete, shear is usually transferred from the base assembly to the concrete through bearing of the bolt at the surface forming a concrete wedge; translation of the wedge under the shear force cannot occur without an upward thrust of the wedge on the base assembly; this thrust induces a clamping force, and this mechanism is called shear friction.

TENSION-ONLY MEMBER: Member with L/r greater than 300, which is assumed to be unable to resist compression.

UNFACTORED LOAD: Load on a structure caused by an assumed loading condition on the wires and/or the structure; the assumed loading condition is usually identified by a combination of wind and/or ice and a temperature condition.

UPLIFT: The upward vertical component of the loads on a foundation.

4.0 LOADING, GEOMETRY, AND ANALYSIS

4.1 INTRODUCTION

This Standard applies to latticed steel transmission structures. These structures may be either self-supporting or guyed. They consist of hot-rolled or cold-formed prismatic members connected by bolts. Structure components (members, connections, guys) are selected to resist design factored loads at stresses approaching failure in yielding, buckling, fracture, or any other specified limiting condition.

4.2 LOADS

Design factored loads shall be determined by the purchaser and shown in the job specification either as load trees or in tabular form. These design loads shall consider: (1) Minimum legislated levels; (2) expected climatic conditions; (3) line security provisions; and (4) construction and maintenance operations.

4.3 GEOMETRIC CONFIGURATIONS

Latticed steel structures shall be designed with geometric configurations based on electrical, economic, and safety requirements.

4.4 METHODS OF ANALYSIS

Member forces caused by the design factored loads shall be determined by established principles of structural analysis.

5.0 DESIGN OF MEMBERS

5.1 INTRODUCTION

The provisions of this section are intended to apply to the design of hot-rolled and cold-formed members.

5.2 MATERIAL

Material conforming to the following standard specifications is suitable for use under this Standard:

ASTM A36, Structural Steel;

ASTM A242, High-Strength Low-Alloy Structural Steel;

ASTM A441, High-Strength, Low-Alloy Structural Manganese Vanadium Steel;

ASTM A529, Structural Steel with 42,000 psi Minimum Yield Point;

ASTM A570, Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality;

ASTM A572, High-Strength Low-Alloy Structural Columbium-Vanadium Steels of Structural Quality;

ASTM A588, High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4-in. Thick;

ASTM A606, Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength Low-Alloy, with Improved Corrosion Resistance;

ASTM A607, Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength Low-Alloy, Columbian or Vanadium; and

ASTM A715, Steel Sheet and Strip, Hot-Rolled, High Strength, Low-Alloy, with Improved Formability.

This listing of suitable steels does not exclude the use of other steels which conform to the chemical and mechanical properties of one of the listed specifications or other published specifications.

which establish the properties and suitability of the material.

5.3 MINIMUM SIZES

Minimum thicknesses of 1/8 in. (3 mm) for members and 3/16 in. (5 mm) for connection plates are suggested. See Section 9.3 for steel exposed to corrosion at the ground line.

5.4 SLENDERNESS RATIOS

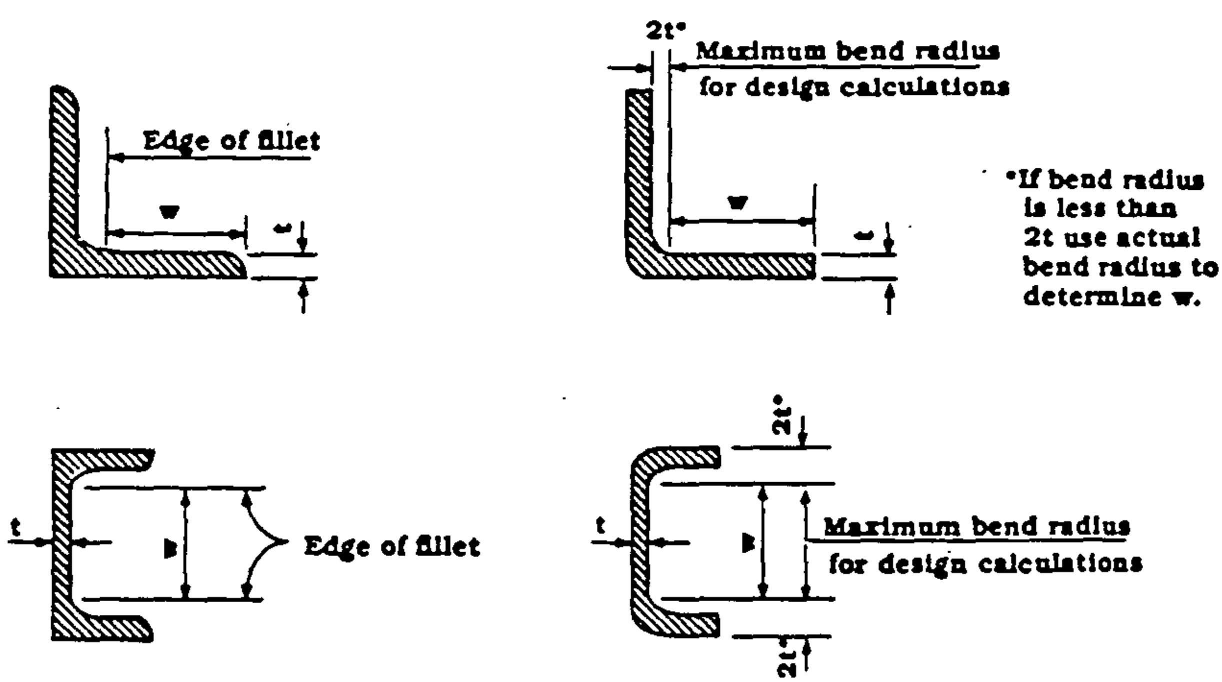
Limiting slenderness ratios for members carrying calculated compressive stress shall be leg members: $L/r \leq 150$; other members: $KL/r \leq 200$. The slenderness ratio KL/r for redundant members shall not exceed 250. The slenderness ratio L/r for tension-only members detailed with draw shall not exceed 500. (See Commentary on Section 5 for hangers.)

5.5 PROPERTIES OF SECTIONS

Section properties, such as area, moment of inertia, radius of gyration, section modulus, etc., shall be based on the gross cross section except where a reduced cross section or a net cross section is specified. The reduced cross section shall consist of all fully effective elements plus those whose widths must be considered reduced in accordance with Section 5.9.3. If all elements are fully effective, the reduced cross section and the gross cross section are identical. Net cross section is defined in Section 5.10.1.

Typical cross sections are shown in Fig. 5C.1 of the Commentary on Section 5. The x- and y-axes are principal axes for all cross sections shown except the angle, for which the principal axes are u and z, with u being the axis of symmetry for equal leg angles.

Fig. 5.1(a) shows the method of determining w/t, the ratio of flat width to thickness of a member element. For hot-rolled sections, w is the



(a) (b)

FIG. 5.1. Determination of w/t Ratios

distance from the edge of the fillet to the extreme fiber, while for cold-formed members it is the distance shown in Fig. 5.1(b). A larger bend radius can be used in fabrication, but for design purposes w shall be based on a maximum inside-bend radius of two times the element thickness.

5.6 ALLOWABLE COMPRESSION

The allowable compression stress F_a on the gross cross-sectional area, or on the reduced area where specified, of axially loaded compression members shall be:

$$F_a = \left[1 - \frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]F_y \qquad \frac{KL}{r} \leq C_c \qquad (5.6\text{-}1)$$

$$F_a = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \frac{KL}{r} \geq C_c \qquad (5.6\text{-}2)$$

$$C_c = \pi\sqrt{\frac{2E}{F_y}} \qquad (5.6\text{-}3)$$

where:

= minimum guaranteed yield stress;
E = modulus of elasticity;
L = unbraced length;
r = radius of gyration; and
K = effective length coefficient

5.7 COMPRESSION MEMBERS: ANGLES

The provisions of this section are applicable only for 90° angles. If the angle legs are closed, as in a 60° angle, the provisions of Section 5.9 shall be followed.

5.7.1 Maximum w/t Ratio

The ratio w/t, where w = flat width and t = thickness of leg, shall not exceed 25; see Fig. 5.1

5.7.2 Allowable Compressive Stress

The allowable compressive stress on the gross cross-sectional area shall be the value of F_a according to Section 5.6, provided the largest value of w/t does not exceed the limiting value given by Eq. 5.7-1.

5.7.3 Determination of F_a

If w/t as defined in Section 5.7.1 exceeds $(w/t)_{lim}$ given by:

$$\left(\frac{w}{t}\right)_{lim} = \frac{80\Psi}{\sqrt{F_y}} \qquad (5.7\text{-}1)$$

the allowable stress F_a shall be the value according to Section 5.6 with F_y in Eqs. 5.6-1 and 5.6-3 replaced with F_{cr} given by:

$$F_{cr} = \left[1.677 - 0.677\frac{w/t}{(w/t)_{lim}}\right]F_y$$
$$\left(\frac{w}{t}\right)_{lim} \leq \frac{w}{t} \leq \frac{144\Psi}{\sqrt{F_y}} \qquad (5.7\text{-}2)$$

$$F_{cr} = \frac{0.0332\,\pi^2 E}{(w/t)^2}$$
$$\frac{w}{t} \geq \frac{144\Psi}{\sqrt{F_y}} \qquad (5.7\text{-}3)$$

For Eqs. 5.7-1 through 5.7-3, Ψ = 1 for F_y in ksi and 2.62 for F_y in MPa.

5.7.4 Effective Lengths

5.7.4.1 Leg Members

For leg members bolted in both faces at connections:

$$\frac{KL}{r} = \frac{L}{r} \qquad 0 \leq \frac{L}{r} \leq 150 \qquad (5.7\text{-}4)$$

5.7.4.2 Other Compression Members

For members with a concentric load at both ends of the unsupported panel:

$$\frac{KL}{r} = \frac{L}{r} \qquad 0 \leq \frac{L}{r} \leq 120 \qquad (5.7\text{-}5)$$

For members with a concentric load at one end and normal framing eccentricity at the other end of the unsupported panel:

$$\frac{KL}{r} = 30 + 0.75\frac{L}{r} \qquad 0 \leq \frac{L}{r} \leq 120 \qquad (5.7\text{-}6)$$

$$r_t = \sqrt{\frac{C_w + 0.04\,J\,(K_tL)^2}{I_{ps}}} \qquad (5.8\text{-}2)$$

where:

C_w = warping constant;
J = St. Venant torsion constant;
K_t = effective length coefficient for warping restraint;
L = unbraced length of member;
r_u = radius of gyration about u-axis;
u_o = distance between shear center and centroid;
r_{ps} = $\sqrt{I_{ps}/A}$ = polar radius of gyration about shear center;
I_{ps} = $I_u + I_z + Au_o^2$ = polar moment of inertia about shear center;
I_u = moment of inertia about u-axis;
I_z = moment of inertia about z-axis; and
A = area of cross section.

Values of KL/r and KL/r_e shall be determined as defined in accordance with Section 5.7.4, using $K_t = 1$ to compute r_t by Eq. 5.8-2.

5.8.4 Minimum Lip Depth

The mininum depth, d, of a lip at the angle θ with the leg (Fig. 5.2) shall be determined by:

$$d = \frac{2.8t}{(\sin\theta)^{1/3}} \sqrt[6]{\left(\frac{w}{t}\right)^2 - \frac{4000\Psi}{F_y}} \geq \frac{4.8t}{(\sin\theta)} \qquad (5.8\text{-}3)$$

where:

Ψ = 1 for F_y in ksi and 6.89 for F_y in MPa;
w/t = flat width to thickness ratio of the leg.

The ratio w_l/t of the lip (shown in Fig. 5.2) shall not exceed $72\Psi/\sqrt{F_y}$; where $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa.

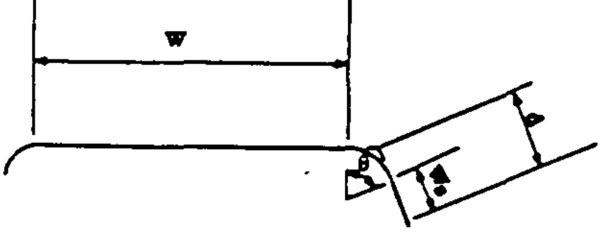


FIG. 5.2. Minimum Lip Depth

5.9 COMPRESSION MEMBERS NOT COVERED IN SECTIONS 5.7 AND 5.8

5.9.1 Allowable Compression Stress

The allowable compressive stress on the gross cross-sectional area, or on the reduced area defined in Section 5.5 if w/t for any element exceeds the limit in Section 5.9.3.1 for which $b = w$, shall be the value of F_a according to Section 5.6. Radii of gyration used to determine F_a shall be computed for the gross cross section, and limiting values of w/t and the effective widths of elements defined in Section 5.9.3.1 shall be determined with $f = F/A$.

If a reduced area applies and the force P does not act at the center of gravity of the reduced area, the resulting moment shall be taken into account according to Section 5.12.

5.9.2 Maximum w/t Ratios

The ratio w/t of flat width to thickness shall not exceed 60 for elements supported on both longitudinal edges and 25 for elements supported on only one longitudinal edge; see Fig. 5.1.

5.9.3 Effective Widths of Elements in Compression

5.9.3.1 Uniformly Compressed Elements

For Eqs. 5.9-1 through 5.9-6, $\Psi = 1$ for f in ksi and 2.62 for f in MPa.

(a) The effective width b of an element supported on only one longitudinal edge shall be taken as follows:

$$b = w \qquad \frac{w}{t} \leq \frac{72\Psi}{\sqrt{f}} \qquad (5.9\text{-}1)$$

$$b = \frac{108\Psi}{\sqrt{f}}\left(1 - \frac{24\Psi}{(w/t)\sqrt{f}}\right)t \qquad \frac{w}{t} \geq \frac{72\Psi}{\sqrt{f}} \qquad (5.9\text{-}2)$$

where f = compressive stress in an element computed for compression members as prescribed in Section 5.9.1 and for members in bending in Section 5.14.1. The effective width shall be taken adjacent to the supported edge.

(b) The effective width b of an element supported on both longitudinal edges shall be taken as follows:

$$b = w \qquad \frac{w}{t} \leq \frac{220\Psi}{\sqrt{f}} \qquad (5.9\text{-}3)$$

For members with normal framing eccentricities at both ends of the unsupported panel:

$$\frac{KL}{r} = 60 + 0.5\,\frac{L}{r} \qquad 0 \le \frac{L}{r} \le 120 \qquad (5.7\text{-}7)$$

For members unrestrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = \frac{L}{r} \qquad 120 \le \frac{L}{r} \le 200 \qquad (5.7\text{-}8)$$

For members partially restrained against rotation at one end of the unsupported panel:

$$\frac{KL}{r} = 28.6 + 0.762\,\frac{L}{r}$$

$$120 \le \frac{L}{r} \le 225 \qquad (5.7\text{-}9)$$

For members partially restrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = 46.2 + 0.615\,\frac{L}{r}$$

$$120 \le \frac{L}{r} \le 250 \qquad (5.7\text{-}10)$$

5.7.4.3 Redundant Members

$$\frac{KL}{r} = \frac{L}{r} \qquad 0 \le \frac{L}{r} \le 120 \qquad (5.7\text{-}11)$$

If members are unrestrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = \frac{L}{r} \qquad 120 \le \frac{L}{r} \le 250 \qquad (5.7\text{-}12)$$

If members are partially restrained against rotation at one end of the unsupported panel:

$$\frac{KL}{r} = 28.6 + 0.762\,\frac{L}{r}$$

$$120 \le \frac{L}{r} \le 290 \qquad (5.7\text{-}13)$$

If members are partially restrained against rotation at both ends of the unsupported panel:

$$\frac{KL}{r} = 46.2 + 0.615\,\frac{L}{r}$$

$$120 \le \frac{L}{r} \le 330 \qquad (5.7\text{-}14)$$

5.7.4.4 Joint Restraint

A single bolt connection at either the end of a member or a point of intermediate support shall not be considered as furnishing restraint against rotation. A multiple bolt connection, detailed to minimize eccentricity, shall be considered to offer partial restraint if the connection is to a member capable of resisting rotation of the joint (see Fig. 5C.2 in the Commentary on Section 5).

5.7.4.5 Test Verification

Where tests and/or analysis demonstrate that specific details provide restraint different from the recommendations of this section, the values of KL/r specified in this section may be modified.

5.8 COMPRESSION MEMBERS: SYMMETRICAL LIPPED ANGLES

5.8.1 Maximum w/t Ratio

The ratio w/t of the leg shall not exceed 60; see Fig. 5.1.

5.8.2 Allowable Compressive Stress

The allowable compressive stress on the gross cross-sectional area shall be the value of F_a according to Section 5.6, provided the width-to-thickness ratio of the leg $w/t \le 220\ \Psi/\sqrt{Fy}$, where $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa. If w/t exceeds $220\ \Psi/\sqrt{Fy}$, the design shall be based on a reduced area according to Sections 5.5 and 5.9.3.1.b.

5.8.3 Equivalent Radius of Gyration

The allowable stress defined in Section 5.8.2 shall be computed for the larger of KL/r_z and KL/r_{eq}, where r_{eq} is an equivalent radius of gyration given by:

$$\frac{2}{r_{eq}^2} = \frac{1}{r_i^2} + \frac{1}{r_o^2} + \sqrt{\left(\frac{1}{r_i^2} - \frac{1}{r_o^2}\right)^2 + 4\left(\frac{u_o}{r_i\, r_o\, r_{ps}}\right)^2}$$

(5.8-1)

$$b = \frac{325\Psi}{\sqrt{f}}\left(1 - \frac{71\Psi}{(w/t)\sqrt{f}}\right)t \qquad \frac{w}{t} \geq \frac{220\Psi}{\sqrt{f}} \qquad (5.9\text{-}4)$$

except that for flanges of square and rectangular sections:

$$b = w \qquad \frac{w}{t} \leq \frac{240\Psi}{\sqrt{f}} \qquad (5.9\text{-}5)$$

$$b = \frac{325\Psi}{\sqrt{f}}\left(1 - \frac{63\Psi}{(w/t)\sqrt{f}}\right)t \qquad \frac{w}{t} \geq \frac{240\Psi}{\sqrt{f}} \qquad (5.9\text{-}6)$$

where f = compressive stress in an element computed for compression members as prescribed in Section 5.9.1 and for members in bending in Section 5.14.1. The portion of the element considered removed to obtain the effective width shall be taken symmetrically about the centerline.

5.9.3.2 Elements with Stress Gradient

For Eqs. 5.9-7 and 5.9-8, $\Psi = 1$ for f_1 in ksi and 2.62 for f_1 in MPa.

(a) The effective width b of an element supported on only one longitudinal edge shall be determined as in Section 5.9.3.1.a, using for f the maximum compressive stress in the element.

(b) The effective widths b_1 and b_2 (Fig. 5.3) of an element supported on both longitudinal edges shall be determined as follows:

$$b_2 = \frac{w}{2} \qquad \frac{w}{t} \leq \frac{110\,C\,\Psi}{\sqrt{f_1}} \qquad (5.9\text{-}7)$$

$$b_2 = \frac{82\,C\,\Psi}{\sqrt{f_1}}\left(1 - \frac{1.36\,C\,\Psi}{(w/t)\sqrt{f_1}}\right)t, \text{ if}$$

$$\frac{w}{t} \geq \frac{110\,C\,\Psi}{\sqrt{f_1}} \qquad (5.9\text{-}8)$$

$$b_1 = \frac{b_2}{1.5 - 0.5\left(\frac{f_2}{f_1}\right)} \qquad (5.9\text{-}9)$$

where

$C = 2 + 0.75\,(1 - f_2/f_1)^2$;

f_1 = compressive stress shown in Fig. 5.3, to be taken positive; and

f_2 = stress shown in Fig. 5.3; positive indicates compression, negative indicates tension.

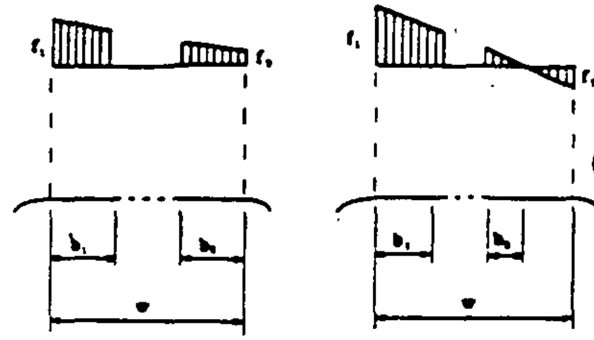


FIG. 5.3. Elements with Stress Gradient

The stresses f_1 and f_2 shall be based on the reduced section, and f_1 shall be the larger if f_2 is compressive. If the sum of the calculated values of b_1 and b_2 exceeds the compressive part of the element, the element is fully effective.

5.9.4 Doubly Symmetric Open Cross Sections

Members with doubly symmetric open cross sections whose unsupported length for torsional buckling exceeds the unsupported length for flexural buckling about the weak axis shall be checked for torsional buckling as well as for flexural buckling. The allowable torsional-buckling stress is the value of F_a according to Section 5.6, using the radius of gyration r_t of Eq. 5.8-2 computed for the gross cross section.

5.9.5 Singly Symmetric Open Cross Sections

Members with singly symmetric open cross sections shall be checked for flexural buckling in the plane of symmetry and for torsional-flexural buckling. The allowable torsional-flexural buckling stress is the value of F_a according to Section 5.6, using the radius of gyration r_{tf} of Eq. 5.8-1 computed for the gross cross section.*

5.9.6 Point-Symmetric Open Cross Sections

Members with point-symmetric open cross sections shall be checked for torsional buckling as well as flexural buckling. The allowable torsion-flexural buckling stress is the value of F_a according to Section 5.6, using the radius of gyration r_t of Eq. 5.8-2, computed for the gross cross section.

* Note that r_t and r_{tf} refer to the principal axes [illegible] of angles. See Commentary Section SC 9.3 for conversion to principal axes of other shapes.

5.9.7 Closed Cross Sections

Members with closed cross sections need to be investigated only for flexural buckling.

5.9.8 Nonsymmetric Cross Sections

The allowable compressive stress for nonsymmetric shapes shall be determined by tests and/or analysis. See Commentary Section 5C.9.8.

5.9.9 Lips

Element lips shall be dimensioned according to Section 5.8.4.

5.9.10 Eccentric Connections

If the centers of gravity of the member connections cannot be made coincident with the center of gravity of the member cross section, either gross or reduced as applicable, the resulting bending stresses shall be taken into account according to Section 5.12.

5.10 TENSION MEMBERS

5.10.1 Allowable Tensile Stress

The allowable tensile stress F_t on concentrically loaded tension members shall be F_y on the net cross-sectional area A_n, where A_n is the gross cross-sectional area A_g (the sum of the products of the thickness and the gross width of each element as measured normal to the axis of the member) minus the loss due to holes or other openings at the section being investigated. Hole diameters shall be as shown on the detail drawings. If there is a chain of holes in a diagonal or zigzag line, the net width of an element shall be determined by deducting from the gross width the sum of the diameters of all the holes in the chain and adding for each gage space in the chain the quantity $s^2/4g$, where s = longitudinal spacing (pitch) and g = transverse spacing (gage) of any two consecutive holes. The critical net cross-sectional area A_n is obtained from that chain which gives the least net width.

Plain and lipped angles bolted in both legs at both ends shall be considered to be concentrically loaded.

5.10.2 Angle Members

The allowable tensile stress F_t on the net area of plain and lipped angles connected by one leg shall be $0.9F_y$. If the legs are unequal and the short leg is connected, the unconnected leg shall be considered to be the same size as the connected leg. If the centroid of the bolt pattern on the connected leg is outside the center of gravity of the angle, the connection shall be checked for rupture (also called block shear) by:

$$P = 0.60\,A_v F_u + A_t F_y \tag{5.10-1}$$

where:

P = allowable tensile force on connection;
F_y = specified minimum yield strength of the member;
F_u = specified minimum tensile strength of the member;
A_v = minimum net area in shear along a line of transmitted force, see Fig. 5.4; and
A_t = minimum net area in tension from the hole to the toe of the angle perpendicular to the line of force; see Fig. 5.4.

5.10.3 Eccentric Connections

Eccentricity of load on angle members is provided for in Section 5.10.2. Other members subjected to both axial tension and bending shall be proportioned according to Section 5.13

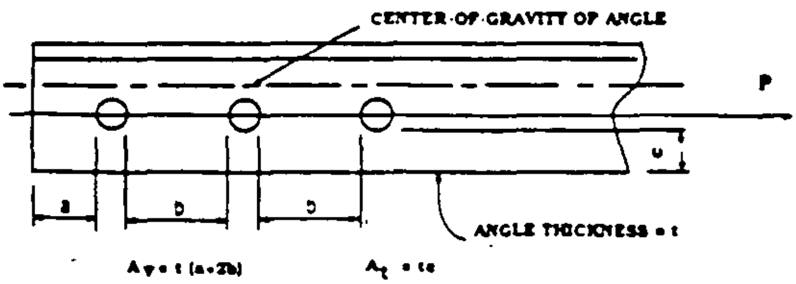


FIG. 5.4. Rupture (Block Shear) Determination

5.10.4 Threaded Rods and Anchor Bolts

Threaded-rod members shall have a minimum guaranteed yield F_y. The allowable tensile stress F_t on the stress area A_s shall be F_y. A_s is given by:

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \tag{5.10-2}$$

where:

d = nominal diameter; and
n = number of threads per unit of length.

Anchor bolts shall have a minimum guaranteed yield F_y. See Sections 6.3.2, 6.3.3, 6.3.4, and Section 9 for design requirements.

5.10.5 Guys

The allowable tension in guys shall not exceed 0.65 times the specified minimum breaking strength of the cable. See Commentary Section 5C.10.5 for recommendations on stretch of cables.

5.11 STITCH BOLTS

Stitch bolts shall be spaced so that the governing slenderness ratio between bolts for any component of the built-up member does not exceed the following:

For compression members: Three-quarters of the governing slenderness ratio of the built-up member.

For tension members: The governing slenderness ratio of the built-up member, or 300.

If the connected leg of a compression member exceeds 4 in. (100 mm), a minimum of two bolts shall be used at each stitch point.

5.12 AXIAL COMPRESSION AND BENDING

Eccentricity of load on angle members is provided for in Sections 5.7.4.2. and 5.8.3. Other members subjected to both axial compression and bending shall be proportioned to satisfy the following equations:

$$\frac{P}{P_a} + \frac{C_m M_x}{M_{ax}}\frac{1}{1 - P/P_{ex}} + \frac{C_m M_y}{M_{ay}}\frac{1}{1 - P/P_{ey}} \leq 1 \tag{5.12-1}$$

$$\frac{P}{P_y} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1 \tag{5.12-2}$$

where:

C_m = coefficient defined below;
P = axial compression;
P_a = allowable axial compression according to Section 5.9;
P_y = axial compression at yield ($= F_y A$);
P_{ex} = $\pi^2 EI_x/(K_x L_x)^2$;
P_{ey} = $\pi^2 EI_y/(K_y L_y)^2$;
I_x = moment of inertia about the x-axis;
I_y = moment of inertia about the y-axis;
$K_x L_x$, $K_y L_y$ = effective lengths in the corresponding planes of bending;
M_x, M_y = moments about the x- and y-axes, respectively, at the point or points defined below; and
M_{ax}, M_{ay} = corresponding allowable moments according to Section 5.14 computed with $C_b = 1$ if Section 5.14.4 applies.

If there are transverse loads between points of support, M_x (and M_y) in Eq. 5.12-1 is the maximum moment *between* these points, while in Eq. 5.12-2 it is the larger of the moments *at* these points. If there are no transverse loads between points of support, M_x (and M_y) in both Eq. 5.12-1 and Eq. 5.12-2 is the larger of the values of M_x (and M_y) at these points.

For restrained members with no lateral displacement of one end relative to the other and with no transverse loads in the plane of bending between supports, $C_m = 0.6 - 0.4M_1/M_2$, where M_1 is the smaller end moment and M_1/M_2 is positive when bending is in reverse (S) curvature and negative when it is in single curvature. If there are transverse loads between supports, $C_m = 1$ for members with unrestrained ends and 0.85 if the ends are restrained.

5.13 AXIAL TENSION AND BENDING

Eccentricity of load on angle members is provided for in Sections 5.10.1 and 5.10.2. Other members subjected to both axial tension and bending shall be proportioned to satisfy the following formula:

$$\frac{P}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1 \tag{5.13-1}$$

where:

P = axial tension;
P_a = allowable axial tension according to Section 5.10;
M_x, M_y = the moments about the x- and y-axes, respectively; and
M_{ax}, M_{ay} = the corresponding allowable moments according to Section 5.14.

5.14 BEAMS

5.14.1 Properties of Sections

Allowable bending moments shall be determined by multiplying allowable bending stresses F_b prescribed in the following sections by the section modulus of the gross cross section or of the reduced section defined in Section 5.5, as applicable. Radii of gyration used to determine the value of F_b for the extreme fiber in compression shall be based on the gross cross section. Effective widths of section elements shall be determined as prescribed in Section 5.9.3, using for f the stress on the element corresponding to the allowable moment defined above. Limiting values of w/t shall be those given in Section 5.9.2.

5.14.2 Allowable Tension

The allowable bending stress F_b on the extreme fiber in tension shall be F_y.

5.14.3 Laterally Supported Beams

The allowable bending stress F_b on the extreme fiber in compression for members supported against lateral buckling shall be F_y.

5.14.4 I, Channel, and Cruciform Sections

The allowable bending stress F_b on the extreme fiber in compression for doubly symmetric I sections, singly symmetric channels, and singly or doubly symmetric cruciform sections in bending about the x-axis (the x-axis is to be taken perpendicular to the web of the I and channel, but may be either principal axis for the cruciform) and not supported against lateral buckling, shall be the value of F_a according to Section 5.6 with $K = \sqrt{K_y K_t}$ and r given by:

$$r^2 = \frac{C_b\sqrt{I_y}}{S_x}\sqrt{C_w + 0.04J(K_tL)^2} \qquad (5.14\text{-}1)$$

where:

K_y = effective-length coefficient for y-axis bending;
K_t = effective-length coefficient for warping restraint;
I_y = moment of inertia about y-axis;
S_x = x-axis section modulus;
C_w = warping constant;
J = St. Venant torsion constant; and
L = unbraced length.

For members with moments M_1/M_2 at the ends of the unbraced length

$$C_b = 1.75 + 1.05\,M_1/M_2 + 0.3(M_1/M_2)^2 < 2.3 \qquad (5.14\text{-}2)$$

where M_1 is the smaller end moment and M_1/M_2 is positive when bending is in reverse (S) curvature and negative when it is in single curvature.

For members for which the moment within a significant portion of the unbraced length equals or exceeds the larger of the end moments, and for unbraced cantilevers, $C_b = 1$.

The allowable stress on the extreme fiber in compression for the I section in bending about the y-axis shall be taken equal to F : for channels, see Section 5.14.7.b.

5.14.5 Other Doubly Symmetric Open Sections

The allowable bending stress F_b on the extreme fiber in compression for laterally unsupported members of doubly symmetric open cross section not covered in Section 5.14.4 shall be the value of F_a according to Section 5.6, determined as follows:

For x-axis bending, follow Section 5.14.4 (Eq. 5.14-1); for y-axis bending, follow Section 5.14.4 (Eq. 5.14-1) but with K_x, I_x, and S_y substituted for K_y, I_y, and S_x, respectively.

5.14.6 Singly Symmetric I and T Sections

The allowable bending stress F_b on the extreme fiber in compression for singly symmetric I-shaped members with the compression flange larger than the tension flange and for singly symmetric single-web T-shaped members with the flange in compression, in bending about the x-axis (the axis perpendicular to the web) and not supported against lateral buckling, may be taken the same as the value for a section of the same depth with a tension flange the same as the compression flange of the I or the T section. The allowable moment shall be calculated by multiplying the allowable stress so obtained by the compression flange section modulus of the singly symmetric shape.

The allowable bending stress on the extreme fiber in compression for the sections described, in bending about the y-axis (the axis of symmetry), shall be determined according to Section 5.14.7.a.

5.14.7 Other Singly Symmetric Open Sections

The allowable bending stress F_b on the extreme fiber in compression for members with singly symmetric open cross section and not supported against lateral buckling, other than those covered in Sections 5.14.4 and 5.14.6, shall be the value of F_a determined according to Section 5.6 as follows:

(a) For members in bending about the axis of symmetry (the y-axis is to be taken as the axis of symmetry), use

$K = \sqrt{K_x K_y}$ and r from Eq. 5.14-1.

(b) For members in bending about the x-axis (the axis perpendicular to the axis of symmetry), use $K = K_y$, and r given by:

$$r^2 = \frac{C_b \sqrt{I_y}}{S_{xc}} \left\{ \pm j \sqrt{I_y} + \sqrt{j^2 I_y + \left(\frac{K_y}{K_t}\right)^2 \left[C_w + 0.04J(K_t L)^2\right]} \right\} \quad (5.14\text{-}3)$$

$$j = \left[\frac{1}{2I_x} \int_A (x^2 + y^2) x \, dA\right] - y_o \quad (5.14\text{-}4)$$

where:

S_{xc} = section modulus of compression flange about x-axis;
y_o = distance from centroid to shear center;
A = area of cross section;
I_y = y-axis moment of inertia;
I_x = x-axis moment of inertia; and
K_y = effective-length coefficient for y-axis bending.

C_b, K_t, C_w, J, and L as defined in Eqs. 5.14-1 and 5.14-2.

The positive direction of the y-axis must be taken so that the shear center coordinate y_o is negative. The plus sign for the term $j\sqrt{I_y}$ in Eq. 5.14-3 is to be used if the moment causes compression on the shear center side of the x-axis, and the minus sign if it causes tension.

5.14.8 Equal Leg Angles

Provided the eccentricity of the load with respect to the shear center is not more than one-half of the leg width (Fig. 5.5), the allowable bending moment for a laterally unsupported equal legs angle may be taken as the smaller of:

(a) The moment M_{yt} that produces tensile yield stress at the extreme fiber; or
(b) The moment M_y that causes lateral buckling given by the following:

$$\text{If } M_e \leq 0.5\ M_{yc} \qquad M_y = M_e \quad (5.14\text{-}5)$$

$$\text{If } M_e \geq 0.5\ M_{yc} \qquad M_y = M_{yc}\left(1 - \frac{M_{yc}}{4\ M_e}\right) \quad (5.14\text{-}6)$$

where:

M_{yc} = moment causing compressive yield at extreme fiber; and
M_e = elastic critical moment.

Values of M_e are given by:
For load perpendicular to a leg:

$$M_e = \frac{0.66Eb^4 t}{(KL)^2}\left[\sqrt{1 - \frac{0.81\ (KL)^2 t^2}{b^4}} \pm 1\right] \quad (5.14\text{-}7)$$

For load at the angle θ with the z-axis (Fig. 5.5):

$$M_e = \frac{2.33Eb^4 t}{(1 - 3\cos^2\theta)(KL)^2} \left[\sqrt{\sin^2\theta + \frac{0.162\,(1 - 3\cos^2\theta)\,(KL)^2 t^2}{b^4}} \pm \sin\theta\right] \quad (5.14\text{-}8)$$

where:

E = modulus of elasticity;
F_y = yield stress;
b = width of leg $- t/2$;
t = thickness of leg;
L = unsupported length; and
K = 1 if the angle is simply supported on the x- and y-axes at each end or 0.5 if it is fixed against rotation about the x- and y-axes at each end.

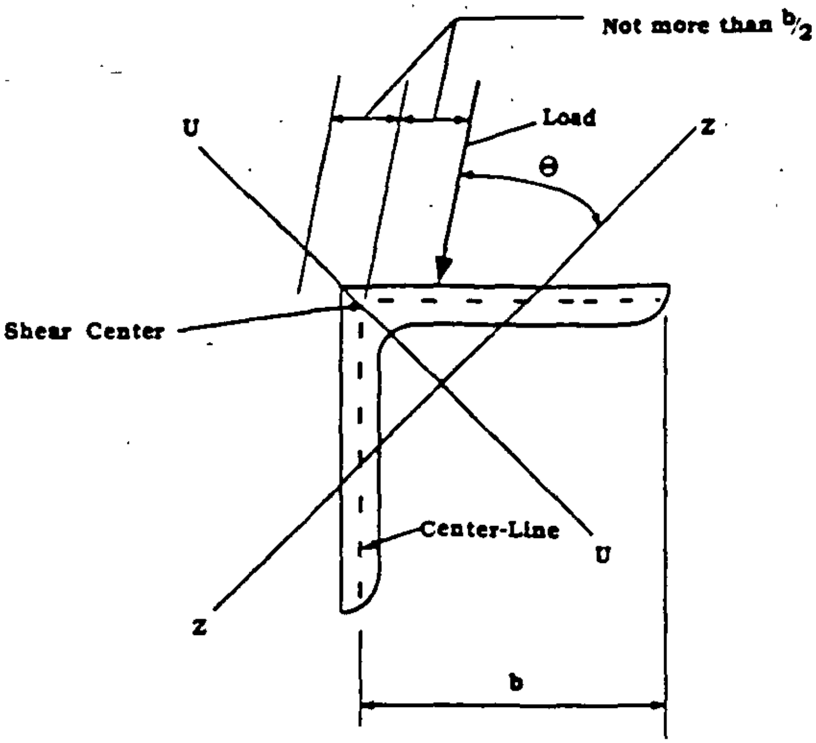


FIG. 5.5. Load on Angles

The plus sign for the last term (±1) in Eq. 5.14-7 and (± sin θ) in Eq. 5.14-8, applies when the load acts in the direction shown in Fig. 5.5 and the minus sign when it acts in the opposite direction.

The yield moments M_u and M_z are given by M_y in the following:

At the heel of the angle:

$$F_v = \pm \frac{M_y \sin\theta}{S_z} \qquad (5.14\text{-}9)$$

At the toe of the angle:

$$F_v = \pm \frac{M_y \sin\theta}{S_z} \pm \frac{M_y \cos\theta}{S_u} \qquad (5.14\text{-}10)$$

where:

S_u and S_z = section moduli for the u- and z-axes, respectively.

The plus sign denotes tension and the minus sign compression. The applicable signs are determined according to the type of stress produced at the extreme fiber being checked. The following section moduli based on centerline dimensions may be used in lieu of those based on overall dimensions:

$$S_u = \frac{b^2 t}{1.5\sqrt{2}} \qquad S_z = \frac{b^2 t}{3\sqrt{2}} \qquad (5.14\text{-}11)$$

5.15 ALLOWABLE SHEAR

5.15.1 Beam Webs

For Eqs. 5.15-1 through 5.15-3, $\Psi = 1$ for F_y in ksi and 2.62 for F_y in MPa.

The ratio h/t of the depth of a beam web to its thickness shall not exceed 200. The allowable average shearing stress F_v on the gross area of a beam web shall not exceed the following:

$$F_v = 0.58F_y \qquad \frac{h}{t} \le \frac{440\,\Psi}{\sqrt{F_y}} \qquad (5.15\text{-}1)$$

$$F_v = \frac{255\sqrt{F_y}}{\Psi\,(h/t)} \qquad \frac{440\,\Psi}{\sqrt{F_y}} \le \frac{h}{t} \le \frac{557\,\Psi}{\sqrt{F_y}} \qquad (5.15\text{-}2)$$

$$F_v = \frac{0.5\,\pi^2 E}{(h/t)^2}; \qquad \frac{h}{t} > \frac{557\,\Psi}{\sqrt{F_y}} \qquad (5.15\text{-}3)$$

where F_y = yield stress.

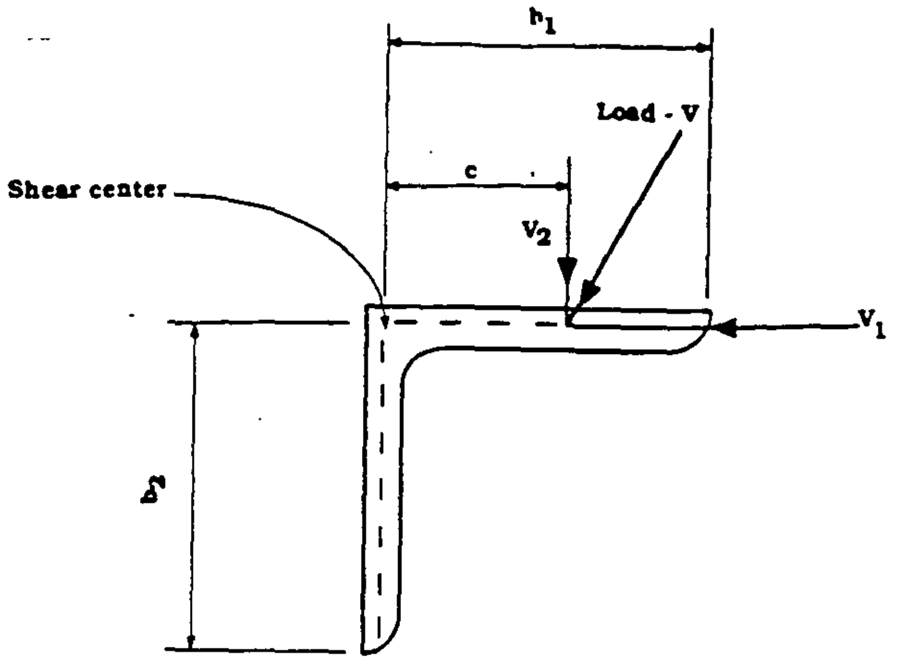


FIG. 5.6. Shear Load on Angles

5.15.2 Angles

The shear components V_1 and V_2 of the allowable shear V on single-angle beams (Fig. 5.6) shall not exceed the value that satisfies the following equations:

$$\frac{3\,V_1}{2b_1t} + \frac{V_2at}{J} \le 0.58\,F_y \qquad (5.15\text{-}4)$$

$$V_2\left(\frac{3}{2b_2t} + \frac{at}{J}\right) \le 0.58\,F_y \qquad (5.15\text{-}5)$$

where:

V_1 = component of V in leg b_1;
V_2 = component of V in leg b_2;
a = distance of shear center to intersection of load plane with leg b_1;
b_1, b_2 = width of leg $- t/2$;
t = thickness of leg;
J = St. Venant torsional constant = $(b_1 + b_2)t^3/3$; and
F_y = yield stress.

5.16 TEST VERIFICATION

Design values other than those prescribed in this Section may be used if substantiated by experimental or analytical investigations.

6.0 DESIGN OF CONNECTIONS

6.1 INTRODUCTION

Bolted connections for transmission structures are normally designed as bearing type connections. It is assumed that bolts connecting one member to another carry the load in the connection equally.

The minimum end and edge distances determined by the provisions of this chapter do not include an allowance for fabrication and rolling tolerances.

Unless otherwise noted, these provisions pertain to standard holes, i.e., holes nominally 1/16 in. (1.6 mm) larger than the bolt diameter.

6.2 GENERAL REQUIREMENTS

The Engineer of Record (EOR) shall approve the shop detail drawings; see Section 7.1.2.

6.3 FASTENERS

6.3.1 Materials

The commonly used fastener specifications for latticed steel transmission towers are ASTM A394 for bolts and A563 for nuts.

6.3.2 Bolt Shear Capacity

Allowable shear for A394 bolts shall be the shear strength tabulated in the ASTM specification.

For bolts that do not have an ASTM-specified shear strength the allowable shear stress F_v on the effective area shall be $0.62F_u$ where F_u is the specified minimum tensile strength of the bolt material. The effective area is the gross cross-sectional area of the bolt if threads are excluded from the shear plane or the root area if threads are in the shear plane.

6.3.3 Bolt Tension Capacity

Bolts shall be proportioned so that the sum of the tensile stresses caused by the applied external load and any tensile stress resulting from prying action does not exceed the allowable tensile stress F_t, as follows:

(a) For bolts having a specified proof-load stress, F_t = ASTM proof-load stress by the length-measurement method.
(b) For bolts with no specified proof-load stress, $F_t = 0.6F_u$.

The stress area A_s is given by:

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \qquad (6.3\text{-}1)$$

where:

d = nominal diameter of the bolt; and
n = number of threads per unit of length.

6.3.4 Bolts Subject to Combined Shear and Tension

For bolts subject to combined shear and tension the allowable tensile stress $F_{t(v)}$ shall be:

$$F_{t(v)} = F_t\,[1 - (f_v/F_v)^2]^{1/2} \qquad (6.3\text{-}2)$$

where:

F_t = allowable tensile stress defined in Section 6.3.3;
F_v = allowable shear stress defined in Section 6.3.2; and
f_v = computed shear stress on effective area.

The combined tensile and shear stresses shall be taken at the same cross section in the bolt.

6.4 ALLOWABLE BEARING STRESS

The maximum bearing stress, calculated as the force on a bolt divided by the product of the bolt diameter times the thickness of the connected part, shall not exceed 1.5 times the specified minimum tensile strength F_u of the connected part or the bolt; see Commentary Section 6C.4.

6.5 MINIMUM DISTANCES

6.5.1 End Distance (See Fig. 6C.2)

For stressed members, the distance e measured from the center of a hole to the end, whether this end is perpendicular or inclined to the line of force, shall not be less than the value of e_{min} determined as the largest value of e from Eqs. 6.5-1, 6.5-2, and 6.5-3:

$$e = 1.2\,P/F_u t \qquad (6.5\text{-}1)$$

$$e = 1.3\,d \qquad (6.5\text{-}2)$$

$$e = t + d/2 \qquad (6.5\text{-}3)$$

where:

F_u = specified minimum tensile strength of the connected part;
t = thickness of the connected part;
d = nominal diameter of bolt; and
P = force transmitted by the bolt.

For redundant members, e_{min} shall be determined as the larger value of e from Eqs. 6.5-3 and 6.5-4:

$$e = 1.2\,d \qquad (6.5\text{-}4)$$

Eq. 6.5-3 does not apply for either stressed members or redundant members if the holes are drilled.

6.5.2 Center-to-Center Bolt Hole Spacing

Along a line of transmitted force, the distances between centers of holes shall not be less than the value of s_{min} determined as:

$$s_{min} = 1.2P/F_u t + 0.6d \qquad (6.5\text{-}5)$$

See Commentary Section 6C.5.2 for suggested minimum spacing requirements for assembly purposes.

6.5.3 Edge Distance, Fig. 6C.2

The distance f from the center of a hole to the edge of the member shall not be less than the value of f_{min} given by:

For a rolled edge:

$$f_{min} = 0.85\ e_{min} \tag{6.5-6}$$

For a sheared or mechanically guided flame-cut edge:

$$f_{min} = 0.85\ e_{min} + 0.0625\Psi \tag{6.5-7}$$

where

e_{min} = end distance according to Section 6.5.1; and
$\Psi = 1$ for f_{min} in in. and 25.4 for f_{min} in mm.

6.6 ATTACHMENT HOLES

This section is valid for hole diameter to bolt diameter ratios ≤ 2. The force P for a bolt in an attachment hole shall be limited by:

$$P \leq 0.75\ (L - 0.5d_h)\ t\ F_u \tag{6.6-1}$$

or:

$$P \leq 1.35\ d\ t\ F_u \tag{6.6-2}$$

where

L = minimum distance from the center of the hole to any member edge;
d = nominal diameter of bolt;
d_h = attachment hole diameter;
t = member thickness; and
F_u = specified minimum tensile strength of the member.

6.7 TEST VERIFICATION

Design values other than those prescribed in this Section may be used if substantiated by experimental or analytical investigations.

7.0 DETAILING AND FABRICATION

7.1 DETAILING

7.1.1 Drawings

Tower detail drawings consist of erection drawings, shop detail drawings, and bills of material. Erection drawings shall show the complete assembly of the structure indicating clearly the positioning of the members. Each member shall be piece-marked and the number and lengths of bolts shall be given for each connection. Shop detail drawings can be shown either by assembled section (in place) or piece by piece (knocked down), either hand drawn or computer drawn. Layout drawings are required when details are not shown by assembled sections. Computer-generated bills of material are generally acceptable.

7.1.2 Approval of Shop Drawings

Shop detail drawings shall be approved by the Engineer of Record (EOR) regarding compliance with the purchaser's specifications and the strength requirements of the design. The EOR shall be the utility structural engineer, a consulting structural engineer, or the fabricator's structural engineer, depending on who generated the structural tower design. The EOR's review and approval of the shop detail drawings include responsibility for the strength of connections but do not pertain to the correctness of dimensional detail calculations, which is the responsibility of the detailer. They also do not imply approval of means, methods, techniques, sequences, or procedure of construction, or of safety precautions and programs.

7.1.3 Connections

Usual detailing practice is to connect members directly to each other with minimum eccentricity. If specific joint details are required by the EOR they shall be shown on the design drawings as referenced in the contract documents.

7.1.4 Bolt Spacing

Minimum bolt spacing, and end and edge distances, as specified in the design sections of this document, shall not be underrun by mill or standard fabrication tolerances. The purchaser's specifications shall state if end distances, edge distances, and center-to-center hole spacing dimensions include provisions for mill and fabrication tolerances; if they do not, dimensions used for detailing must be adjusted to ensure that minimum dimensions are provided in the fabricated member.

7.1.5 Detail Failures During Testing

If a structural failure occurs during testing of a tower, a review shall be made by the EOR to determine the reasons and to specify the required revisions.

7.1.6 Material

Detail drawings shall clearly specify member and connection materials, such as ASTM specification and grade designation.

7.1.7 Weathering Steel

If the structure is made of weathering steel, special detailing procedures may be required; see Brockenbrough (1983) and Brockenbrough and Schmitt (1975).

7.1.8 Tension-Only Members

Tension-only members shall be detailed sufficiently short to provide draw. Draw must consider the length and size of the member. To facilitate erection, these members should have at least two bolts on one end. Members 15 ft (4.6 m) in length, less, are detailed 1/8 in. (3.2 mm) short. Members more than 15 ft (4.6 m) long are detailed 1/8 in. (3.2 mm) short, plus 1/16 in. (1.6 mm) for each additional 10 ft (3.1 m) or fraction thereof. If such members are spliced, the draw should provide for the slippage at the splice.

7.1.9 Shop Check Assembly

The purchaser's specifications should include a requirement for shop assembly of new tower details, to be done partially by sections and in the horizontal position. This helps validate detailing calculations and dimensions, minimize fit-up conflicts, and assure proper assembly in the field.

7.1.10 Other Considerations

All dimensions on detail drawings shall be shown with dimensional accuracy to the nearest 1/16 in. (1.6 mm).

Welded connections and built-up components may require seal welds. Closed sections should be detailed with vent or drain holes if they are to be galvanized. Caution should be used to avoid explosive effects, which can injure workers or damage the component during the galvanizing process.

7.2 FABRICATION

7.2.1 Material

Since various steels are used in transmission towers, a quality control program, as specified in this Section, is necessary. A36 steel is considered the basic steel. All other steels shall have a special marking starting at the mill, be inventoried separately at the fabrication plant, and be properly identified during the fabricating process. Mill test reports shall be considered sufficient as certification of material, unless the purchaser's specification calls for other requirements.

7.2.2 Specifications

Fabrication shall be performed according to the purchaser's specification. If this specification does not cover fabrication procedures, the latest edition of the AISC Specification or a specification applicable to transmission towers shall be used. These documents provide a description of acceptable fabrication methods and procedures.

7.2.3 Shop Operations

Shop operations consist essentially of cutting (sawing, shearing, or flame cutting), punching, drilling, blocking or clipping, and either cold or hot bending. Hot bending will require steel to be heated to 1400-1600°F (760-871°C) if the steel is not produced to fine grain practice; see *The Making, Shaping and Treating of Steel* (1985).

Cold bending is normally done on pieces with simple bends at small bevels. Hot bending is necessary on pieces with moderate bevels and/or compound bends; heating should be done evenly and should be of sufficient length and temperature to minimize necking down of the section at the bend line. Pieces requiring bends at several bevels may have to be cut, formed, and welded. Specific preparation instructions and welding symbols must be shown on the shop detail drawings in this case.

The actual position of any punched or drilled hole on a member shall not vary more than 1/32 in. (0.8 mm) from the position for that hole shown on the shop detail drawing.

The purchaser should review fabricators' quality control procedures and agree on methods before fabrication begins. If there is disagreement, this should be settled in writing prior to fabrication.

7.2.4 Piece Marks

Each tower member shall have a number conforming to the piece mark on the erection drawings stamped with a metal die. For galvanized material these marks shall be stamped prior to galvanizing. Marks shall be minimum of 1/2 in. (12.7 mm) high. For special pieces, such as anchor bolts, where die stamping is not feasible, an indelible ink marking or special tagging which is durable and waterproof may be used. Some purchasers require that higher strength steel members include a suffix, such as "HJ," on the piece mark.

7.2.5 Welding

Welding procedures shall comply with ANSI/-AWS D1.1. Special care shall be taken regarding seal welds to assure proper galvanizing and to avoid acid "bleeding" at pockets in structural assemblies.

7.2.6 Galvanizing

Galvanizing shall be in accordance with ASTM A-123 and A-153. Procedures to avoid material embrittlement are given in ASTM A-143.

7.2.7 Shipping

The purchaser's specification shall clearly state the packing, bundling methods, and shipping procedures required.

8.0 TESTING

8.1 INTRODUCTION

The purchaser shall specify in the contract documents which structures or components of structures will be tested. If a proof test of a structure or a component of a structure is specified, the test shall be performed on a full size prototype of the structure or component in accordance with the following sections.

8.2 FOUNDATIONS

Tests shall be performed with the prototype attached to reaction points that have the same strengths and freedoms of movement as the reaction points that will be present in the structure in service. The EOR shall specify the anchorage requirements, including acceptable tolerances, in the contract documents.

8.3 MATERIAL

The prototype shall be made of material that is representative of the material that will be used in the production run. Mill test reports or coupon tests shall be available for all important members in the prototype including, as a minimum, the members designed for only tension loading, and compression members with KL/r less than 120.

8.4 FABRICATION

Fabrication of the prototype shall be done in the same manner as for the production run.

8.5 STRAIN MEASUREMENTS

The purchaser shall specify if any special strain determination methods are required for the prototype being tested.

8.6 ASSEMBLY AND ERECTION

The method of assembly of the prototype shall be specified by the purchaser. If tight bolting of subassemblies is not permitted by the construction specifications, the prototype shall be assembled and erected with all bolts finger-tight only, and tightening to final torque shall be done after all members are in place. Pickup points that are designed into the structure shall be used during erection as part of the test procedure.

8.7 TEST LOADS

The design factored loads (see Section 4.2) shall be applied to the prototype in accordance with the load cases specified. The test specification shall state if the structure is to be tested to destruction. Wind-on-structure loads shall be applied as concentrated loads at selected points on the prototype. These loads shall be applied at panel points where stressed members intersect so the loads can be resisted by the main structural system. The magnitudes and points of application of all loads shall be designated by the responsible engineer and approved by the purchaser.

8.8 LOAD APPLICATION

Load lines shall be attached to the load points on the prototype in a manner that simulates the in-service application as closely as possible. The attachment hardware for the test shall have the same degrees of freedom as the in-service hardware.

8.9 LOADING PROCEDURE

The number and sequence of load cases tested shall be specified by the responsible engineer and approved by the purchaser.

Loads shall be applied to 50%, 75%, 90%, 95%, and 100% of the design factored loads. After each increment is applied, there shall be a "hold" to allow time for reading deflections and to permit the engineers observing the test to check for signs of structural distress. The 100% load for each load case shall be held for 5 minutes.

Loads shall be removed completely between load cases except for noncritical load cases where, with the responsible engineer's permission, the loads may be adjusted as required for the next load case. Unloading shall be controlled to avoid overstressing any members.

8.10 LOAD MEASUREMENT

All applied loads shall be measured at the point of attachment to the prototype. Loads shall be measured through a verifiable arrangement of strain devices or by predetermined dead weights. Load measuring devices shall be used in accordance with manufacturer's recommendations and calibrated prior to and after the conclusion of testing.

8.11 DEFLECTIONS

Structure deflections under load shall be measured and recorded as specified by the responsible engineer. Deflection readings shall be made for the before- and off-load conditions, as well as at all intermediate holds during loading.

All deflections shall be referenced to common base readings taken before the first test loads are applied.

8.12 FAILURES

When a premature structural failure occurs, the cause of the failure, the corrective measures to be taken, and the need for a retest shall be determined by the EOR and approved by the purchaser.

If a retest is ordered, failed members and members affected by consequential damage shall be replaced. The load case that caused the failure shall be repeated. Load cases previously completed need not be repeated.

After completion of testing, the prototype shall be dismantled and all members inspected. The following shall not be considered as failures:

(a) Residual bowing of members designed for only tension;
(b) Ovalization of no more than one-half the holes in a connection; and
(c) Slight deformation of no more than one-half the bolts in a connection.

8.13 DISPOSITION OF PROTOTYPE

The test specification shall state what use may be made of the prototype after the test is completed.

8.14 REPORT

The testing organization shall furnish the number of copies required by the job specifications of a test report that shall include:

(a) The designation and description of the prototype tested;
(b) The name of the purchaser;
(c) The name of the person or organization (responsible engineer) that specified the loading, electrical clearances, technical requirements, and general arrangement of the prototype;
(d) The name of the Engineer of Record;
(e) The name of the fabricator;
(f) A brief description and the location of the test frame;
(g) The names and affiliations of the test witnesses;
(h) The dates of each test load case;
(i) Design and detail drawings of the prototype, including any changes made during the testing program;

(j) A rigging diagram with details of the points of attachment to the prototype;
(k) Calibration records of the load-measuring devices;
(l) A loading diagram for each load case tested;
(m) A tabulation of deflections for each load case tested;
(n) In case of failure:
Photographs of failure;
Loads at the time of failure;
A brief description of the failure;
The remedial actions taken;
The dimensions of the failed members; and
Test coupon reports of failed members;
(o) Photographs of the overall testing arrangement and rigging;
(p) Air temperature, wind speed and direction, any precipitation, and any other pertinent meteorological data;
(q) Mill test reports as submitted according to the requirements of Section 8.3; and
(r) Additional information specified by the purchaser.

9.0 STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

9.1 INTRODUCTION

This Section specifies design procedures for steel members and connections embedded in concrete foundations or the earth. Additional requirements for structural members and connections of grillages, pressed plates, anchor bolts, and stub angles are covered in Sections 5 and 6. Fig. 9.1 illustrates some typical foundations.

9.2 GENERAL CONSIDERATIONS

9.2.1 Steel Grillages

The members forming the pyramid, Fig. 9.1(a), shall be designed considering no lateral support from the surrounding soil. The stub angle, or leg member, Fig. 9.1(b), shall be designed considering support only at the shear plate and the base of the grillage.

9.2.2 Pressed Plates

The stub angle, or leg member, Fig. 9.1(c), shall be designed considering support only at the pressed plate base and the shear plate.

9.2.3 Stub Angles

The tensile and compressive loads in the stub angle, Fig. 9.1(d), shall be transferred to the concrete by the bottom plate or the shear connectors shown in Fig. 9.2. The shear load shall be transferred to the concrete by side bearing pressure.

9.2.4 Anchor Bolts, Fig. 9.1(e)

9.2.4.1 Smooth Bars with Base Assembly in Contact with Concrete or Grout

The anchor bolt shall be designed to transfer the tensile load to the concrete by the end connec-

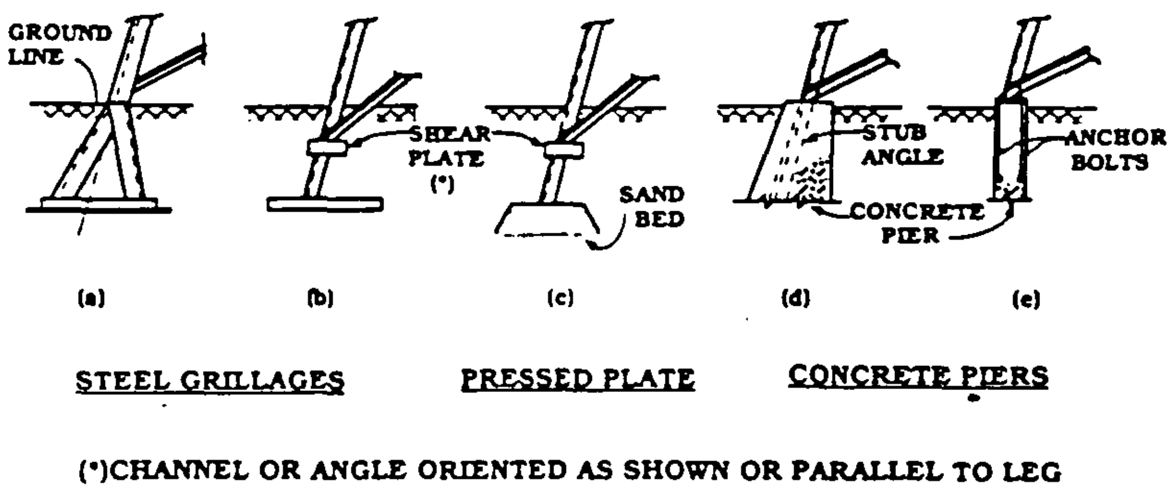


FIG. 9.1. Typical Foundations

tion. The compressive load shall be transferred to the concrete or grout by the base assembly. The shear load is assumed to be transferred to the concrete by shear friction based upon the clamping force on the base assembly.

9.2.4.2 Deformed Bars with Base Assembly in Contact with Concrete or Grout

The anchor bolt shall be designed with sufficient embedded length to transfer the tensile load to the concrete by bond between the bolt and the concrete. If the anchor bolt lacks sufficient embedment length, the tensile load shall be transferred to the concrete by the end connection. The compressive load shall be transferred to the concrete or grout by the base assembly. The shear load is assumed to be transferred to the concrete by shear friction based upon the clamping force on the base assembly.

9.2.4.3 Smooth or Deformed Bars with Base Assembly Not in Contact with Concrete or Grout

If the base assembly is permanently supported on anchor bolt leveling nuts, the transfer of the tensile or compressive load to the concrete shall conform to the following:

(a) For smooth bars by the end connection: and
(b) For deformed bars by bond between the concrete and the bar; if sufficient embedment length is not provided the end connection shall take the entire load.

The shear load shall be transferred to the concrete by side bearing pressure. The anchor bolt shall be checked for a combination of tension, bending, and shear, as well as compression, bending, and shear. If the clearance between the base assembly and the concrete does not exceed twice the bolt diameter, a bending stress analysis of the anchor bolt is not normally required.

9.3 DETERIORATION CONSIDERATIONS

Steel that is galvanized, or otherwise protected, shall have a minimum thickness of 3/16 in. (4.8 mm) when exposed to corrosion at the ground level or below.

9.4 DESIGN OF STUB ANGLES AND ANCHOR BOLTS

9.4.1 Stub Angles in Concrete

The stub angle, at the plane of intersection with the concrete, shall be checked for a combination of tension plus shear and compression plus shear, as follows:

$$A_g = \frac{P}{F_y} + \frac{V}{0.75F_y} \qquad (9.4\text{-}1)$$

where:

A_g = gross area of stub angle, or net area, if there is a hole at the intersecting plane;
P = tensile or compressive load on the stub angle;
V = shear load parallel to the intersection plane; and
F_y = specified minimum yield strength of stub angle.

9.4.2 Anchor Bolts with Base Assembly in Contact with Concrete or Grout

When the anchor bolt bases are subjected to uplift and shear loads, the shear load shall be assumed to be transferred to the concrete by shear friction based upon the clamping force of the anchor bolts. The area of steel required shall be:

$$A_s = \frac{T}{F_y} + \frac{V}{(\mu)0.85F_y} \qquad (9.4\text{-}2)$$

The stress area through the threads is given by:

$$A_s = \frac{\pi}{4}\left(d - \frac{0.974}{n}\right)^2 \qquad (9.4\text{-}3)$$

where:

T = tensile load on anchor bolt;
V = shear load perpendicular to anchor bolts;
F_y = specified minimum yield strength of anchor bolt;
d = nominal diameter;
n = number of threads per unit of length; and
μ = coefficient of friction.

The values for μ (Fig. 9.3) are:

(a) 0.9 for concrete or grout against as-rolled steel with the contact plane a full plate thickness below the concrete surface;

)

(b) 0.7 for concrete or grout placed against as-rolled steel with contact plane coincidental with the contact surface; and
(c) 0.55 for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface.

When anchor bolt bases are subjected to a shear load or a combination of downthrust and shear loads, the anchor bolt area shall be checked by:

$$A_t = \frac{V - 0.3\ D}{(\mu)\ 0.85\ F_y} \qquad (9.4\text{–}4)$$

where:

D = downthrust load and the other terms as defined after Eq. 9.4-3.

When shear lugs are attached to the base assembly to transfer the shear to the concrete, the area of the anchor bolt need not be checked by Eqs. 9.4-2 and 9.4-4.

A combination of shear lugs and shear friction is not allowed.

9.5 DESIGN REQUIREMENTS FOR CONCRETE AND REINFORCING STEEL

The ultimate design stresses and strength factors of ACI 318 shall be used for the design of concrete and reinforcing steel in conjunction with the structure design factored loads specified in Section 4.

9.5.1 Stub Angles

When a bottom plate is used, Fig. 9.2(a), the plate shall transfer the entire load in the stub angle to the concrete; concrete anchorage value shall be determined by the requirements of Section 9.5.2.

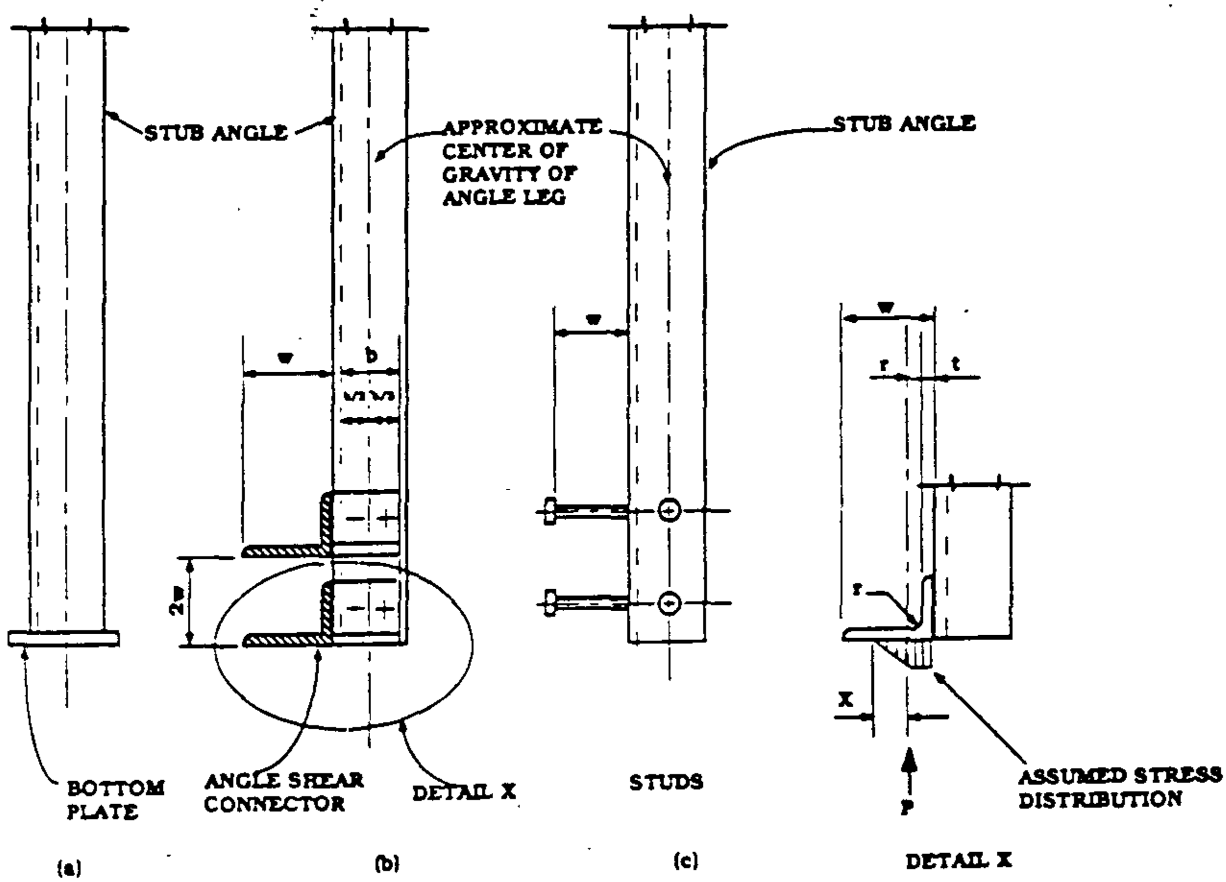


FIG. 9.2. Stub Angles

When shear connectors, Figs. 9.2(b) or 9.2(c), are used and spaced along the length of the stub angle, the requirements of Section 9.6 shall apply.

9.5.2 Smooth Bar Anchor Bolts

The anchorage value shall be limited by the pull-out strength of the concrete based on a uniform tensile stress, in ksi, of $0.126\,\phi\,\sqrt{f'_c}$ (in MPa, of $0.33\phi\,\sqrt{f'_c}$), acting on an effective stress area which is defined by the projecting area of stress cones radiating towards the surface from the bearing edge of the anchors.

The effective area shall be limited by overlapping stress cones, by the intersection of the cones with concrete surfaces, by the bearing area of anchor heads, and by the overall thickness of the concrete. The angle for calculating projected area shall be 45°. The factor shall be 0.65 for an embedded anchor head. When there is more than one anchor bolt in a line, overlapping stress cones shall be taken into account in determining the effective area.

The anchor head can be a nut, bolt head, or plate. The bearing requirements of ACI 318 need not be met if the anchor head satisfies the following conditions:

(a) The bearing area of the anchor head (excluding the area of the anchor bolt) is at least 1.5 times the area of the anchor bolt;
(b) The thickness of the anchor head is at least equal to the greatest dimension from the outermost bearing edge of the anchor head to the face of the anchor bolt; and
(c) The bearing area of the anchor head is approximately evenly distributed around the perimeter of the anchor bolt.

9.5.2.1 Minimum Embedment for Anchor Bolts

The minimum embedment depth shall be $12d\sqrt{F_u/58\Psi}$

where:

d = nominal diameter;
F_u = specified minimum tensile strength; and
Ψ = 1 for F_u in ksi and 5.89 for F_u *in MPa*.

9.5.3 Deformed Bar Anchor Bolts

The embedment for deformed bars that are threaded and used as anchor bolts shall be in accordance with ACI 318; see Sections 9.2.4.2 and 9.2.4.3. Bars Grade 60 and above shall have a minimum Charpy-V notch requirement of 15 ft.-lbs. (20 m-N) at −20°F (−29°C), when tested in the longitudinal direction.

9.6 SHEAR CONNECTORS

9.6.1 Stud Shear Connectors, Fig. 9.2c

The capacity, Q_n, of a stud shear connector shall be as given by Eq. 9.6-1, but not to exceed the value determined by Section 6.3.2.

$$Q_n = 0.5\,\phi\,A_{sc}\sqrt{f'_c\,E_c} \qquad (9.6\text{-}1)$$

where:

ϕ = 0.85;
A_{sc} = cross-sectional area of a stud shear connector;
f'_c = specified compressive strength of concrete; and
E_c = modulus of elasticity of concrete.

Values of Q_n from Eq. 9.6-1 are applicable only to concrete made with ASTM C33 aggregates. All AISC requirements (*Load and Resistance Factor Design Manual of Steel Construction* 1986) for stud material and configuration, spacing, ratio of stud diameter to minimum thickness of material to which it is welded, and concrete properties and coverage shall be met.

9.6.2 Angle Shear Connectors, Fig. 9.2b

The capacity, P, of angle shear connectors shall be determined by the following:

$$P = 1.19\,f'_c\,b\,(t + r + x/2) \qquad (9.6\text{-}2)$$

$$x = t\left[\frac{F_y}{1.19\,f'_c}\right]^{\frac{1}{2}} \le w - r - t$$

where:

f'_c = specified compressive strength of concrete;
b = length of angle shear connector;
t = thickness of angle shear connector;
r = radius of fillet;
F_y = specified minimum yield strength of steel; and
w = width of angle shear connector leg.

The angle shear connector shall be located with its length symmetrical about the center of gravity of the stub angle leg. The connector shall be fastened to the stub with sufficient bolts or welds to take both shear and moment. The minimum center-to-center spacing of shear connectors shall be $2w$.

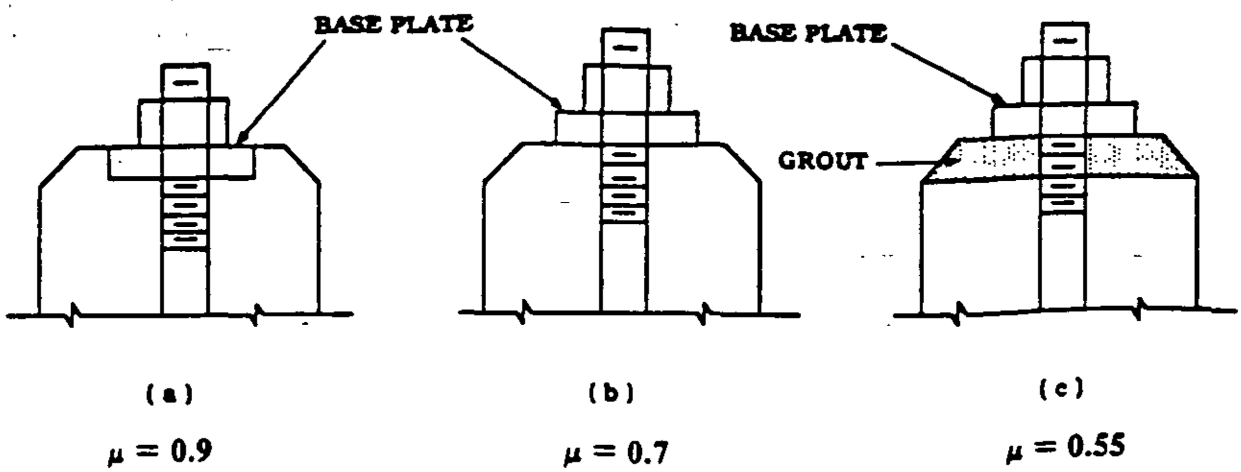


FIG. 9.3. Coefficient of Friction (μ) Values for Various Conditions

9.7 TEST VERIFICATION

Design values other than those prescribed in this Section may be used if substantiated by experimental or analytical investigations.

SECTION 10 QUALITY ASSURANCE/ QUALITY CONTROL

10.1 INTRODUCTION

The contract between the purchaser and the supplier shall state the responsibilities of each party and the conditions under which the work will be accepted or rejected.

10.2 QUALITY ASSURANCE

Quality Assurance (QA) is the responsibility of the purchaser. The purchaser's bid documents shall outline the QA methods, types of inspections and records that will be required to determine the acceptability of the product at each stage of the design, manufacturing, and construction process.

10.3 QUALITY CONTROL

Quality Control (QC) is the responsibility of the supplier. The supplier shall have a QC program consisting of a written document or a series of departmental memoranda that establishes the procedures and methods of operation which affect the quality of the work.

This Commentary is not a part of the Standard. It is included for informational purposes only. This information is provided as explanatory and supplementary material designed to assist in applying the recommended requirements.

The sections of this Commentary are numbered to correspond to the sections of the Standard to which they refer. Since it is not necessary to have supplementary material for every section in the Standard, there are gaps in the numbering sequence of the Commentary.

SECTION 4 LOADING, GEOMETRY, AND ANALYSIS

4C.1 INTRODUCTION

Design factored loads are loads multiplied by load factors. The overload capacity factors specified by the National Electric Safety Code (1990) are load factors according to the terminology of this Standard.

4C.2 LOADS

Extensive background information on the selection of design-factored loads can be found in *Guidelines for Transmission Line Structural Loading* (1991, presently under revision) the *National Electrical Safety Code* (1990), and the *Guide for Design of Steel Transmission Towers*, Second Edition (1988).

Minimum legislated loads are specified in applicable codes covered by state and local authorities. Local climatic conditions may dictate loads in excess of legislated loads. These may include conditions of wind, or ice, or any of their combination at a specified temperature. Line security, which covers measures to prevent progressive line failure (cascading), should be addressed. This is often handled by specifying one or more longitudinal load conditions. Loads from anticipated construction and maintenance operations should be specified to ensure the safety of the personnel involved in those operations. The *Guide for Design of Steel Transmission Towers*, Second Edition (1988), provides guidance on other special loading considerations.

It is suggested that tower members that may be used for support by maintenance personnel when climbing a tower be capable of supporting a vertical load of 250 pounds (1100 N) applied independently of all other loads without permanent distortion of the member. If end connection assembly bolts are properly tightened, the frictional restraining effect may be considered.

4C.3 GEOMETRIC CONFIGURATIONS

Three basic structure definitions are recommended: suspension, strain, and dead-end structures. The conductor phases pass through and are suspended from the insulator support points of a suspension structure. The strain structure conductor attachment points are made by attaching the conductor to a dead-end clamp, a compression or bolted fitting, and connecting the clamp, through the insulator string, directly to the structure. A jumper is looped through or around the structure body to electrically connect the adjacent spans. Dead-end structure conductor attachments are made the same way as for the strain structure.

Dead-end structures often have different tensions or conductor sizes on opposite sides of the structure; this creates an intact unbalanced longitudinal load. Overhead groundwires are attached to the structure using similar methods as outlined for the conductors. Additional nomenclature for the basic structure types is used to help identify the line angle at a particular structure.

The term "tangent" is prefixed to a basic structure type for zero line angle and the term "angle" is used when there is a line angle. Therefore, the following terminology is recommended: tangent suspension, angle suspension, tangent strain, angle strain, tangent dead end, and angle dead end.

Guyed structures rely on internal or external guy cables for their stability. They are normally less rigid than self-supporting structures and their deflections may affect electrical clearances. Some typical self-supporting and guyed tower configurations are shown in the *Guide for Design of Steel Transmission Towers*, Second Edition (1988).

In most structures, horizontal bracing is required to distribute shear and torsional forces. It is normally used at levels where there is a change in the slope of the structure leg. Horizontal bracing is also used in square and rectangular configuration structures to support horizontal struts and to provide a stiffer system to assist in reducing distortion caused by torsional and/or oblique wind loads.

For structures which are taller than 200 ft. (61 m), or heavy dead-end towers, it is suggested that horizontal bracing be installed at intervals not exceeding 75 ft. (23 m). The spacing of horizontal bracing is dictated by general stiffness requirements to maintain tower geometry and face alignment. Factors which affect this determination are: type of bracing system, face slope, dead load sag of the face members, and erection considerations that affect splice locations and member lengths.

4C.4 METHODS OF ANALYSIS

A latticed structure is described by a one-line design drawing which shows overall dimensions, member sizes, and locations. Because of the high degree of symmetry of most latticed structures, a transverse view, a longitudinal view, and a few horizontal cross-section or plan views are sufficient to describe the entire structure (Fig. 4C.1). For purposes of analysis, a latticed structure is represented by a model composed of members (and sometimes cables) interconnected at joints. Members are normally classified as primary and redundant members. Primary members form the triangulated system (3-dimensional truss) that carries the loads from their application points to the foundation. Redundant members are used to provide intermediate bracing points to the primary members to reduce the unbraced lengths of these primary members. They can easily be identified on a drawing (see dotted lines in Fig. 4C.1) as members inside triangles formed by primary members.

The locations of the joints in any model should be at the intersections of the centroidal axes of the members. Slight deviations from these locations will not significantly affect the distribution of forces.

Latticed structures are analyzed almost exclusively as ideal elastic 3-dimensional trusses made up of straight members or cables and pin connected at joints. Such elastic analyses produce only joint displacements, tension, and compression in members and tension in cables: moments from normal framing eccentricities are not calculated in the analysis. Moments in members because of framing eccentricities, eccentric loads, distributed wind load on members, etc., can affect the member selection. These moments are considered in the member selection by the procedures described in Section 5.

First-order linear elastic truss analysis treats all members as linearly elastic (capable of carrying compression as well as tension), and assumes that the loaded configuration of the structure (used to verify final equilibrium) is identical to its unloaded configuration. Therefore, in a first-order linear analysis, the secondary effects of the deflected structure are ignored and the forces in the redundant members are equal to zero. The redundant members need not be included in this type of analysis, since they have no effect on the forces in the load-carrying members. This type of analysis is generally used for conventional, relatively rigid, self-supporting structures.

In a second-order (geometrically nonlinear) elastic analysis, structure displacements under loads create member forces in addition to those obtained in a first-order analysis. These additional member forces are called the PΔ effects in building frameworks or transmission pole structures. They are automatically included in a second-order (or geometrically nonlinear) analysis that produces member forces which are in equilibrium in the deformed structure configuration (Peyrot 1985; ETADS 1989). A second-order elastic analysis may show that redundant members carry some load. Flexible self-supporting structures and guyed structures normally require a second-order analysis.

For purposes of analysis, it is sometimes assumed that bracing members with an L/r value greater than 300 cannot resist compressive loads. Such members are called tension-only members (see Section 7.1.8). Fig. 4C.2 shows the difference in load distribution between a tension-compression system and a tension-only system.

Other methods of analysis, which account for the load distribution in bracing members, may be used. There is some question as to the load that can be carried by a bracing member strained in compression beyond the value e_{max} that corresponds to its theoretical capacity F_{max}, Fig. 4C.3(a). Three different analysis methods have been used to model

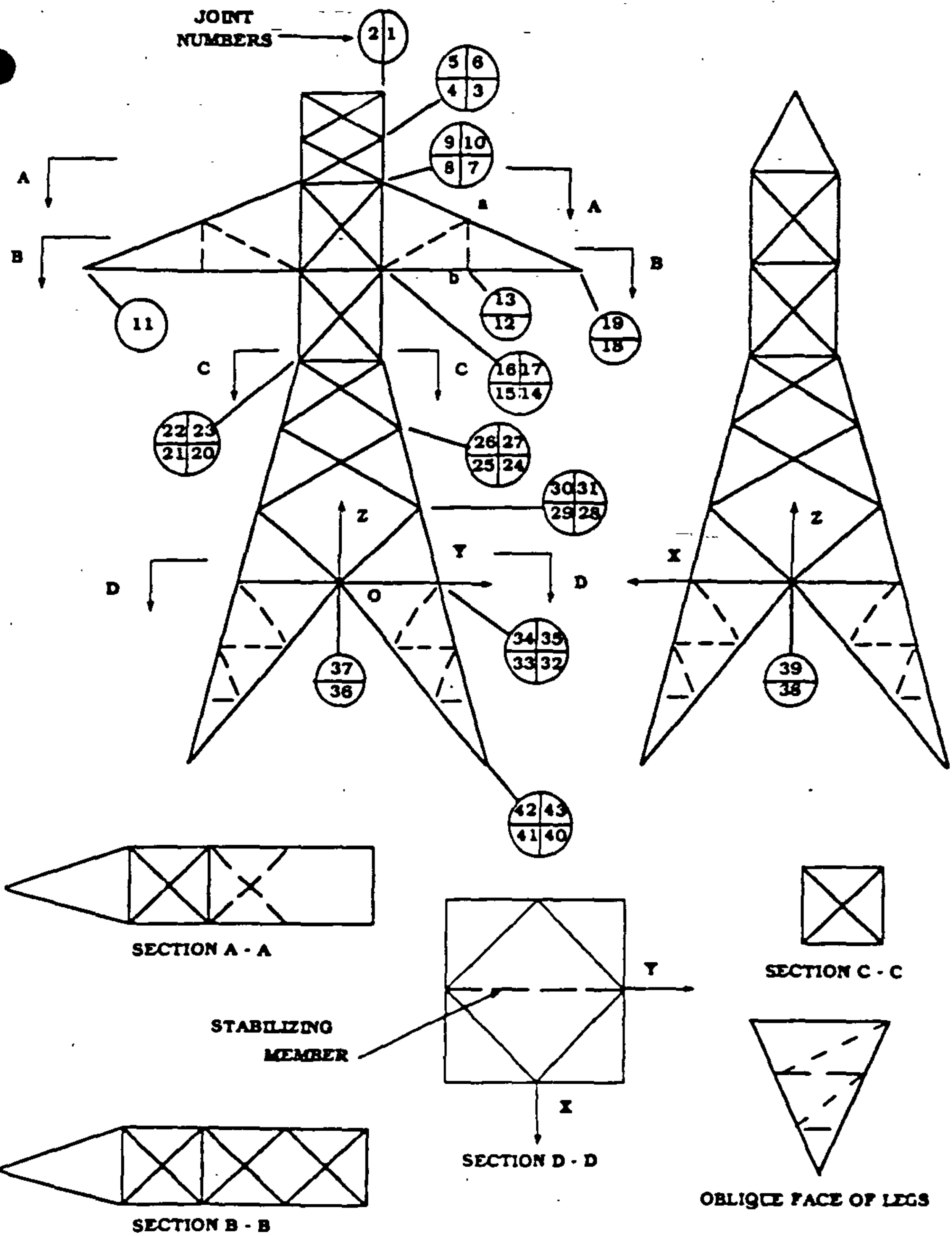
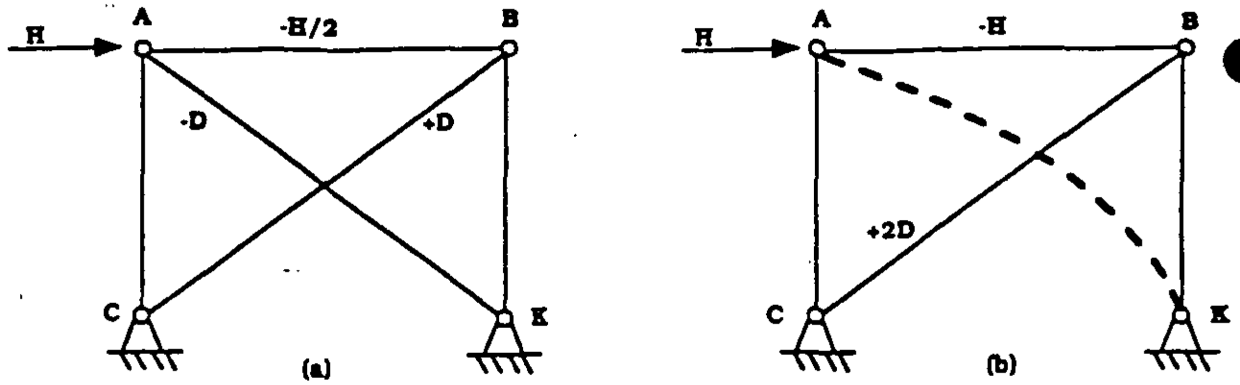


FIG. 4C.1. Model of Simplified Tower



(-) INDICATES COMPRESSION

FIG. 4C.2. (*a*) Tension-Compression System; (*b*) Tension-Only System

post-buckling member behavior. Method 1 assumes that the member is still capable of carrying its buckling load, irrespective of the amount of strain beyond e_{max}, Fig. 4C.3(*b*). Method 2 assumes that the member carries no load after passing e_{max}, Fig. 4C.3(*c*). Method 3 assumes that the compression member can carry a reduced load beyond e_{max} (Prickett 1989); the member load is modeled using a post-buckling performance curve, Fig. 4C.3(*a*).

When performing a computer analysis of an existing structure, careful attention must be given to the method of analysis employed when the structure was originally designed (Kravitz 1982). If the structure was originally designed by manual (algebraic or graphical) methods and the design loads are not changed, a 3-dimensional computer analysis may indicate forces in the members which are different from those from the manual methods. The engineer should determine and document why the differences exist before proceeding with the new analysis. If the tower is to be upgraded and new design factored loads specified, then it is nor-

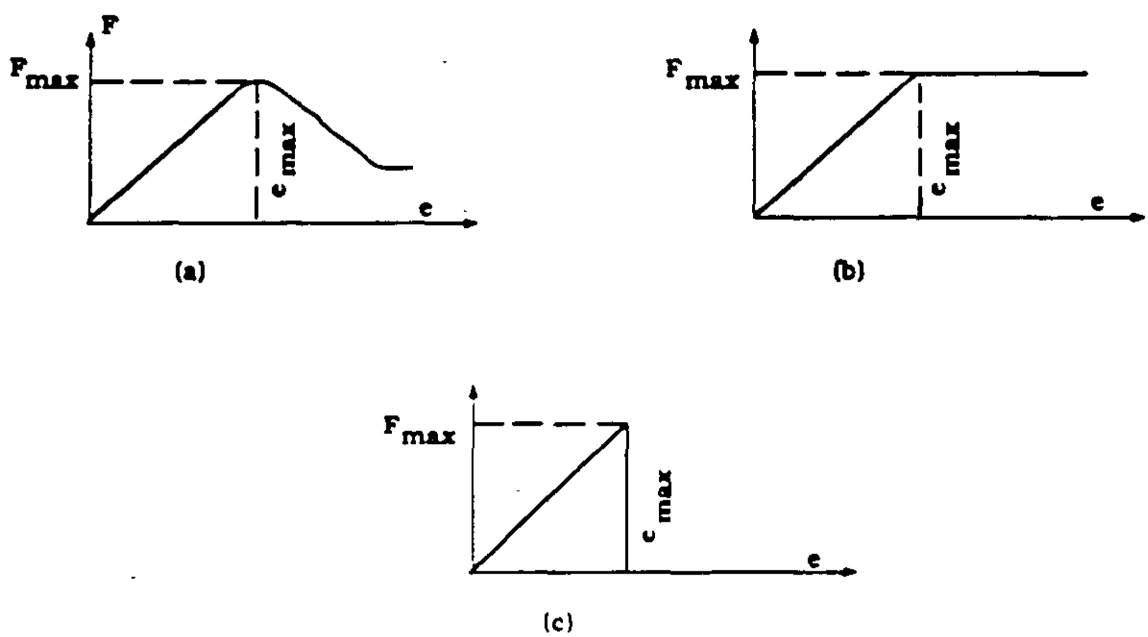


FIG. 4C 3. Relationships Between Member Compression Force and Shortening: (*a*) Actual Member; (*b*) Liberal Assumption; (*c*) Conservative Assumption

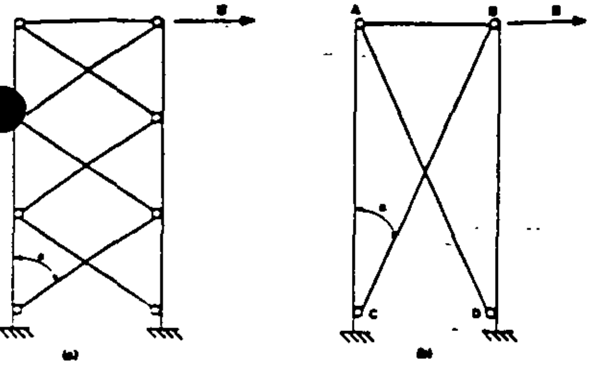


FIG. 4C.4. Braced Systems Under Shear Load: (*a*) Stiff Bracing; (*b*) Insufficiently Stiff Bracing

mally more cost-effective to rely on a computer analysis. A correlation of past model assumptions with present model assumptions should be performed for the entire structure. Detail drawings should be reviewed to ensure that members and connections are in agreement with the original design drawings.

If the included angle between a bracing member and the member it supports is small, the bracing member should not be considered as providing full support; as the included angle approaches 15°, the supported member should be investigated for stability.

Moments can occur in a leg or cross-arm chord if the bracing is insufficiently stiff (Roy et al. 1984). Fig. 4C.4(*a*) represents qualitatively a situation where the bracing is sufficiently stiff to carry all the shear load *H*. Fig. 4C.4(*b*) represents a case where the bracing system, because of the small value of the angle and small diagonal member sizes, is insufficiently stiff; therefore, the diagonals carry only a portion of the shear; the remainder of the shear produces moments in the vertical members AC and BD. If moments are anticipated in leg members, it is prudent to use an analysis method that models such members as beams. Other members in the structure can still be modeled by truss elements.

Dynamic analysis of latticed structures can be performed with a general-purpose finite element computer program. However, there is no indication that such an analysis is needed for design purposes, even in earthquake-prone areas (Long 1974).

Specialized computer programs for the analysis of latticed transmission structures (Rossow et al. 1975; ETADS 1989; Tower 1987; TOWER 1988; etc.) should include the following features: automatic generation of nodes and members that utilize

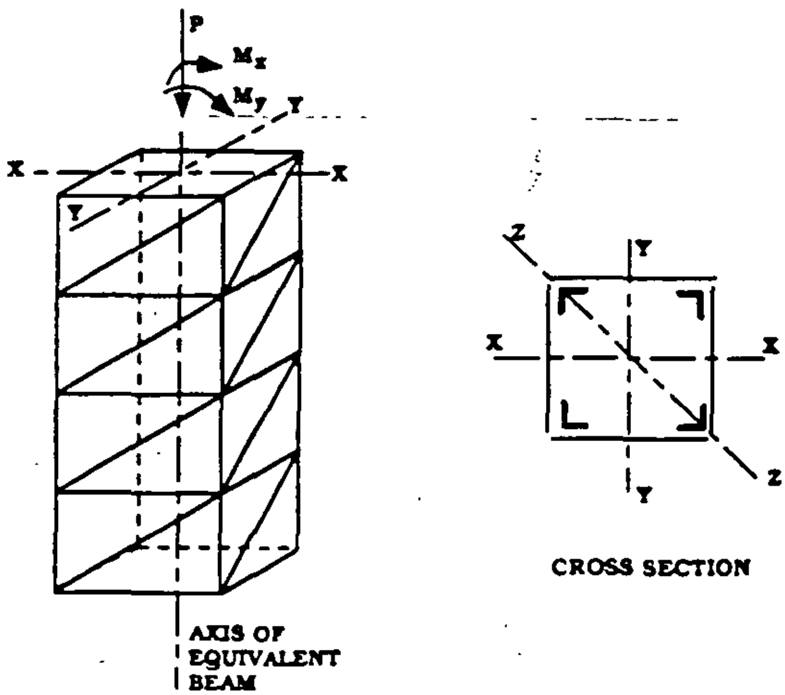


FIG. 4C.5. Segment of Latticed Mast Idealized as Beam linear interpolations and symmetries, interactive graphics to ascertain model correctness, provisions for tension-only members and for automatic handling of planar nodes and mechanisms (unstable subassemblies) which may develop in a small group of nodes and members. Out-of-plane instabilities or mechanisms are generally prevented in actual structures by the bending stiffness of continuous members that pass through the joints.

Joints 3, 4, 5, and 6 of Fig. 4C.1 are planar nodes, i.e., all members meeting at those points lie in the same plane. Joints 12 and 13 are also planar joints if the redundant member "ab" is not included in the model. The horizontal bracing in section D-D is a mechanism in the absence of the member shown as a dotted line.

Guyed structures and latticed H-frames may include masts built-up with angles at the corners and lacing in the faces as shown in Fig. 4C.5. The overall cross section of the mast is either square, rectangular, or triangular. Latticed masts typically include a very large number of members and are relatively slender, i.e., may be susceptible to second-order stresses. One alternative to modeling a mast as a 3-dimensional truss system is to represent it by a model made up of one or several equivalent beams. The properties of an equivalent beam that deflects under shear and moment can be worked out from structural analysis principles. The beams are connected to form a 3-dimensional model of the mast or an entire structure. That model may be analyzed with any 3-dimensional finite element computer program. If large deflections are expected, a second order (geometrically nonlinear) analysis should be used. Once the axial loads, shears, and moments are determined in each equivalent beam, they can be converted into axial loads in the members that make up the masts.

SECTION 5 DESIGN OF MEMBERS

5C.1 INTRODUCTION

This standard is suitable for steels with yield points up to 65 ksi (448 MPa) and for width-to-thickness values of 25 for projecting elements, such as angle legs and channel flanges. The recommendations are intended for both hot-rolled and cold-formed members. Recommendations have also been included covering guyed transmission structures.

Test experience may indicate that these recommendations are conservative for specific shapes or connections. Higher values may be used where they are verified by tests, provided the results are adjusted to the ASTM yield and tensile values of the material and for differences between the nominal design dimensions of the member and the actual cross-sectional dimensions of the test specimens. The properties of the material should be determined by tests on standard coupons taken in accordance with the requirements of the AISI Specification (1986).

The Guide for Design of Steel Transmission Towers, Second Edition (1988) provides supplementary background material and illustrative examples of the design recommendations outlined in this Standard.

5C.2 MATERIAL

A ratio of $F_u/F_y \geq 1.15$ is suggested for steel used for members.

5C.3 MINIMUM SIZES

If weathering steel is to be used, consideration should be given to increasing the suggested minimum thickness.

5C.4 SLENDERNESS RATIOS

Damaging vibration of steel members in latticed towers usually occurs at wind speeds less than 20 mph (32 km/h) since a nearly constant velocity is required to sustain damaging vibration. Tests on a number of shapes with L/r values of 250 show that the possibility of damaging vibration is minimal (Carpena and Diana 1971; Cassarico et al. 1983). Tension-hanger members are prone to vibration, but L/r values as large as 375 have been used successfully.

In areas of steady winds over extended periods, such as mountain passes or flat plains, allowable L/r values may need to be reduced. Where severe vibration is a concern, careful attention must be given to framing details. The practice of blocking the outstanding leg of angles to facilitate the connection should be avoided.

5C.5 PROPERTIES OF SECTIONS

Evaluation of torsional-flexural buckling involves some properties of the cross section which are not encountered in flexural buckling. Procedures for computing the torsional constant J, the warping constant C_w, the shear center, and other properties are given in *Cold-Formed Steel Design Manual* (1987), Timoshenko and Gere (1961), Yu (1986), and other sources. The AISC Manuals of Steel Construction (1986, 1989) give tabular values for various standard shapes.

For cold-formed shapes with small inside-bend radii (twice the thickness), section properties can be determined on the basis of square corners. Equations based on round corners are given in the *Cold-Formed Steel Design Manual* (1987). Normally, the differences in properties based on square and round corners are not significant.

5C.6 ALLOWABLE COMPRESSION

The Structural Stability Research Council (SSRC) formula for the ultimate strength of the centrally loaded column in the inelastic range and the Euler formula in the elastic range are used in this Standard. Test experience on tower members is limited in the range of L/r fropm 0 to 50, but indications are that the SSRC formula applies equally well in this range if concentric framing details are used.

5C.7 COMPRESSION MEMBERS: ANGLES

A single angle in axial compression can fail by torsional-flexural buckling, which is a combination

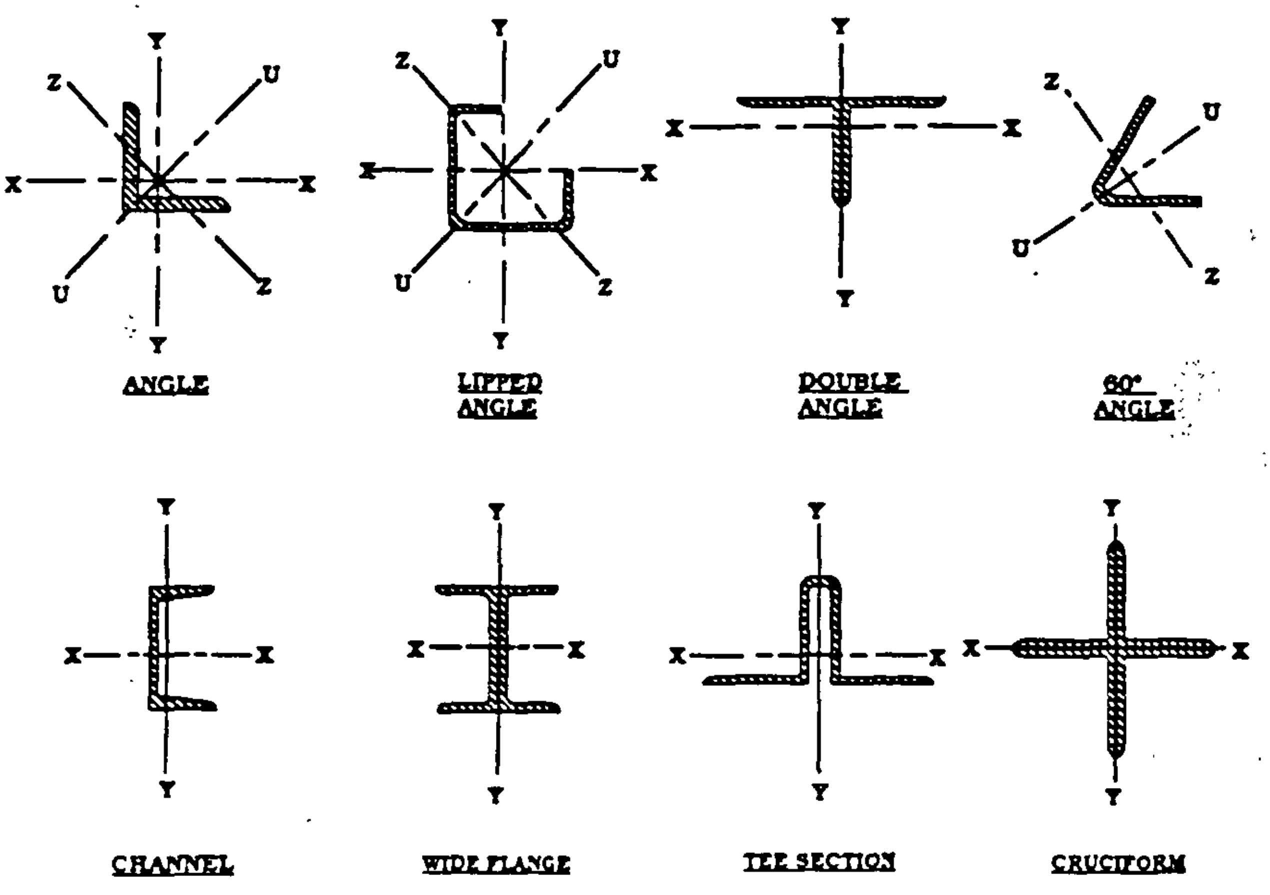


FIG. 5C.1. Typical Cross Sections

of torsional buckling and flexural buckling about the u-axis; by z-axis flexural buckling; or by local buckling of the legs. Local buckling and purely torsional buckling are identical if the angle has equal legs and is simply supported and free to warp at each end; furthermore, the critical stress for torsional-flexural buckling is only slightly smaller than the critical stress for purely torsional buckling, and for this reason such members have been customarily checked only for flexural and local buckling.

5C.7.3 Determination of F_a

The ratio w/t of flat width to thicknesses which enables the leg to reach yield stress without buckling locally has been set at $80/\sqrt{F_y}$ for F_y in ksi ($210/\sqrt{F_y}$ for F_y in MPa). The reduced strength of legs with larger values of w/t is given by Eqs. 5.7-2 and 5.7-3. The effect of the reduced local-buckling strength on the flexural-buckling strength is accounted for by substituting the reduced value F_{cr} for F_y in Eqs. 5.6-1 and 5.6-3. Unequal-leg angles can be designed following this procedure by establishing the allowable stress based on the w/t value of the long leg. Member strengths computed by this procedure are in very good agreement with test results on both hot-rolled and cold-formed single angle members (Gaylord and Wilhoite 1985).

5C.7.4 Effective Lengths

The K factors for angle members depend upon the connection design for the member. The effective length of leg sections having bolted connections in both legs is assumed to be the actual length ($K = 1$). For other angles in compression, eccentricity of the connection is the predominant factor in the lower L/r range and is accounted for by specifying effective slenderness values KL/r. In the higher L/r range, rotational restraint of the members becomes the predominant factor and is also accounted for by specifying effective slenderness values KL/r. The break point is taken as $L/r = 120$. Background for these recommendations is shown in *Guide for Design of Steel Transmission Towers*, Second Edition (1988). Member strengths computed by this procedure are in very close agreement with numerous tests on both hot-rolled and cold-formed angles (Gaylord and Wilhoite 1985). Eccentricities at leg splices should be minimized, and the thicker sections should be properly butt-spliced.

To justify using the values of KL/r specified in Eqs. 5.7-9, 5.7-10, 5.7-13, and 5.7-14 the following evaluation is suggested:

(a) The restrained member must be connected to the restraining member with at least two bolts; and
(b) The restraining member must have a stiffness factor I/L in the stress plane (I = moment of inertia and L = length) that equals or exceeds the sum of the stiffness factors in the stress plane of the restrained members that are connected to it. An example is shown in Fig. 5C.2.

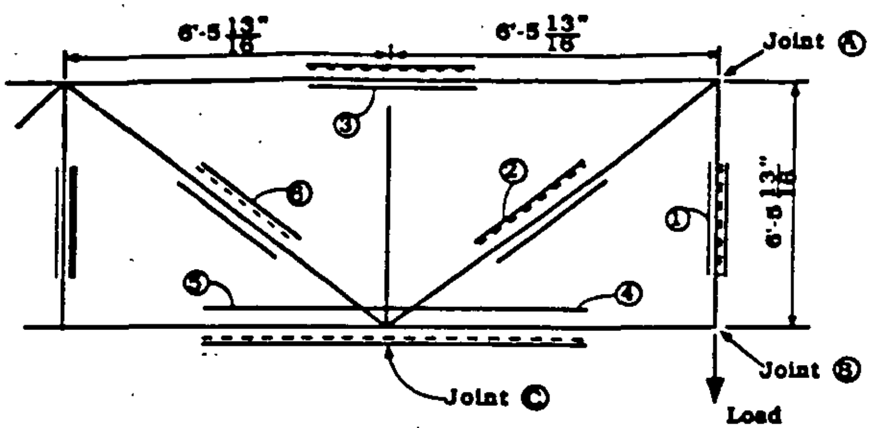
Angle members connected by one leg should have the centroid of the bolt pattern located as close to the centroid of the angle as practicable. Except for some of the smaller angles, normal framing eccentricity implies that the centroid of the bolt pattern is located between the heel of the angle and the centerline of the connected leg. When this is not the case, due consideration should be given to the additional stresses induced in the member.

Redundant members provide intermediate support for stressed members. The L/r limitations specified in Section 5.7.4.3 normally ensure that the redundant is adequate to provide support for the stressed member. Studies indicate that the magnitude of the redundant support required is dependent on the initial crookedness and the L/r value of the supported member. The magnitude of the load in the redundant member can vary from 0.5 to 2.5% of the load in the supported member.

5C.8 COMPRESSION MEMBERS: SYMMETRICAL LIPPED ANGLES

Lips increase the local buckling strength of the legs of an angle and in some applications lipped angles show an advantage over plain angles. Since the local buckling strength of the angle is not equivalent to torsional buckling of the angle, torsional-flexural buckling must be considered. The allowable compressive stress for torsional-flexural buckling is determined by using an equivalent radius of gyration r_q (Eq. 5.8-1) in the allowable column stress formulas (Eqs. 5.6-1 and 5.6-2).

The effective-length coefficient in Eqs. 5.6-1, 5.6-2, and 5.8-2 is $K = 1$ if the member is free to warp and to rotate about the u axis at each end. If



$I = r^2A$ and $I/L = (rA)/(L/r)$

(1) $L2 \times 2 \times 1/8$: one bolt connection—joint restraint not a consideration—tension member.

(2) and (6) $L2\frac{1}{2} \times 2\frac{1}{2} \times 3/16$—two bolt connection—$L = 110$ in.; $r_u = 0.778$; $A = 0.902$ sq. in.

$$\frac{rA}{L/r} = \frac{0.778 \times 0.902}{110/0.778} = 0.005 \text{ (in truss plane).}$$

(3) $L2\frac{1}{2} \times 2\frac{1}{2} \times 3/16$; $L = 155.6$ in.; $r_u = 0.778$; $A = 0.902$ sq. in.

$$\frac{rA}{L/r} = \frac{0.778 \times 0.902}{155.6/0.778} = 0.004 \text{ (in truss plane).}$$

Joint restraint not a consideration—tension member.

(4) and (5) $L4 \times 4 \times \frac{1}{4}$; $L = 77.8$ in.; $r_u = 1.25$; $A = 1.94$ sq. in.

$$\frac{rA}{L/r} = \frac{1.25 \times 1.94}{77.8/1.25} = 0.039 \text{ (in truss plane).}$$

Joint (A) (3) < (2) 0.004 < 0.005 No restraint for (2).
Joint (B) Single bolt connection—no restraint.
Joint (C) (4) + (5) > (2) + (6) 2 × 0.039 > 2x 0.005—Partial restraint for (2) and (6) at this joint.

Member (2)—Partial restraint at one end: $r_z = 0.495$; (Eq. 5.7-9)
$L/r_z = 110/0.495 = 222.2$; $KL/r_z = 28.6 + 0.762 \times 222.2 = 198$;
Member meets requirements of compression member.

FIG. 5C.2. Member Restraint Determination (All dimensions in inches—1 in. = 25.4 mm)

warping and u axis rotation are prevented at both ends, $K = 0.5$; if they are prevented at only one end, $K = 0.7$. Mixed end conditions can be treated by replacing r_t and r_u in Eq. 5.8-1 with r_t/K_t and r_u/K_u where K_t and K_u are the effective-length coefficients for torsional and u-axis buckling, respectively. Eq. 5.8-1 in this form gives the value of K/r_{tf} by which L is multiplied for use in Eqs. 5.6-1 and 5.6-2. However, $K_t = 1$ should be used in Eq. 5.8-2 when it is used to compute the adjusted values KL/r specified in Section 5.7.4. Gaylord and Wilhoite (1985) and Zavelani (1984) provide additional test verifications.

If there are no intermediate supports, the allowable stress is given by Eqs. 5.6-1 and 5.6-2, using for KL/r the larger of KL/r_z and KL/r_{tf}. If there are intermediate supports, the length L used to determine the slenderness ratio depends on the nature of the support, i.e., whether it restrains only flexural buckling, only torsional-flexural buckling, or both.

5C.9 COMPRESSION MEMBERS NOT COVERED IN SECTIONS 5.7 AND 5.8

5C.9.2 Maximum w/t Ratios

Most of the shapes other than angles that are likely to be used in transmission towers will have element slenderness ratios, w/t, small enough to develop a uniform distribution of the stress F_a given by Eqs. 5.6-1 and 5.6-2 over the full cross-sectional area. Where this is not the case, the post-buckling strength of elements which buckle prematurely is taken into account by using an effective width of the element in determining the area of the member cross section. The effective width of an element is the width which gives the same resultant force at a uniformly distributed stress F_a as the nonuniform stress which develops in the entire element in the post-buckled state.

5C.9.3 Effective Widths of Elements in Compression

Effective widths in this section are derived from formulas in *Specification for the Design of Cold-Formed Steel Structural Members* (1986). Only the effective widths of Section 5.9.3.1 are needed for the uniform stress distribution in axially loaded compression members.

Stress gradients (Section 5.9.3.2) occur in members in bending, and effective widths for this case are needed only for beams and eccentrically loaded compression members.

Note that the effective widths for axially loaded members are determined at the allowable stress F_a based on the radius of gyration of the gross cross section, while the allowable force P is obtained by multiplying F_a by the gross area if all elements are fully effective and by the reduced area if the effective widths of the elements are smaller than the actual widths.

The types of buckling that must be checked for axially loaded members with symmetric cross sections are covered in Sections 5.9.4-5.9.7. For members which may be subject to torsional or torsional-flexural buckling, an equivalent radius of gyration, r_t, for doubly symmetric and point symmetric sections and r_{tf} for singly symmetric sections are specified to be used in determining the allowable stress F_a by Eqs. 5.6-1 and 5.6-2. Note that r_{tf} and r_t (Eqs. 5.8-1 and 5.8-2) are referred to the principal axes u and z of angles (Fig. 5C.1). In adjusting the formulas for the x and y principal axes of other sections, u is the axis of symmetry. Therefore, when either the x-axis or the y-axis is the axis of symmetry, it must be substituted for u in Eq. 5.8-1, and also wherever it appears in the list of symbols following Eq. 5.8-2.

5C.9.8 Nonsymmetric Cross Sections

An analysis based on the elastic buckling stress of a nonsymmetric member, which requires the solution of a cubic equation, is suggested in Zavelani and Faggiano (1985). In general, this will give an upper bound to the allowable value. A lower bound can be obtained by proportioning the member so that the maximum combined stress due to the axial load and moment equals the yield stress (Madugula and Ray 1984).

5C.10 TENSION MEMBERS

5C.10.5 Guys

The determination of the tension of guys must be based on the movement of the guy anchor under load, the length and size of the guy, the allowable deflection of the structure, and the modulus of the guy. On tangent structures, pretensioning of guys to 10% of their rated breaking strength is normally sufficient to avoid a slack guy.

5C.12 AXIAL COMPRESSION AND BENDING

Eq. 5.12-1 and 5.12-2 are the same as the corresponding formulas in the AISC Allowable Stress Design specification and the AISI specification, except that in the AISC specification they are given in terms of axial stress f_a and bending stress f_b instead of force P and moment M. Values of C_m are given here only for the case where there is no lateral displacement of one end relative to the other, since the lateral-displacement (sidesway) case is not likely to be found in latticed transmission towers.

Both the AISC and AISI specifications give an alternative simplified formula which may be used if f_a/F_a is less than 0.15. Because this case is likely to be rare in transmission tower work, the corresponding formula is not given in this Standard.

5C.13 AXIAL TENSION AND BENDING

The terms $1/(1 - P/P_e)$ in Eq. 5.12-1 account for the increase in the moments M_x and M_y due to the eccentricity of the compression force P caused by the bending of the member. If the axial force is tension, however, its effect decreases the moments. Therefore, the inclusion of terms in Eq. 5.13-1 for decreasing the moments would be logical. This is not usually done, however, and the resulting simpler formula is used in this Standard.

Note that M_{ax} and M_{ay} are to be determined according to Section 5.14. Therefore, the effects of lateral buckling for members not supported laterally are taken into account even though the lateral-buckling moment is based on a compressive stress. In other words, Eq. 5.13-1 is not based on the addition of axial tension and tensile stresses due to bending. This logic can be seen by considering the case in which P and M_x are both zero. This gives $M_y = M_{ay}$, and if the member is not supported against lateral buckling, M_{ay} should be the lateral-buckling moment.

Eq. 5.13-1 is the same as the corresponding equation in the *AISC Load and Resistance Factor Design Specification for Steel Buildings* (1986).

5C.14 BEAMS

Formulas in this section for determining allowable moments differ in form from those in the AISC and AISI specifications in that the allowable compressive stress for laterally unsupported members is computed from the allowable-stress formulas for axial compression through the use of an equivalent radius of gyration.

5C.14.4 I, Channel and Cruciform Sections

Eq. 5.14-1, which gives the equivalent radius of gyration for doubly symmetric I's, symmetric channels, and singly or doubly symmetric cruciform sections, takes both the St. Venant torsional stiffness, J, and the warping stiffness, C_w, into account. Values of $K = \sqrt{K_xK_y}$ and C_b for a number of cases for which the member and conditions are the same for warping and y-axis rotation (e.g., warping and y-axis rotation both permitted or both prevented) are given by Clark and Hill (1960) and Gaylord, Gaylord, and Stallmeyer (1992). Two formulas are used in the *AISC Specification for Structural Steel Buildings* (1989). One is obtained by taking $C_w = 0$ and the other by taking $J = 0$ and expressing the results in terms of other more familiar properties of the cross section. The larger of the two allowable stresses so obtained is used because both underestimate the buckling strength due to the omissions just mentioned. The two formulas can be used only for doubly symmetric I's and singly symmetric I's with the compression flange larger than the tension flange. Only one of the two applies to channels. On the other hand, formulas in the AISI specification for Design of Cold-Formed Steel for Structural Members (1986), are derived by assuming $J = 0$ because it is usually relatively small for thin-walled shapes of cold-formed members. However, Eq. 5.14-1 is not difficult to use, and it has the advantage of giving more accurate values of the buckling stress.

5C.14.6 Singly Symmetric I and T Sections

The approximate procedures for T's and singly symmetric I's give very good results.

5C.14.7 Other Singly Symmetric Open Sections

The formulas in this section are expressed in different terms from those in the AISI specification. However, they give identical results. Gaylord, Gaylord, and Stallmeyer (1992) give a typical example.

5C.14.8 Equal Leg Angles

The formulas in this section give critical moments for pure bending (constant moment) of equal leg angles and therefore are conservative for cases where there is a moment gradient, as for a uniformly loaded beam. However, they do not account for twisting of the angle, so they are unconservative if the load does not act through the shear center. Tables of allowable uniformly distributed load perpendicular to a leg, based on formulas that account for twist due to load eccentricity of $\pm b/2$, and which have been confirmed by an extensive series of tests, are available (Madugula and Kennedy 1985; Leigh et al. 1984). The tables cover angles with unequal legs, as well as those with equal legs. The loads are based on an allowable stress of 0.6 F_y in Madugula and Kennedy 1985, and 0.66 F_y in Leigh et al., so that the tabulated values must be divided by 0.6 and 0.66, respectively, to obtain values in conformity with tower design practice.

Predictions of the formulas in this section for load perpendicular to a leg are in good agreement with the values in Leigh et al. (1984) for beam spans up to $L/r_z = 250$. The tabular values range from about 92% for the relatively thin angles ($b/t = 16$) to about 115% (b/t about 8) of the formula values using the centerline section moduli of Eq. 5.14-11, and from about 94% to 130% using overall dimension section moduli. The overprediction of the thin angles is partially compensated for by the fact that the formulas give the critical uniform moment. Nevertheless, if the formulas are used for angles with b/t larger than 16, it is suggested that the resulting allowable moments be reduced by 10% if the member is not to be tested.

Deflections may be computed as the resultant of the u- and z-axis deflection components determined by resolving the moment M into the u- and z-axis. There are no established limits of deflection for beams in transmission tower applications.

5C.15 ALLOWABLE SHEAR

5C.15.1 Beam Webs

The upper limit of h/t in the AISC specification is given by a formula involving F_y. The limiting value 200 of this section, which is the same as the AISI specification limit, equals the AISI limit for $F_y = 60$ ksi (413 MPa) and is, therefore, adequate for steels with $F_y \leq 60$ ksi (413 MPa). It is unlikely that webs thinner than those allowed by the 200 limit will be needed in transmission towers.

The allowable shearing stresses given here are the same as those in the AISC and AISI specifications multiplied by the AISC and AISI factors of safety.

SECTION 6 DESIGN OF CONNECTIONS

6C.1 INTRODUCTION

The purchaser's procurement specifications should specify if the end and edge distances are minimum values which cannot be underrun. Tolerances for sheared and cut ends are normally established by the supplier. Edge distances are controlled by the gage lines selected, and the detailer must provide for normal rolling tolerances to avoid possible underrun of the edge distances. The rolling tolerances contained in ASTM Specification A6 should be used as a guide (*Standard Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use*).

Bolts, such as A394, are installed to the drawn-tight condition or to some specified minimum torque. Even if the bolt is supplied with a lubricant, it is difficult to fully torque a hot-dipped galvanized bolt due to the buildup of the zinc coating on the threads as the nut is tightened. Consequently, locking devices are used by many utilities to minimize possible loosening of the nut due to vibration or flexure of the structure joints.

6C.3 FASTENERS

6C.3.2 Bolt Shear Capacity

Bolts, such as A394 bolts (*Standard Specifications for Zinc-Coated Steel Transmission Tower Bolts*) are typically installed to the drawn-tight condition or to some specified minimum torque. Thus, the load transfer across a bolt is governed by direct shear rather than friction. ASTM A394 provides the specified minimum shear values when threads are included in or excluded from the shear plane. The allowable shear of $0.62F_u$ for bolts that do not have an ASTM specified shear stress is conservative (Kulak, Fisher, and Struik, 1988).

6C.3.3 Bolt Tension Capacity

The specified tensile stresses approximate those at which the rate of elongation of the bolt begins to increase significantly. The ASTM proof load stress is approximately equal to the yield stress, and $0.6F_u$ is a conservative estimate for bolts for which the proof load is not specified.

If allowable stresses exceed the yield stress, permanent stretch can occur in the bolt. This could loosen the nut and cause a loss of tightness in the joint.

6C.3.4 Bolts Subject to Combined Shear and Tension

Test on rivets and bolts indicate that the interaction between shear and tension in the fastener may be represented by formulas which plot as ellipses (Kulak, Fisher, and Struik 1988; Higgins and Munse 1952; Chesson et al. 1965). Therefore, an elliptical expression, with major and minor axes based upon the allowable shear and tension values given in Sections 6.3.2 and 6.3.3, has been specified.

6C.4 ALLOWABLE BEARING STRESS

The allowable bearing stress is the same as the allowable value in the 1978 AISC specification. This may seem unduly conservative since in the AISC specification the allowable value is applied to the service (unfactored) load while in the Standard it is applied to the factored load. However, it conforms with experience in the tower industry. Designs produced with bearing values less than or equal to $1.5F_u$ and conforming with the other provisions of this document have demonstrated satisfactory control over bolt-hole ovalization during full-scale tower tests. Furthermore, the AISC value was reduced to $1.2\ F_u$ in the 1989 edition of the specification if deformation around the hole is a consideration.

When applying these provisions, the designer must recognize that the required end and edge distances depend upon the bearing stress in the connection. It may be useful to reduce the bearing stress below the allowable value, as this may permit a reduction in the end and edge distances required.

The allowable bearing value on the bolts must be checked if the tensile strength, F_u, of the member material exceeds the F_u value of the bolt. This would occur if a A394, Type 0 bolt (F_u = 74 ksi = 510 MPa) is used to connect a A572-Grade 65 material (F_u = 80 ksi = 551 MPa). The full diameter of the bolt should be used for this calculation, with an allowable bearing stress equal to 1 [illegible] F_u of the bolt (Wilhoite 1985).

6C.5 MINIMUM DISTANCES

6C.5.1 End Distance

The provisions of this section are applicable to sheared and mechanically guided flame cut ends.

Eq. 6.5-1 provides the end distance required for strength. The required end distance is a function of the load being transferred in the bolt, the tensile strength of the connected part, and the thickness of the connected part. Test data confirm that relating the ratio of end distance to bolt diameter to the ratio of bearing stress to tensile strength gives a lower bound to the published test data for single fastener connections with standard holes (Kulak, Fisher, and Struik 1988). The end distance required by the above expression has been multiplied by 1.2 to account for uncertainties in the end distance strength of the members (Kulak, Fisher, and Struik 1988). For adequately spaced multiple bolt connections, this expression is conservative.

Eq. 6.5-2 is a lower bound on end distance which has been successfully used in tower practice in stressed members. A minimum end distance of 1.2d has been specified for redundants since they carry only secondary stresses, which are much less than stresses in the members they brace.

Latitude is provided to use the minimum end distances and determine the allowable bearing stress for this condition. Eq. 6.5-1 and Eq. 6.5-2 allow one to determine what combination of bearing value and end distance satisfies the engineering and detailing requirements. Eq. 6.5-3 places an end distance restriction on thick members such that punching of the holes does not create a possible breakout condition. If the holes are drilled in members where the end distance would be governed by Eq. 6.5-3, this requirement is not necessary. Satisfactory punching of the holes in thick material is dependent on the ductility of the steel, the adequacy of the equipment (capability of the punching equipment and proper maintenance of punches and dies), the allowed tolerances between the punch and die, and the temperature of the steel. The following guidelines have been used satisfactorily:

For 36 ksi (248 MPa) yield steel, the thickness of the material should not exceed the hole diameter

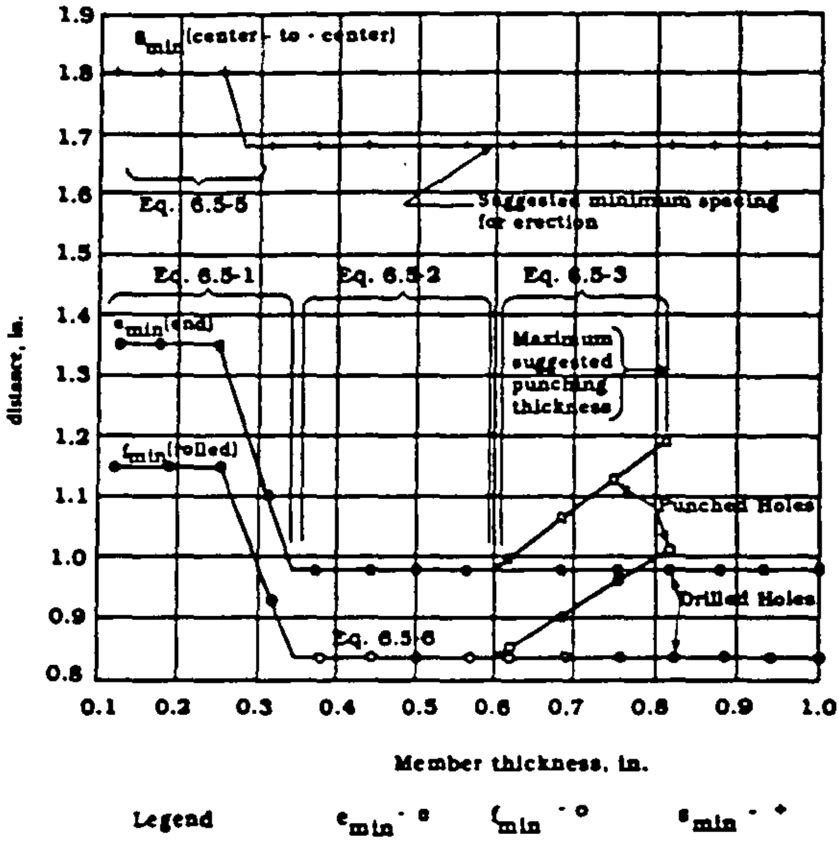
For 50 ksi (345 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/16 in. (1.6 mm); and

For 65 ksi (448 MPa) yield steel, the thickness of the material should not exceed the hole diameter minus 1/8 in. (3.2 mm).

Fig. 6C.1 shows the end distances for a A394, Type 0, 3/4 in. (19 mm) diameter bolt used to connect A36 steel stressed members. The maximum bearing stress of $1.5F_u$ has been used unless shear governs the maximum force in the bolt. Table 6C.1 contains the tabulated values for Fig. 6C.1 and shows that Eq. 6.5-1 governs up through a thickness of 5/16 in. (6.9 mm); Eq. 6.5-2 governs for thicknesses of 3/8 in. (9.5 mm) through 9/16 in. (14.3 mm); and, Eq. 6.5-3 governs thicknesses of 5/8 in. (15.9 mm) and above, unless the holes are drilled. For drilled holes, Eq. 6.5-2 would continue to govern over this range of thicknesses. The range of thicknesses over which each equation governs changes if the bearing stress is reduced below the maximum allowable bearing stress of $1.5F_u$. Fig. 6C.2 illustrates the proper application of the required end distance when a member is clipped and the sheared surface is not perpendicular to the primary axes of the member.

6C.5.2 Center-to-Center Bolt Hole Spacing

Two factors must be considered when determining the minimum center-to-center bolt spacing. Eq. 6.5-5 is the strength expression for end distance (Eq. 6.5-1) plus 0.6 times the diameter of the adjacent bolt. The term 0.6d has been specified instead of the more commonly recognized 0.5d, to provide greater control over the reduction in center-to-center material due to hole breakout dur-



End, rolled edge and center to center distances for a A394, Type O, 3/4 in. diameter bolt used to connect A36 stressed steel members. Bolt in single shear (through threads). Bearing stress = 1.5 Fu

FIG. 6C.1 A394 Bolt Connecting A36 Steel (1 in. = 25.4 mm)

TABLE 6C.1. A394 Bolt Connecting A36 Steel (1 in. = 25.4 mm; 1 ksi = 5.89 MPa; 1 kip = 4,450 N)

Values for end, rolled edge and center-to-center distances for a A394, Type 0, 3/4 in. diameter bolt used in single shear through the threads to connect A36 steel stressed members. Bearing stress = 1.5 F_u

Member Thickness t (in.) (1)	Bearing Stress 1.5 F_u (ksi) (2)	Force Bolt P (kips) (3)	Eq 6.5-1 e (in.) (4)	Eq 6.5-2 e (in.) (5)	Eq. 6.5-3 e (in.) (6)	e_{min} (in.) (7)	Eq. 6.5-5 S_{min} (in.) (8)	Suggested Erection Spacing (in.) (9)	S_{max} (in.) (10)	Eq 6.5-6 t_{max} (in.) (11)
1/8	87.0	8.16	1.35	0.98	0.50	1.35	1.80	1.67	1.80	1.15
3/16	87.0	12.23	1.35	0.98	0.56	1.35	1.80	1.67	1.80	1.15
1/4	87.0	16.31	1.35	0.98	0.63	1.35	1.80	1.67	1.80	1.15
5/16	87.0	16.65[a]	1.10	0.98	0.69	1.10	Less	1.67	1.67	0.94
3/8	87.0	16.65[a]	Less	0.98	0.75	0.98	Than	1.67	1.67	0.83
7/16	87.0	16.65[a]	Than	0.98	0.81	0.98	Col. 9	1.67	1.67	0.83
1/2	87.0	16.65[a]	Col. 5	0.98	0.88	0.98		1.67	1.67	0.83
9/16	87.0	16.65[a]		0.98	0.94	0.98		1.67	1.67	0.83
5/8	87.0	16.65[a]		0.98	1.00	1.00		1.67	1.67	0.85
11/16	87.0	16.65[a]		0.98	1.06	1.06		1.67	1.67	0.90
3/4	87.0	16.65[a]		0.98	1.13	1.13		1.67	1.67	0.96
13/16	87.0	16.65[a]		0.98	1.19	1.19		1.67	1.67	1.01
7/8	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]
15/16	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]
1	87.0	16.65[a]		0.98	0.98[b]	0.98[b]		1.67	1.67	0.83[b]

Eq 6.5-1 = $1.2P/(F_u t)$.
Eq 6.5-2 = $1.3d$.
Eq 6.5-3 = $t = d/2$.

Eq 6.5-5 = $1.2P/(F_u t) + 0.6d$
Erection = nut c dimension + 0.375.
Eq 6.5-6 = $85 e_{min}$.

Bolt diameter = 0.75 in.
Bolt shear strength = 16.65 kips
Point to point nut dimension = 1.30 in
Member tensile strength (F_u) = 58.0 ksi

a P Limited by single shear strength (through threads) of bolt.
b t = 13/16; suggested maximum punching thickness (13/16 in. dia. hole in A36 steel). Distance shown is for drilled holes

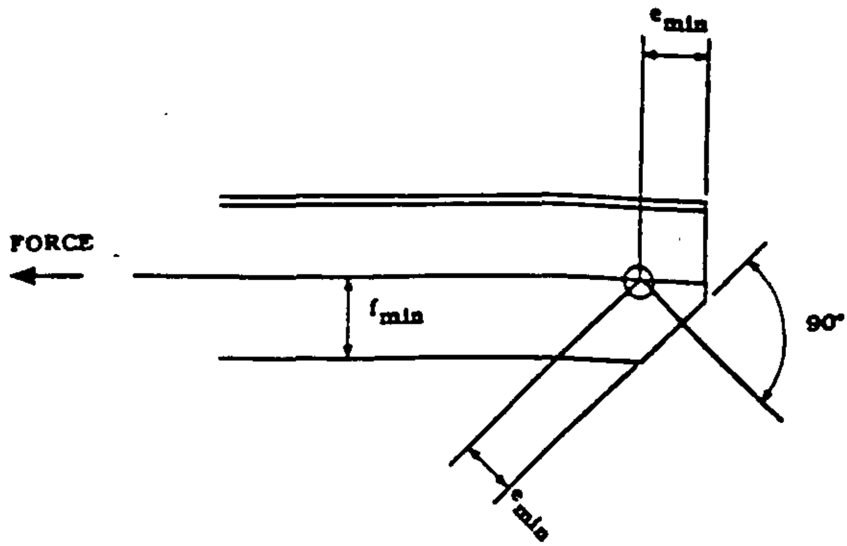


FIG. 6C.2. Application of e_{min} to Clipped Member

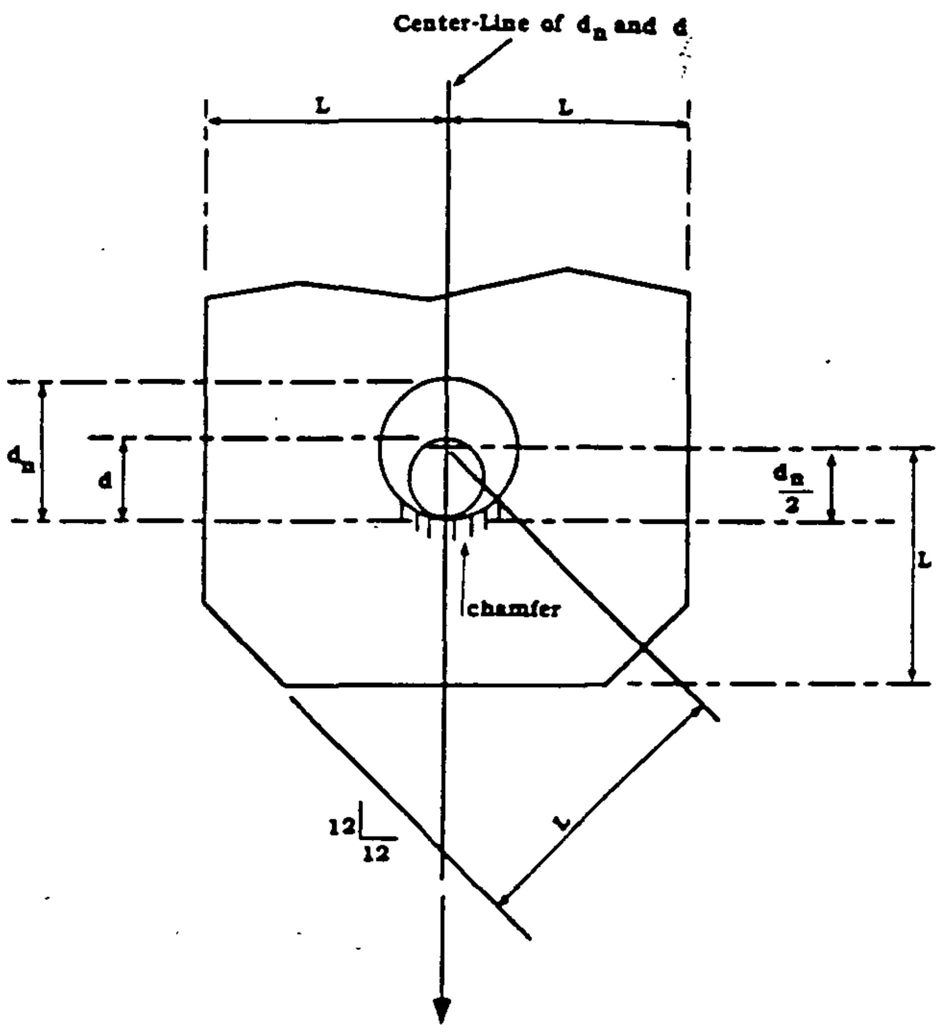


FIG. 6C.3. Application of Oversized Holes ing punching. For low bearing values, the spacing requirements predicted by Eq. 6.5-5 may be less than the spacing required for installation. Spacing requirements for convenient installation should be determined by adding 3/8 in. (9.5 mm) to the width across the points of the nut being used. Fig. 6C.1 and Table 6C.1 show that the installation requirement governs for thick members.

6C.5.3 Edge Distance

Several studies have considered the effects of end and edge distance on the strength of connections for stressed members (Bodegom et al. 1984; Kennedy and Sinclair 1969; Gilchrist and Chong 1979). Tests by Bodegom et al. (1984) establish a ratio of the rolled edge distance to the sheared edge distance to prevent a tension tear-out of the rolled edge. Eq. 6.5-6 combines the results given in Bodegom et al. (1984) with the minimum end distances in Eq. 6.5-2. The same ratio is retained for determining the relationship between all end distances and the required rolled edge distance. When the edge distance is to a sheared or mechanically guided flame-cut edge, 1/16 in. (1.6 mm) is added to the rolled edge distance. Fig. 6C.1 shows the relationship between the rolled edge and end distances.

6C.6 ATTACHMENT HOLES

Oversized holes are commonly used as load attachment points for insulator strings, overhead ground wires, and guys. These holes are not used where connections are designed for load reversal. The possible failure modes considered are bearing, tension and shear. These recommendations do not exclude the use of other attachment holes or slots designed by rational analysis.

Eq. 6.6-1 assumes the member will fail in shear with shear planes developing at each side of the bolt through the edge of the member. Fig. 6C.3 illustrates the various terms used. The equation was developed by replacing e in Eq. 6.5-1 with (L-$0.5d_h$), solving for P, and multiplying the result by 0.9. Given that L is defined as the distance from the center to any member edge, the member will always fail in shear before it fails in tension. The dimension L shown for the edge distance, perpendicular to the line of action of P, may be reduced if analysis shows that the sum of the tensile stress P/A and the tensile bending stress does not exceed $0.67F_y$, where F_y is the member yield stress.

Eq. 6.6-1 has been limited to hole diameters less than or equal to two bolt diameters. This represents the range of experience over which the equation has been used in practice. No adjustment to the equation is required for slight chamfering of the hole.

For attachment plates subject to bending, additional analysis will be required to determine the plate thickness. Eq. 6.6-2 limits the bearing stress to 0.9 times the allowable value for standard holes to provide an additional factor for possible wear.

For everyday loading, as specified by the purchaser, P should not exceed $0.5dtF_u$ to avoid indentation of the material under sustained loading and excessive wear. Everyday loading may be defined as that resulting from the bare weight of the conductor at 60°F (16°C) final sag, unless the location is subjected to steady prevailing wind. If the location is subjected to steady prevailing wind, the everyday loading may be considered to be the resultant load caused by the bare weight of the conductor and the prevailing wind at 60°F (16°C) final sag.

SECTION 8 TESTING

8C.1 INTRODUCTION

Although this Standard provides specific recommendations for determining the allowable strengths of individual members and connections, analysis techniques used to predict forces in individual members and connections are dependent upon assumptions relative to the distribution of forces and reactions between portions of the overall structure, so new or unique structure configurations are often tested to verify that the stress distribution in the members is in accordance with the analysis assumptions, and that the members selected, and the connections as detailed, are adequate to carry the design loads. This type of test is referred to as a proof test and is performed on a prototype of the full structure or a component of the structure, usually before that structure or a structure of similar design is fabricated in quantity.

A traditional proof test is set up to conform to the design conditions, that is, only static loads are applied, the prototype has ideal foundations, and the restraints at the load points are the same as in the design model. This kind of test will verify the adequacy of the members and their connections to withstand the static design loads specified for that structure as an individual entity under controlled conditions. Proof tests can provide insight into actual stress distribution of unique configurations, fit-up verification, action of the structure in deflected positions, adequacy of connections, and other benefits. The test cannot confirm how the structure will react in the transmission line where the loads may be dynamic, the foundations may be less than ideal, and where there is some restraint from intact wires at the load points.

The testing procedures provided in this document are based on performing a proof test using a test station that has facilities to anchor a single prototype to a suitable base, load and monitor pulling lines in the vertical, transverse, and longitudinal directions, and measure deflections.

8C.2 FOUNDATIONS

8C.2.1 General

The type, rigidity, strengths, and moment reactions of the actual attachments of a prototype to a test bed have a major effect on the ability of the prototype members to resist the applied loads; consequently, the reactions of the test foundations should be similar to the expected reactions of the in-service foundations.

8C.2.2 Rigid Structures

Tests of a fairly rigid, four-legged, latticed structure designed for stub angles set in concrete, anchor bolts and base plates on concrete, or earth grillage foundations, are usually performed on special stub angles bolted or welded to the test station's rigid base. Accurate positioning of the footings is necessary to prevent abnormal stresses in the structure members.

8C.2.3 Direct Embedded Structures

It is recommended that structures designed for direct embedment be tested in a two-part program.

8C.2.3.1 Embedded Portion

Soil properties at a permanent test station probably will not match the properties of the soil on the transmission line. Tests that are dependent on soil resistance should be done at the line site, since only load cases that control the anchorage design need be performed.

8C.2.3.2 Above-Ground Portion

The above-ground portion of the prototype should be modified to be bolted or welded to the test station's foundation. All controlling load cases should be applied to this prototype.

8C.2.4 Components

For component tests, especially single members, the amount of rotational rigidity of the supports is critical. All the other parameters of the attachments of the prototype to the test bed must also be as close to the in-service conditions as possible.

8C.3 MATERIAL

Proper interpretation of the data obtained from testing is critical in establishing the true capacity of individual members. There is concern about using members in a prototype which have yield points considerably higher than the minimum guaranteed yield value that is used as the basis for design. The actual yield points of tension members and of compression members with KL/r values less than 120 are critical in determining the member capacity. Consequently, the guidelines shown in Fig. 8C.1 are suggested as a basis for determining the maximum yield point values for these members of the prototype. All other members of the prototype must conform to the standard material specifications but their actual yield points are not as critical to their load-carrying capabilities.

8C.4 FABRICATION

Normally, the prototype is not galvanized for the test unless the purchaser's order states otherwise.

8C.5 STRAIN MEASUREMENTS

Stress determination methods, primarily strain gaging, may be used to monitor the loads in individual members during testing. Comparison of the measured unit stress to the predicted unit stress is useful in validating the proof test and refining analysis methods. Care must be exercised when instrumenting with strain gages, both as to location and number, to assure valid correlation with design stress levels.

8C.6 ASSEMBLY AND ERECTION

The erected prototype shall conform with the special requirements of the purchaser's instructions; many purchasers specify minimum torque for bolt tightening and that the vertical axis shall not be out of plumb by more than 1 in. (25 mm) for every 40 ft. (12 m) of height.

8C.8 LOAD APPLICATION

V-type insulator strings shall be loaded at the point where the insulator strings intersect. If the insulators for the towers in the line are V-type strings that will not support compression, it is recommended that articulated bars or wire rope slings be used to simulate the insulators. If compression or cantilever insulators are planned for the transmission line, members that simulate those conditions should be used in the test.

Compression on unbraced panels due to bridling of load lines should be avoided.

As a structure deflects under load, load lines may change their direction of pull. Adjustments shall be made in the applied loads or the test rigging shall be offset accordingly so that the vertical, transverse, and longitudinal vectors at the load points are the loads specified in the tower loading schedule.

8C.9 LOADING PROCEDURE

It is recommended that the primary consideration in establishing the sequence of testing should be that those load cases having the least influence on the results of successive tests be tested first. A secondary consideration should be to simplify the operations necessary to carry out the test program.

8C.10 LOAD MEASUREMENT

The effects of pulley friction should be minimized. Load measurement by monitoring the load

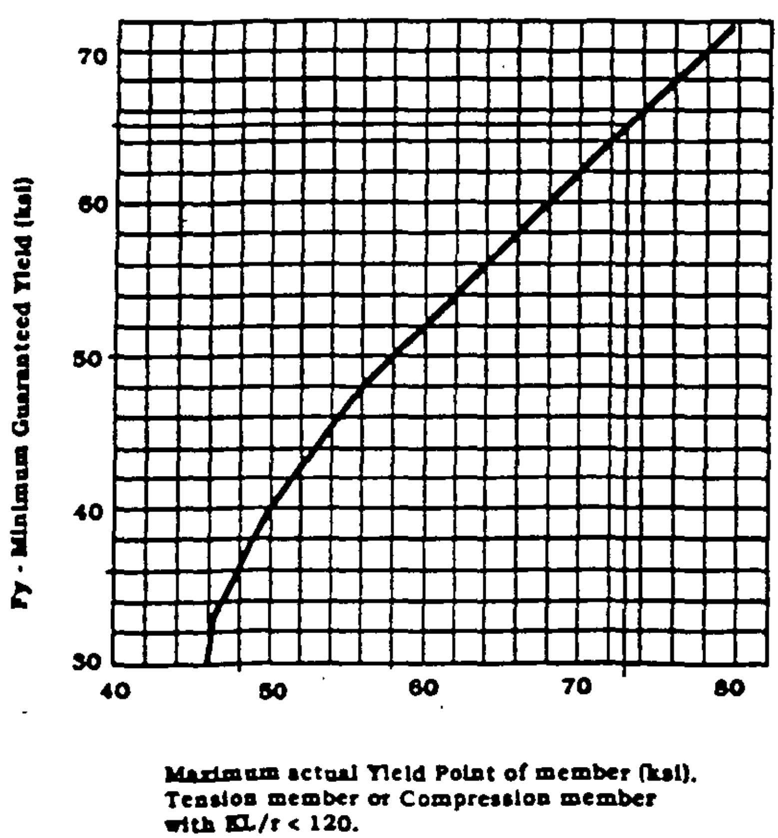


FIG. 8C.1. Maximum Overstrengths for Members of the Prototype (1 ksi = 5.89 MPa) in a single part of a multipart block and tackle arrangement should be avoided.

8C.11 DEFLECTIONS

Points to be monitored shall be selected to verify the deflections predicted by the design analysis.

8C.13 DISPOSITION OF PROTOTYPE

An undamaged structure is usually accepted for use in the transmission line after all components are visually inspected and found to be structurally sound and within tolerances.

SECTION 9 STRUCTURAL MEMBERS AND CONNECTIONS USED IN FOUNDATIONS

9C.1 INTRODUCTION

The material in Section 9 covers structural members and connections normally supplied by the steel fabricators. The *Guide for Design of Steel Transmission Towers*, Second Edition (1988), provides extensive references for additional guidance on appropriate loadings and supplementary design procedures.

9C.2 GENERAL CONSIDERATIONS

The *Guide for Design of Steel Transmission Towers*, Second Edition (1988), provides additional geotechnical considerations and special situations which should be considered in establishing overall design criteria.

9C.2.2 Pressed Plates

For the pressed plate foundation, a relatively thick steel plate washer is welded to the bottom of the stub angle. This allows an attachment for bolting to the pressed plate and provides a transfer of the axial load from the stub angle to the corners of the pressed plate.

9C.3 DETERIORATION CONSIDERATIONS

Concrete foundations should be properly sloped to drain so that water pockets do not accumulate with ground material and cause excessive corrosion of the tower base material. If towers are located where ground water can be highly corrosive, such as ash pits, industrial drainage areas, and oil refineries, concrete foundations should be used. If steel is exposed to such a ground water environment, special protection is essential. Proper drainage around the steel should be established and periodic inspections conducted. In some cases, it may be necessary to apply an additional protective coating such as a bitumastic compound to the steel. If new towers are located in such an environment, any steel members exposed to this severe ground condition should be increased in thickness at least 1/16 in. (1.6 mm) as a corrosion allowance.

9C.4 DESIGN REQUIREMENTS FOR CONCRETE AND REINFORCING STEEL

ACI 349 requirements for anchorage material have been modified for use in this standard. The formulas in this standard use yield strength, F_y, of the anchorage material and are suitable to ensure a ductile failure of the anchor material prior to a brittle failure of the concrete.

The *Guide for Design of Steel Transmission Towers*, Second Edition (1988), outlines some procedures for determining side cover distance for tension and shear in concrete for smooth anchor bolts and stub angles. These formulas were developed following ACI 349 criteria.

9C.6 SHEAR CONNECTORS

9C.6.1 Stud Shear Connectors, Fig. 9.2c

Additional information is available on stud shear connectors in Salmon and Johnson (1990).

9C.6.2 Angle Shear Connectors, Fig. 9.2b

The values obtained using Eq. 9.5.2 are based on initial yielding of the angle in bending at a stress of 1.19 f'_c in the concrete. The 1.19 constant is based on the concrete pier having an area at least four times as large as the bearing area of a pair of shear connector angles.

SECTION 10.0 QUALITY ASSURANCE/ QUALITY CONTROL

10C.2 QUALITY ASSURANCE

Detailed suggestions on establishing a QA program are covered in *Guide for Design of Steel Transmission Towers*, Second Edition (1988).

10C.3 QUALITY CONTROL

Detailed suggestions on establishing a QC program are covered in *Guide for Design of Steel Transmission Towers*, Second Edition (1988).

SECTION 11.0 REFERENCES

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APPENDIX I
NOTATION

The following symbols are used in this standard:

- A = cross-sectional area (in.^{2}) (mm^2);
- A_g = gross cross-sectional area (in.^{2}) (mm^2), or gross area of angle, or net area if there is a hole in the angle at the intersecting plane of the concrete foundation and the angle (in.^{2}) (mm^2)
- A_n = net cross-sectional area (in.^{2}) (mm^2);
- A_s = tensile stress area of bolt (in.^{2}) (mm^2);
- A_{sc} = cross-sectional area of stud shear connector (in.^{2}) (mm^2);
- A_t = minimum net area in tension (in.^{2}) (mm^2); or, minimum net area in tension from the hole to the toe of the angle perpendicular to the line of force (in.^{2}) (mm^2);
- A_v = minimum net area in shear (in.^{2}) (mm^2);
- a = distance from shear center to load plane (in.) (mm);
- b, b_1, b_2 = effective design widths of elements (in.) (mm), or width of leg $- t/2$ (in.) (mm);
- C = constant based on ratio of f_1 and f_2;
- C_b = coefficient in formula for allowable bending stress;

C_c = column slenderness ratio separating elastic and inelastic buckling;

C_m = coefficient applied to bending term in interaction formula for prismatic members;

C_w = warping constant of cross section (in.^{6}) (mm^6);

D = downthrust load: net difference in compression and uplift reactions on anchor bolts (kips) (N);

d = nominal diameter of bolt (in.) (mm), or minimum depth of stiffener (in.) (mm);

d_h = diameter of attachment hole (in.) (mm);

E = modulus of elasticity of steel (29,000 ksi) (200,000 MPa);

E_c = modulus of elasticity of concrete (ksi) (MPa);

e = distance from center of hole to end of member (in.) (mm);

e_m = required distance from center of hole to end of member (in.) (mm);

F_a = axial compressive stress permitted in primatic member in absence of bending moment (ksi) (MPa);

F_b = bending stress permitted in primatic member in absence of axial force (ksi) (MPa);

F_{cr} = critical stress for local buckling of plain angle members (ksi) (MPa);

F_t = allowable axial tensile strength (ksi) (MPa);

$F_{t(v)}$ = allowable axial tensile stress in conjunction with shear stress (ksi) (MPa);

F_u = specified minimum tensile strength (ksi) (MPa);

F_v = allowable shear stress (ksi) (MPa), or allowable average shear stress for beam webs (ksi) (MPa);

F_y = specified minimum yield stress (ksi) (MPa);

f = stress in compression element computed on the basis of effective design width (ksi) (MPa); distance from center of hole to edge of member (in.) (mm);

f_1, f_2 = stress, in tension or compression, on an element (ksi) (MPa);

f'_c = specified compressive strength of concrete at 28 days (ksi) (MPa);

f_m = required distance from center of hole to edge of member (in.) (mm);

f_v = computed shear stress (ksi) (MPa);

g = transverse spacing locating fastener gage lines (in.) (mm);

h = clear distance between flanges of beam (in.) (mm);

I = moment of inertia in truss plane (in.^{4}) (mm^4);

I_{ps} = polar moment of inertia about shear center (in.^{4}) (mm^4);

I_u = moment of inertia about U-U axis (in.^{4}) (mm^4);

I_x = moment of inertia about X-X axis (in.^{4}) (mm^4);

I_y = moment of inertia about Y-Y axis (in.^{4}) (mm^4);

I_z = moment of inertia about Z-Z axis (in.^{4}) (mm^4);

J = torsional constant of cross section (in.^{4}) (mm^4);

j = special section property for torsional-flexural buckling (in.) (mm);

K = effective length factor for prismatic member;

K_t = effective length factor for warping and rotation;

K_u, K_x, K_y = effective length factor for buckling in designated axis;

L = unbraced length of column (in.) (mm), distance from center of attachment hole to member edge (in.) (mm);

L_x, L_y = unbraced length in designated axis (in.) (mm);

M_{ax} = allowable bending moment about X-X axis (in.-kip) (mm-N);

M_{ay} = allowable bending moment about Y-Y axis (in.-kip) (mm-N);

M_b = lateral buckling moment for angles (in.-kip) (mm-N);

M_e = elastic critical moment (in.-kip) (mm-N);

M_x = bending moment about X-X axis (in.-kip) (mm-N);

M_y = bending moment about Y-Y axis (in.-kip) (mm-N);

M_{yc} = moment causing yield at extreme fiber in compression (in.-kip) (mm-N);

M_{yt} = moment causing yield at extreme fiber in tension (in.-kip) (mm-N);

M_1 = smaller moment at end of unbraced length of beam column (in.-kip) (mm-N);

M_2 = larger moment at end of unbraced length of beam column (in.-kip) (mm-N);

n = number of threads per unit of length (in.) (mm);

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P = capacity of angle shear connector (kips) (N), axial tension or compression load on member (kips) (N), force transmitted by a bolt (kips) (N);
P_a = allowable axial compression load on member (kips) (N);
P_{ex} = Euler buckling load in X-X axis (kips) (N);
P_{ey} = Euler buckling load in Y-Y axis (kips) (N);
Q_n = capacity of a shear connector (kips) (N)
r = governing radius of gyration (in.) (mm);
r_{ps} = polar radius of gyration about shear center (in.) (mm);
r_t = equivalent radius of gyration for torsional buckling (in.) (mm);
r_{tf} = equivalent radius of gyration for torsional-flexural buckling (in.) (mm);
r_u = radius of gyration for U-U axis (in.) (mm);
r_x = radius of gyration for X-X axis (in.) (mm);
r_y = radius of gyration for Y-Y axis (in.) (mm);
r_z = radius of gyration for Z-Z axis (in.) (mm);
S_u, S_w, S_x, S_z = elastic section modulus in designated axis (in.^{3}) (mm^3);
S_{xc} = elastic section modulus about X-X axis of compression flange (in.^{3}) (mm^3);
s = longitudinal center-to-center spacing (pitch) of any two consecutive holes (in.) (mm);
s_m = required spacing between centers of adjacent holes (in.) (mm);
T = axial tensile load on anchor bolts (kips) (N);
t = thickness of element (in.) (mm);
u = U-U axis designation;
u_o = distance between shear center and centroid (in.) (mm);
V = shear load perpendicular to anchor material or parallel to the intersecting plane (kips) (N);
V, V_1, V_2 = shear in a single-angle beams (kips) (N);
w = flat width of element (in.) (mm);
w_s = flat width of edge stiffener (in.) (mm);
x = X-X axis designation;
y = Y-Y axis designation;
y_o = distance between shear center and centroid (in.) (mm);
z = Z-Z axis designation;
α = angle between bracing member and supported members (degrees); and
θ = angle between flange and stiffener lip, angle between load and z-axis (degrees);
μ = coefficient of friction;
ϕ = resistance factor;
Ψ = unit factor as specified in text.

NOTA:

Los escalones se localizarán a 300 cm a partir del cimiento espaciados alternativamente a 40 cm en sentido vertical hasta 50 cm abajo de la corona e irán colocados horizontalmente en arcos de 50 cm en diámetros del poste mayores de 30 cm , y a 180° en diámetros menores a 30 cm .Colocando adicionalmente 4 escalones de trabajo a cada 90° y a 120 cm abajo de la itersección de cada brazo con el fuste,100 cm arriba de estos escalones y a 180° ,se colocarán 2 omegas de acero redondo de 16 mm de diámetro para el cinturón de seguridad.

NOTA:

Los escalones se localizarán a 300 cm a partir del cimiento espaciados alternativamente a 40 cm en sentido vertical hasta 50 cm abajo de la corona e irán colocados horizontalmente en arcos de 50 cm en diámetros del poste mayores de 30 cm , y a 180° en diámetros menores a 30 cm .Colocando adicionalmente 4 escalones de trabajo a cada 90° y a 120 cm abajo de la itersección de cada brazo con el fuste,100 cm arriba de estos escalones y a 180° ,se colocarán 2 omegas de acero redondo de 16 mm de diámetro para el cinturón de seguridad.

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

HERRAMIENTAS PARA EL MONTAJE

USO Y DESCRIPCIÓN

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

3.7 Herramientas para montaje

Una lista completa de revisión del equipo y herramientas que se utilizan en el montaje de estructuras de acero para edificios deberá incluir los conceptos que se indicarán a continuación. Por conveniencia se ha tabulado en orden alfabético. La selección de las partidas para una lista estándar de herramientas dependerá de las necesidades del montador, del tipo de estructuras que espera montar, y deberá arreglarse para cumplir sus necesidades particulares. Cuando la pieza indicada es de uso diario, no se da ninguna explicación o descripción, pero cuando es poco común o especial para el montaje de estructuras, se describirá e ilustrará lo suficiente para identificarla, o para explicar su utilidad y necesidad en el trabajo.

Lista de Herramientas

Azuela.

Anclas: tipo horquilla (Fig. 3.7.1); con extremo abierto (Fig. 3.7.2). Se deberá tener a mano una amplia existencia en diferentes dimensiones y capacidades, de tal manera que puedan enviarse con anticipación al lugar de la obra para su instalación. Con frecuencia, el contrato de montaje se concede hasta después de que se han iniciado los trabajos de cimentación; por lo tanto, las anclas deben despacharse con prontitud, a fin de que las anclas tipo horquilla se puedan colocar en la cimbra antes que se cuele el concreto; o bien barrenar los agujeros en la roca, para las anclas de extremos abiertos, mientras el contratista de la cimentación tenga su equipo disponible.

Taladro: (Fig. 3.7.3); para perforaciones en madera.

Automóvil: de pasajeros; camión, camioneta, camión para trabajo pesado, Jeep. Las licencias se deben revisar para asegurar el uso legal de los carros en el área en que se van a utilizar. En algunos estados las grúas montadas sobre camión requieren placas especiales de circulación y deberán obtenerse si se requiere transportar la grúa a través de varios estados, o simplemente para su uso en el lugar de la obra.

Hacha.

Punzón para sacar conectores: de mano (Fig. 3.7.4). Cuando es necesario sacar remaches, una de las cabezas de éstos se quema con soplete o se corta con un cortaremaches y después se utiliza el punzón de mano para empujar el vástago fuera del agujero. También es muy útil para extraer tornillos, cuando éstos, después de quitarles las tuercas, permanecen muy apretados dentro de sus agujeros.

Viga equilibradora (Fig. 3.7.5): es un mecanismo que se utiliza cuando se requiere levantar una pieza del equipo, que debido a su flexibilidad o longitud es inestable cuando se iza por el centro, cuando los estrobos de izaje no se pueden utilizar con seguridad.

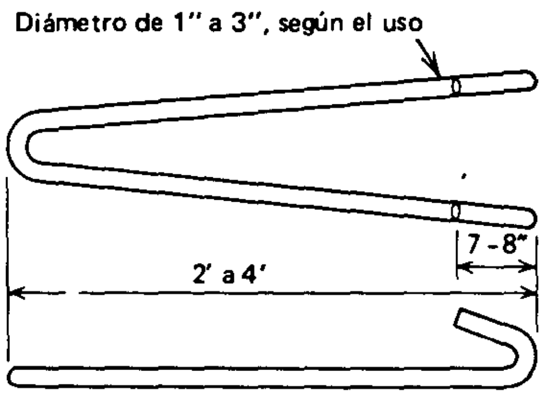


Figura 3.7.1 Ancla tipo horquilla (para concreto).

También puede invertirse, para utilizar dos piezas de equipo para el izaje, si el elemento que se izará puede sujetarse de su centro de gravedad.

Barriles.

Barras: cincel, (Fig. 3.7.6) pesadas, livianas; conectoras, de paleta

(Fig. 3.7.7); planos (rectos) (Fig. 3.7.8); sacaclavos (Fig. 3.7.9) barreta (Fig. 3.7.10); palanca (Fig. 3.7.11). El cincel tiene un extremo diseñado para cortar y el otro para golpearlo con el mazo u otra herramienta. La barra sacaclavos y la barreta tienen su extremo de agarre ligeramente inclinado con respecto al resto de la barra, para proporcionar brazo de palanca. El extremo puntiagudo de la barra conectora se utiliza para alinear los agujeros de los miembros que se van a conectar, mientras que el extremo plano se utiliza para guiar la pieza a su lugar.

Canasta: para tornillos (Fig. 3.7.12); una canasta de metal que permita mantener el buen orden y manejo de los tornillos, rondanas y herramientas pequeñas, así como también proporcionar seguridad al transportar tales materiales por medio de una cuerda de izaje. Deberán estar hechas con tales dimensiones que no se volteen cuando se levanten por el asa.

Campanas: equipo manual de señalación (campanillas), cuerda, poleas (giratorias); equipos eléctricos de señales, luces, caja de control de señales, cable de alambre. Se utilizan sobre todo para las maniobras con grúas y cuando el operador del malacate no pueda ver clara o directamente al señalador. En el caso del sistema manual, se utiliza un badajo con resorte para golpear la campana, jalando el cordón y levantando el badajo hasta un punto donde al soltarlo, el resorte hace que golpee a la campana. El equipo dependerá de lo que se necesite para operar la pluma, la carga, el carro transversal, la rueda impulsora, etc. Las campanas tendrán diferentes tonos para cada una de las distintas operaciones.

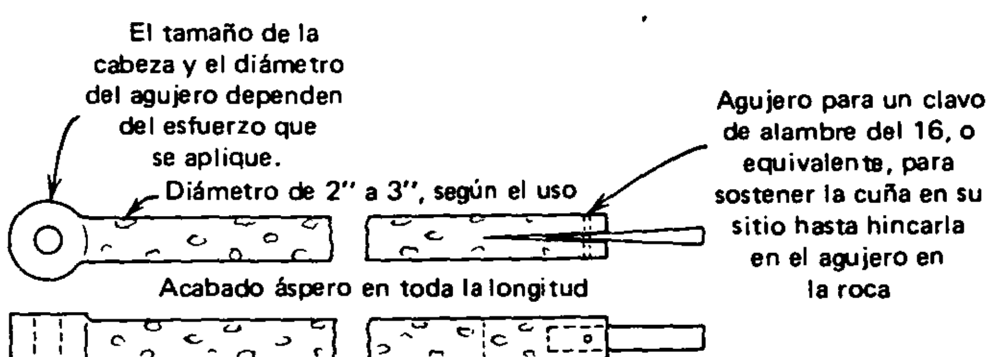


Figura 3.7.2 Ancla de extremo abierto (para roca).

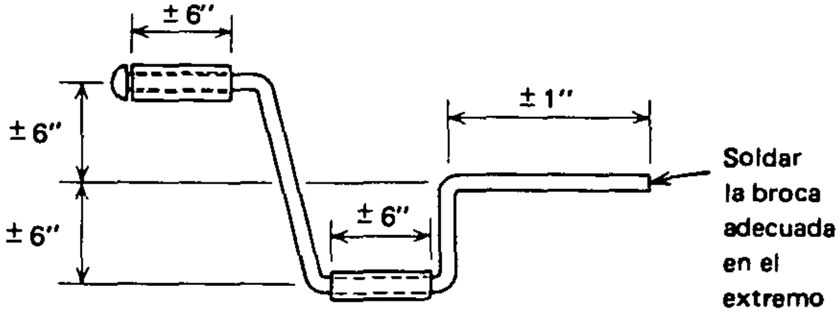


Figura 3.7.3 Taladro.

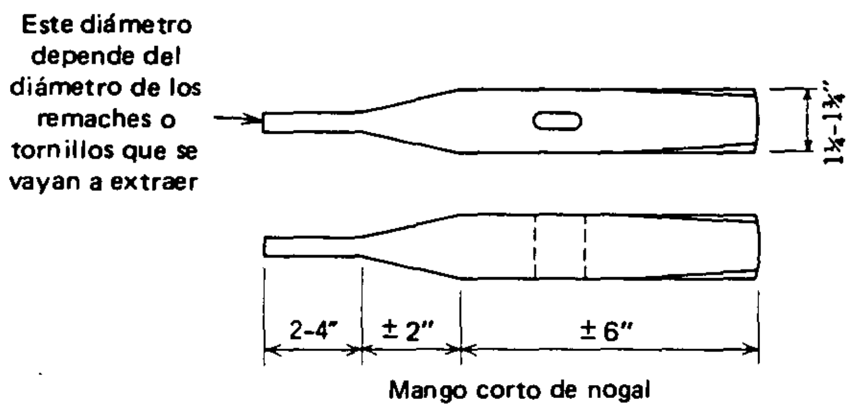


Figura 3.7.4 Punzón de mano para sacar conectore:

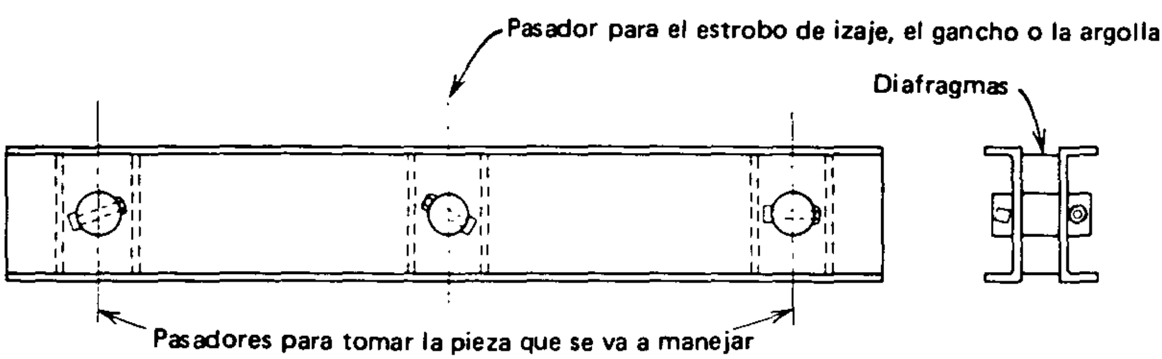


Figura 3.7.5 Viga equilibradora (esquema típico). Puede invertirse, para que dos piezas de equipo puedan izar una pieza en el centro.

El cordón de la campana para el sistema manual debe ser de un tejido grueso o de alambre flexible y ligero. Se utilizan poleas para permitir al señalador "jalar las campanas" en varias direcciones desde el punto en donde la cuerda baja al malacate; estas poleas se montan sobre un soporte temporal de madera o metal, el cual se traslada al piso en el cual se está trabajando después de cada cambio de nivel en un edificio, alimentando los cordones para las campanas a través de una polea giratoria en el soporte, hasta llegar al malacate.

En el sistema eléctrico, las luces pueden ser todas blancas o de diferentes colores, si van a estar montadas sobre sus propios tambores de operación; si se van a utilizar campanas operadas por electricidad, en lugar de luces, éstas deberán tener diferentes tonos, como en el caso del sistema operado a mano. La caja de señales debe ser impermeable y provista de botones para accionar las luces o las campanas; en general estas cajas llevan una correa para que el señalador se la pueda colgar al cuello. En el sistema eléctrico se deberá contar con un mecanismo integrado de seguridad, que indique al operador de la grúa cuando exista una falla eléctrica o cuando los alambres se hayan desconectado o cortado.

Yunque de herrero: de fuelle, de forja, equipo para tornillo de.

Garruchas: de cable manila: de una polea (Fig. 3.7.13); con doble polea; con triple polea; con poleas múltiples, como gancho, con mango,

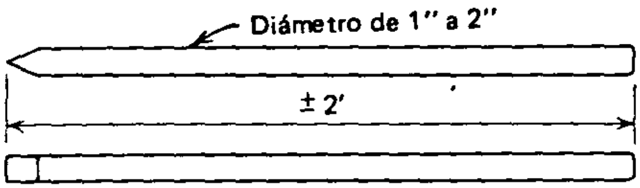


Figura 3.7.6 Cincel.

con polea de bisagra (Fig. 3.7.14); de compuerta (Fig. 3.7.15). Para cable de acero, de una polea (Fig. 3.7.16); con doble polea, con triple polea, con cuatro poleas, con poleas múltiples; con gancho, con mango; tipo violín (Fig. 3.7.17); con compuerta, con polea de bisagra, en tándem (Fig. 3.7.18); con pesos, poleas de repuesto.

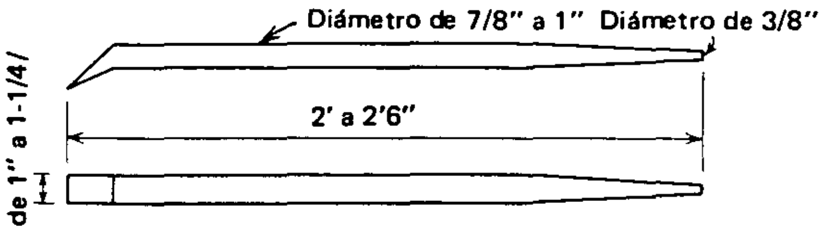


Figura 3.7.7. Barra de paleta.

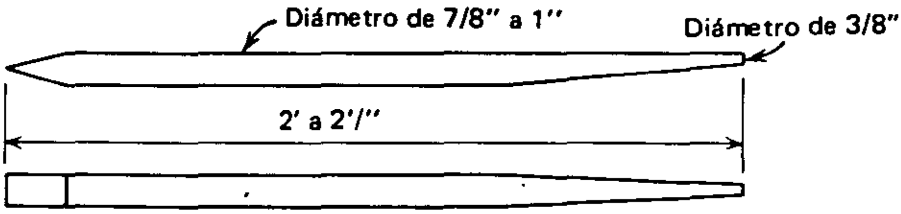


Figura 3.7.8 Barra plana (recta).

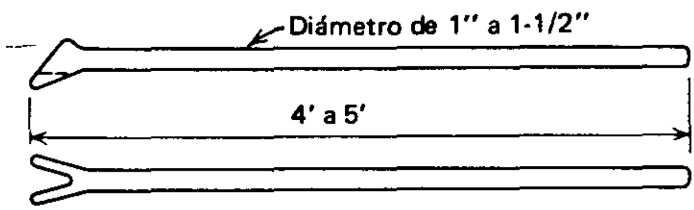


Figura 3.7.9 Barra sacaclavos.

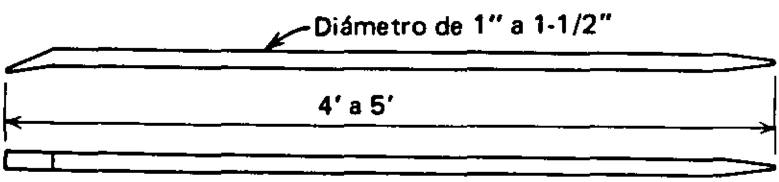


Figura 3.7.10 Barreta.

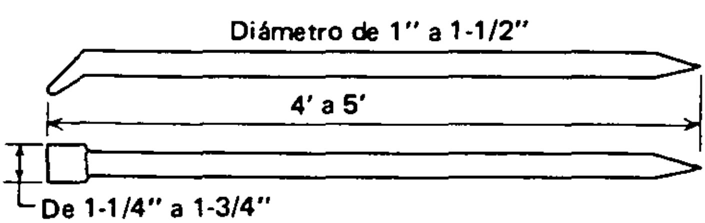


Figura 3.7.11 Barra de palanca.

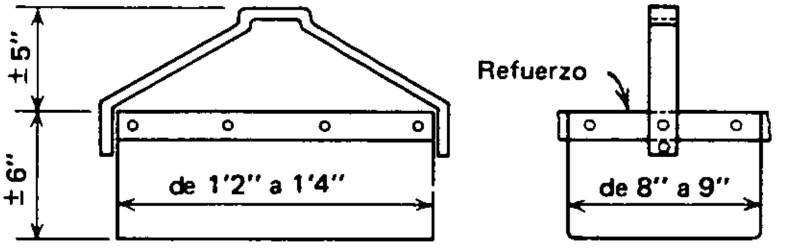


Figura 3.7.12 Canasta para tornillos. Material principal: lámina calibre 16, rolada en frío, con perforaciones.

Tornillos: para montaje; rondanas para montaje, para obra falsa, rondanas para obra falsa, para empalmes de plumas, de mástiles, de plumas giratorias, largueros, pies derechos, postes grúas, rueda impulsora, etc. Si los tornillos permanentes de la estructura son de alta resistencia, éstos pueden utilizarse en vez de los tornillos especiales para montaje. En general los tornillos para la obra falsa tienen cuerda en sus dos extremos, en vez de tener la cabeza en un extremo y la cuerda para la tuerca en el otro.

Bolsa para tornillos: de hombro; de cinturón. Las bolsas de lona para tornillos pueden asegurarse con argollas al cinturón del montador; o se puede utilizar una correa larga para el hombro. Este último

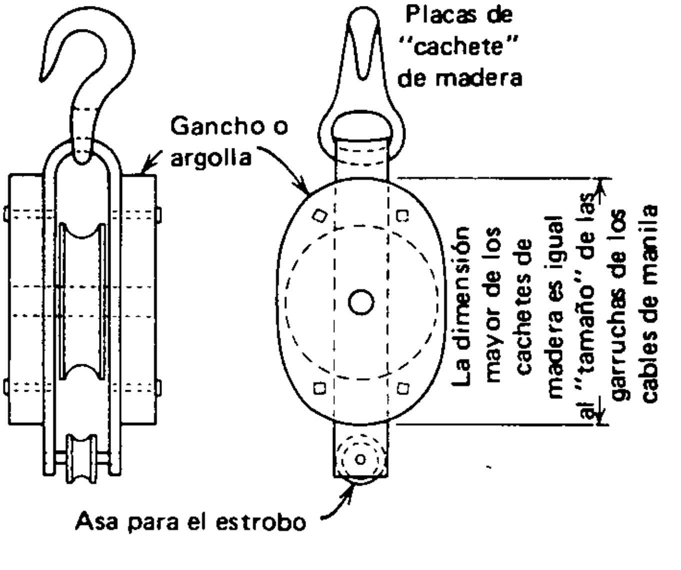


Figura 3.7.13 Garrucha con polea sencilla, para cable de manila.

tipo es más recomendable, ya que permite al obrero quitarse la bolsa con más rapidez cuando tiene que efectuar otras operaciones que no sean las de conectar o atornillar, tales como moverse entre el equipo, o manejar tablones, etc.

Taladro (para madera): neumático; eléctrico, brocas, porta broca. Estas herramientas difieren en el manera en que se insertan las brocas, ya que la forma de las que requieren portabrocas deben ajustarse a la configuración de los extremos de la broca, así como a la máquina misma.

Berbiquí: de carpintero, brocas de.

Hierro para marcar: para la identificación de tablas y tablones.

Escoba.

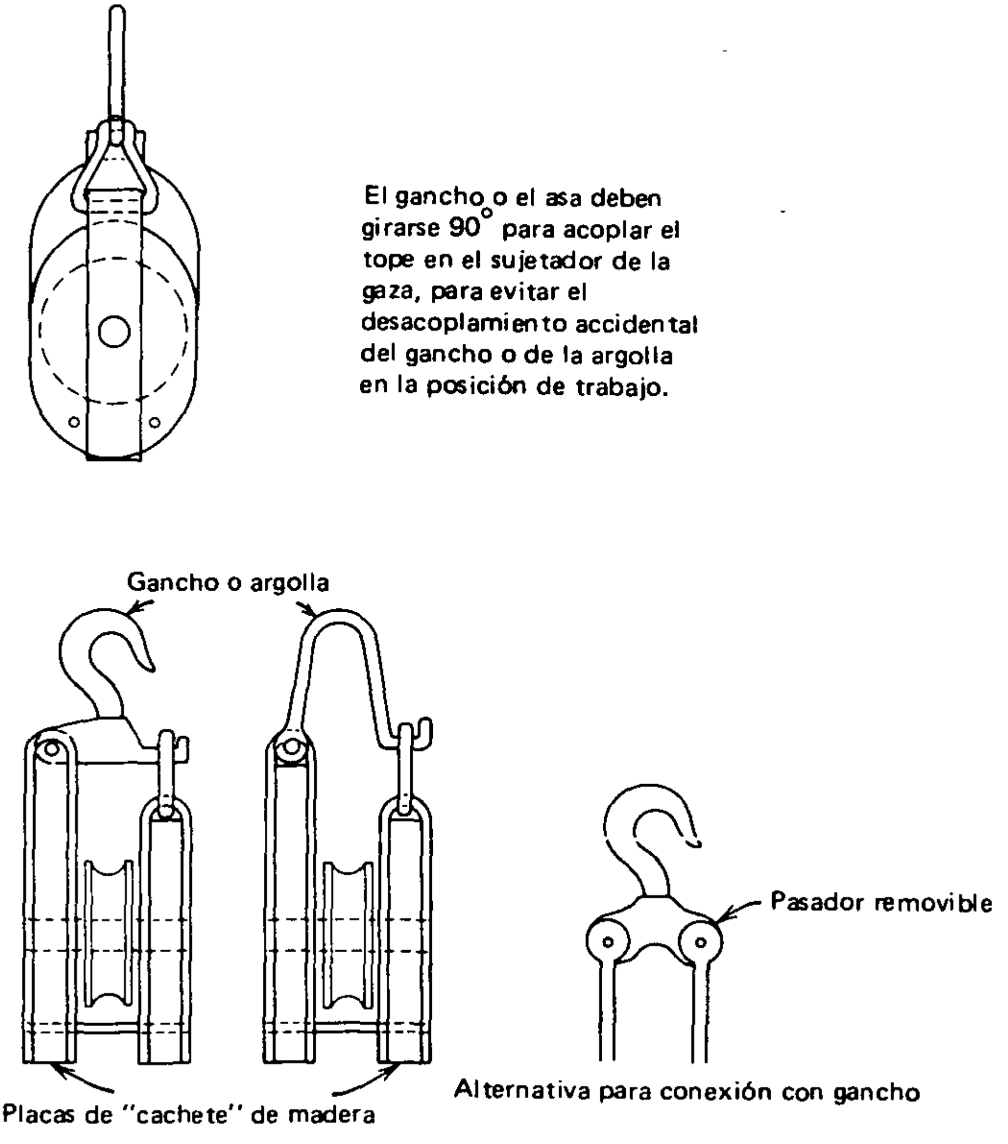


Figura 3.7.14 Garrucha de bisagra, para cable de manila (la garrucha de bisagra para cable de acero es similar, excepto por las placas extremas).

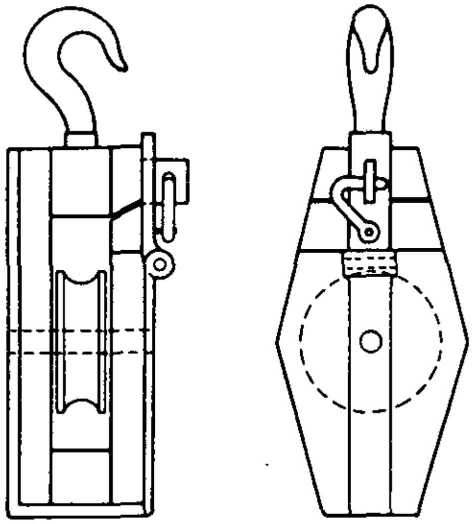


Figura 3.7.15 Garrucha de compuerta para cable de manila.

Cepillos: para pintura, de alambre, de copa, para raspar, circulares. Los cepillos de copa, para raspar, y los cónicos metálicos se utilizan en herramientas accionadas mecánicamente, que al hacer girar el cepillo permiten limpiar la oxidación, la pintura y cualquier otro material extraño al acero.

Cubetas: para pintura, para agua. Un recipiente con grifo y suficientes vasos desechables de papel son más higiénicos que una cubeta de agua y un cucharón común, además así lo exigen las leyes sanitarias en muchas ciudades y estados.

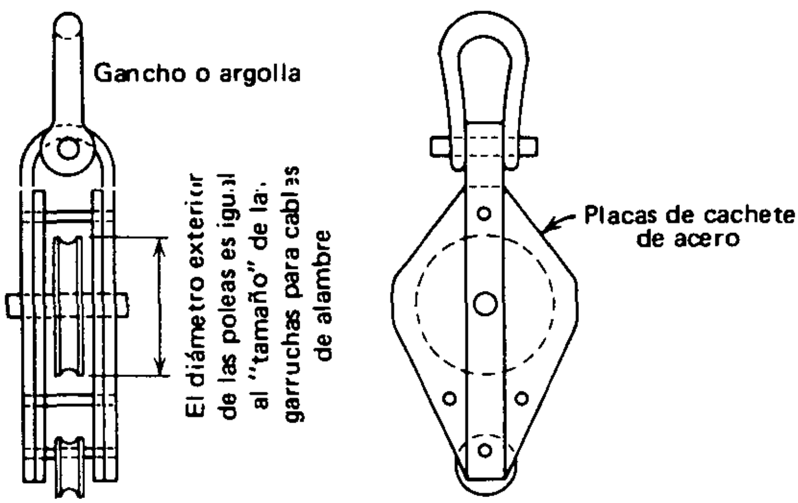


Figura 3.7.16 Garrucha de polea sencilla para cable de alambre.

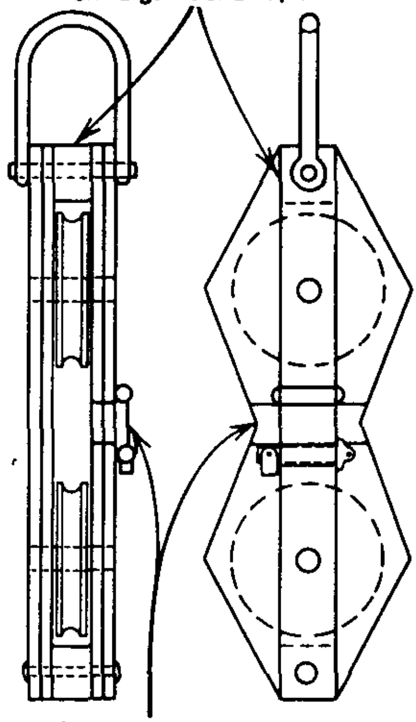


Figura 3.7.17 Garrucha tipo "violín" (tipo "banjo")

Soplete: (para cortar): caja, calibradores, oxígeno, acetileno; reguladores, manguera, oxígeno y acetileno, combinados; coples para manguera; remendador de manguera, encendedor, llave del tanque; boquillas para corte, llaves para soplete, para accesorios.

Rompedor: de mano; (Fig. 3.7.19) útil para cortar las cabezas de los remaches que se van a quitar.

Botes: para gasolina, con seguro. Para llenar los tanques de gasolina del equipo. Todos estos tanques deberán tener un filtro de seguridad contra incendio en la abertura para llenado. En los tanques se deberá pintar una señal de peligro, prohibiendo la extracción de tales filtros cuando se llenan los tanques.

Martillo para cincelar: *Ver* Martillo

Cincel: de mano, brocas (para herramientas accionadas mecánicamente).

Malacate de cajón: un mecanismo patentado, del tipo de malacate de izaje, para jalar un conjunto de piezas con el mínimo esfuerzo.

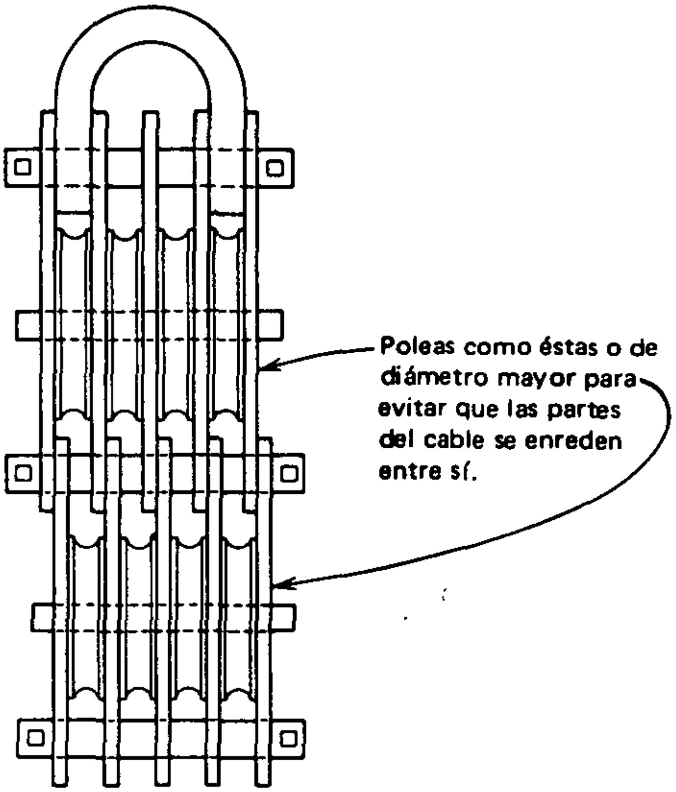


Figura 3.7.18 Garrucha en tándem (para 16 líneas).

Tirador: un dispositivo que se conecta a un cable de alambre y una vez asegurado permite jalar el cable con un polipasto por otro medio.

Compresor: de diesel; de gasolina; eléctrico; de vapor (se usa raras veces). La lista de herramientas deberá especificar el número y la capacidad que se requieren; además, si es necesario, las salidas del múltiple y el tipo de recipiente de aire. La capacidad del compresor deberá ser un poco mayor que la requerida para accionar las herramientas neumáticas que se vayan a utilizar, ya que de esta

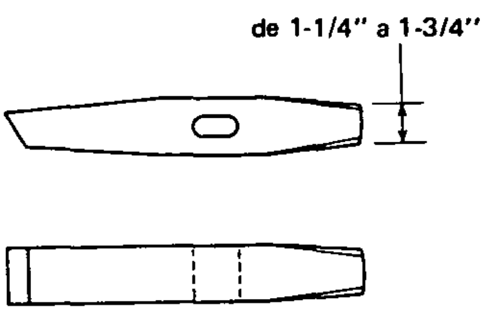


Fig. 3.7.19 Rompedor manual.

manera habrá menos fluctuaciones de la presión de salida, y con un recipiente de aire en la línea, el compresor sufrirá menos interrupciones y por lo tanto "trabajará" menos para obtener la demanda de aire comprimido, todo lo cual prolongará el uso de la máquina. Además, con un exceso en la capacidad de aire, el aire comprimido que permanece en el tanque o recipiente del compresor se enfriará; esto produce un escape de humedad, mejor suministro de aire y mejor funcionamiento de las herramientas neumáticas.

Cabrestante (torno): de contramarcha sencilla (de acción sencilla) (Fig. 3.7.20) de contramarcha doble (de doble acción) (Fig. 3.7.21). Se utiliza con polipastos para cable de alambre con polea sencilla o poleas múltiples, en la que al ejercer una pequeña fuerza sobre las manivelas se produce una fuerza muy grande en la línea de conexión de la cuerda de alambre colocada sobre el tambor. El mecanismo puede sujetarse con abrazaderas o atornillarse a una grúa poste, a un poste guía o a una columna, para usarse cuando el montaje se realiza a mano, en lugar de utilizar equipo accionado mecánicamente.

Grúas: de orugas; diesel, de gasolina, eléctrica, de vapor (se usa raras veces); pluma; brazo; bandas de refacción. Montadas sobre camión: diesel; de gasolina; pluma; brazo; llantas de refacción. Con carro: diesel, de gasolina; pluma; brazo; llantas de refacción. Locomotora: diesel, de gasolina, de vapor; pluma, brazo. Grúa móvil:

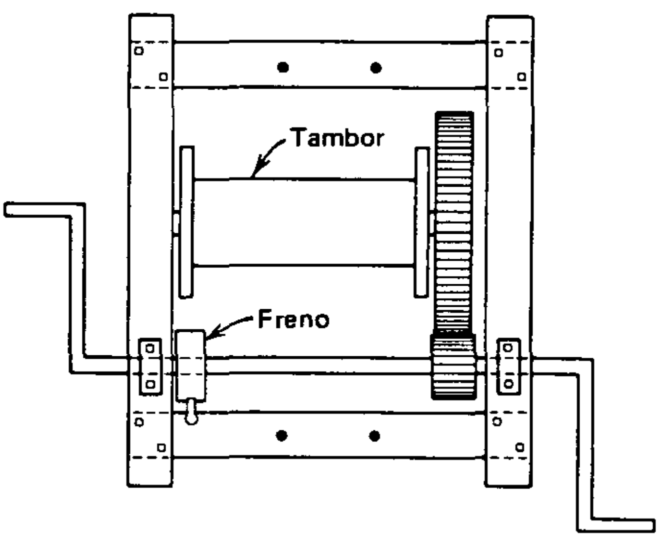


Figura 3.7.20 Cabrestante manual o winch; de acción sencilla.

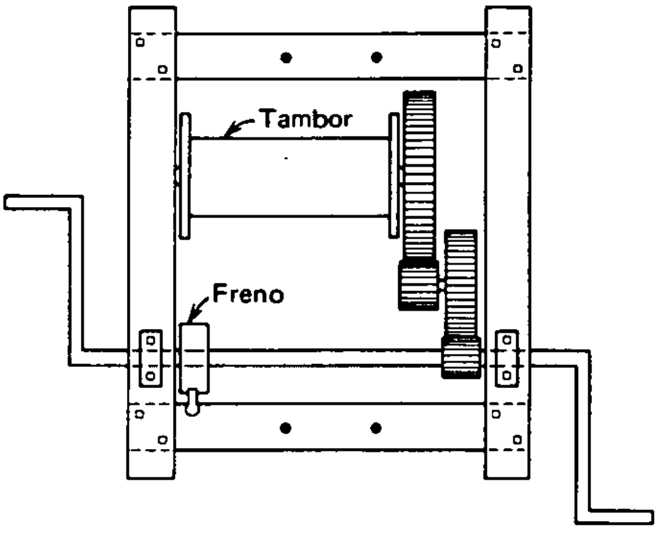


Figura 3.7.21 Cabrestante manual o winch; de acción doble.

diesel, de gasolina, pluma; brazo; llantas de refacción. De torre: diesel, de gasolina, eléctrica; pluma; brazo; torre; llantas de refacción.

Vasos: de papel (*Ver* Comentarios en cubetas).

Cortadoras: de mano; transversales, con punta de diamante, laterales (Fig. 3.7.22); axiales o rectilíneas (Fig. 3.7.23); fresas para equipo accionado mecánicamente. Estas utilizan para cortar material y su forma depende de la configuración del material que se vaya a remover. El extremo cortante puede ser con naríz redonda, plano, para ranurar, para calafatear, con punta de diamante, etc.

Grúa de Torre: marco tipo A: pies derechos delanteros, pie derecho trasero, larguero delantero, larguero trasero, eslabones, pasadores. Atirantada: pluma, mástil, bloque de apoyo, zapata de la pluma, poste principal, rueda impulsora, estrella o araña, eslabones, pasadores. Grúa de pescante ligero: pies derechos delanteros, larguero, pie derecho trasero, eslabones, pasadores. De patas rígidas: pluma, mástil, brazo giratorio, pies derechos traseros, larguero, rueda

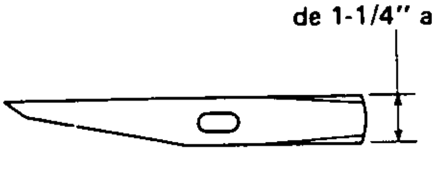


Figura 3.7.22 Cortador lateral.

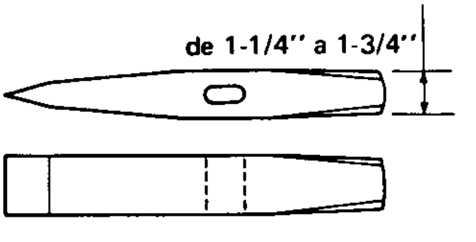


Figura 3.7.23 Cortador recto (con mango corto de nogal).

impulsora, marco, plataforma. Poste guía: mástil, brazo giratorio, contravientos, accesorios, cabria con pies derechos laterales, travesaño, accesorios, cadena de malacate. (Para la descripción de grúas de torre y sus usos, ver los capítulos 7 y 8 sobre grúas viajeras de torre).

Locomotora tipo Dinkey: pequeña máquina locomotora, por lo general de cuatro ruedas, corre sobre rieles estándar, impulsada por un motor eléctrico o de combustión interna; se utiliza para jalar equipo de ferrocarril donde no se dispone de facilidades para instalar una espuela de ferrocarril o cuando ésta resulta antieconómica debido a que se requiere poco servicio.

Perros: para vigas (Fig. 3.7.24); para trabes (Fig. 3.7.25). Los perros para vigas se deslizan sobre el patín superior de ellas y aprietan al producirse un tirón sobre el arillo de conexión al cual están unidos sus brazos en forma de tijera. En el caso de los perros para trabes, sus puntas se enganchan perfectamente al alma, bajo el patín superior de la trabe y donde es posible, las ranuras de las mordazas de los perros se colocan entre los atiesadores o en las cabezas de los tornillos o remaches, para evitar que la trabe se deslice hacia los lados de los perros. Ambos tipos se fabrican en diferentes tamaños, según la capacidad que se requiera. A menudo, los perros para trabes son de acero fundido o forjado. Se deberá colocar una pequeña pieza de madera entre las puntas y la superficie del acero, para obtener una mejor sujección y evitar los movimientos laterales de las trabes dentro de los perros.

Buterola, remachadora: de presión (Fig. 3.7.26); de percusión. En general, operadas neumáticamente. La remachadora de presión tiene un pistón accionado por aire comprimido, que actúa contra un remache que está colocado contra el extremo de la cabeza

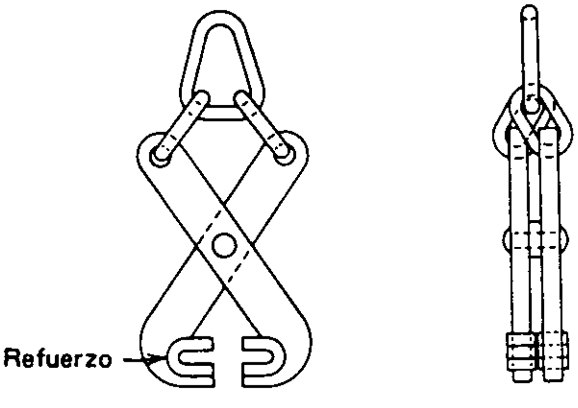


Figura 3.7.24 Perros para viga.

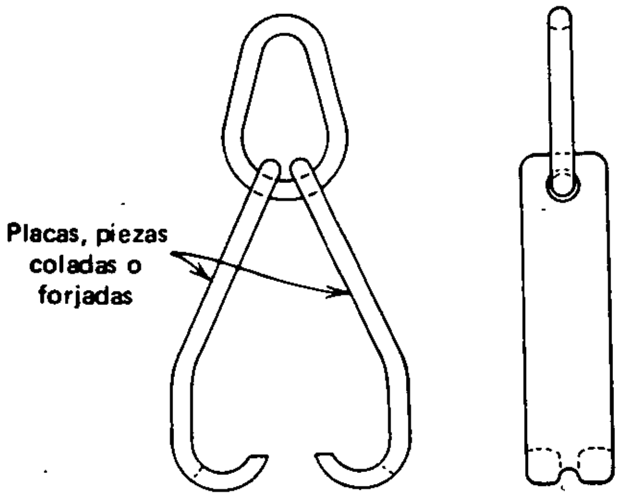


Figura 3.7.25 Perros para trabe.

manufacturada de un remache caliente, colocado en el agujero donde será hincado. En el otro extremo de la remachadora, una barrena con extremo roscado o una preparación para conectar un tubo largo con una barrena en su otro extremo, permite presionar la buterola contra el acero adyacente. Cuando se aplica el aire comprimido, la contraremachadora se sostiene con firmeza contra el remache que se está hincando. La remachadora de percusión es similar a la de presión, excepto que en vez de que se aplique una presión constante a la contraremachadora, el pistón es accionado para golpear repetidas veces contra el extremo de ella, en forma similar al martillo remachador que se utiliza para formar la nueva cabeza. Esta última es más útil cuando se trata de remaches de mayor diámetro y con vástagos más largos. La buterola de presión también se conoce como "sujetadora".

Rodillos para madera: (Fig. 3.7.27) utilizada para rodar madera, tubo, acero, etc. sobre superficies parejas.

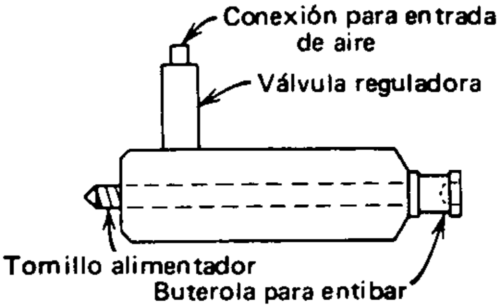


Figura 3.7.26 Remachadora.

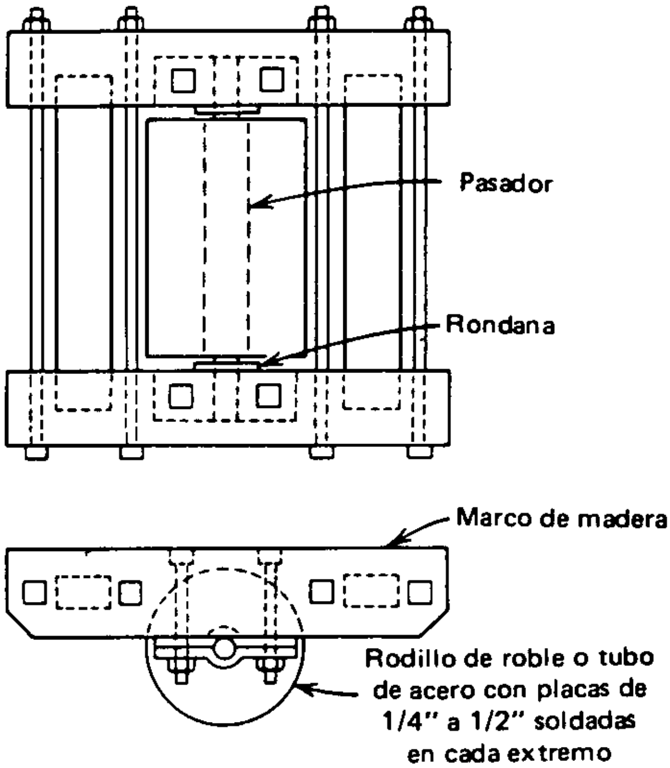


Figura 3.7.27 Rodillo para madera.

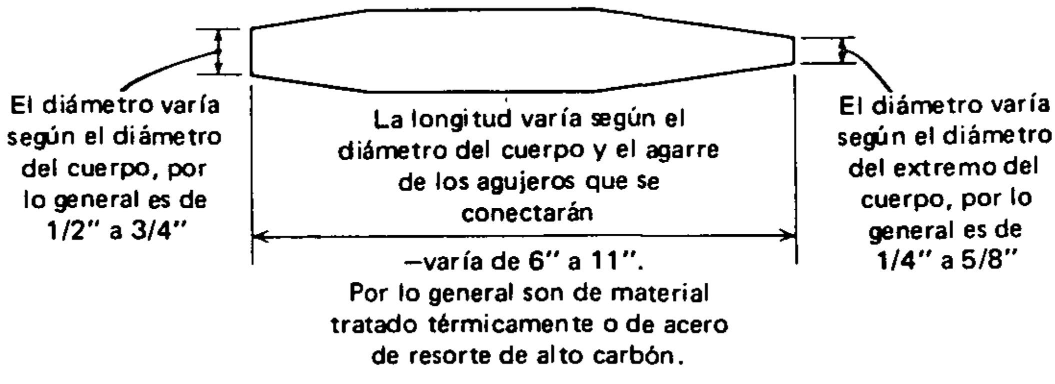


Figura 3.7.28 Pasador ahusado.

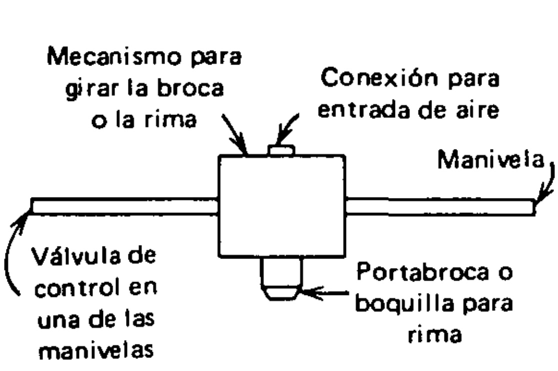


Figura 3.7.29 Taladro con husillo centrado.

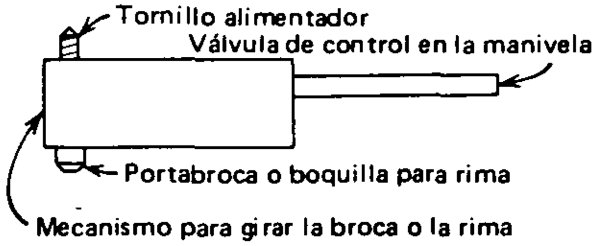


Figura 3.7.30 Taladro con husillo en la esquina.

Pasador ahusado: (Fig. 3.7.28) Estos se fabrican en varios diámetros; el barril tiene del mismo diámetro que los agujeros de las placas que se van a conectar y asegurar con pasadores. La punta se empuja a través de las placas de acero que se van a ensamblar en la conexión y permite alinear las diferentes placas. La cantidad de cada diámetro que se ordene deberá estar basada en un porcentaje del número de agujeros del diámetro respectivo que se van a atornillar, remachar, o en todo caso en la cantidad que se vaya a suministrar para el montaje provisional, cuando se trate de conexiones soldadas. También dependerá de la cantidad de cuadrillas de trabajadores para izaje, montaje, atornillado y remachado.

Taladros: con husillo centrado (Fig. 3.7.29); con husillo descentrado; en la esquina (Fig. 3.7.30); eléctricos, neumáticos, de trinquete manual. El taladro con husillo centrado se utiliza cuando el área de trabajo es accesible y requiere de dos hombres, para sujetarlo y colocar con seguridad la broca en su lugar. Los taladros con husillo descentrado y en la esquina se utilizan en donde un hombre puede manejar la máquina con seguridad; este último donde no exista suficiente espacio para usar los otros tipos. En los taladros con

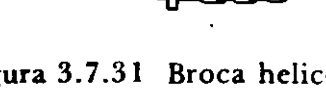


Figura 3.7.31 Broca helicoidal.

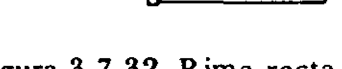


Figura 3.7.32 Rima recta.

Figura 3.7.33 Rima de sección variable.

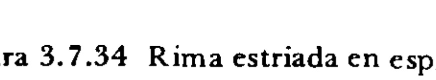


Figura 3.7.34 Rima estriada en espiral.

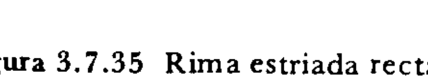


Figura 3.7.35 Rima estriada recta.

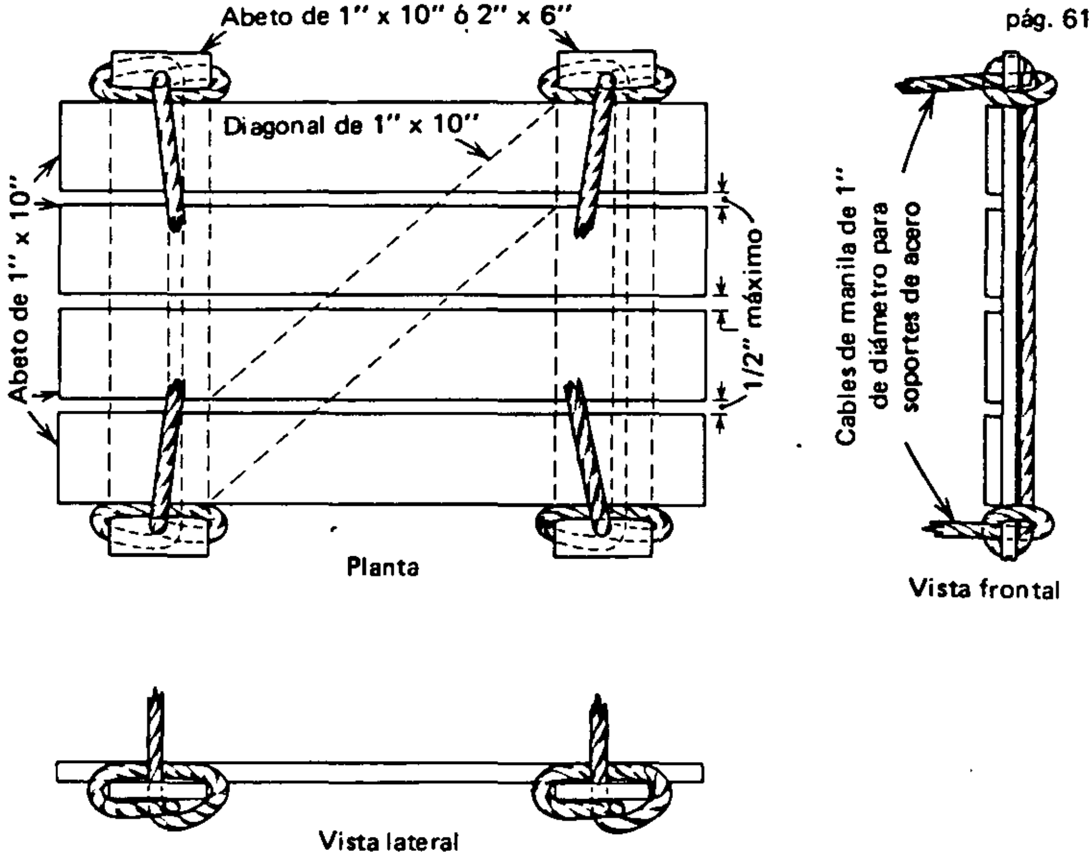


Figura 3.7.36 Andamio de tablones. Se prefiere usar las piezas de 1" X 10", atornilladas a los soportes con tornillos de coche, o bien clavados, doblando los clavos por debajo.

trinquete manual, la broca se hace girar accionando la manivela hacia atrás y hacia adelante, lo cual presiona el trinquete contra un engrane, para hacer girar la broca del taladro.

Brocas: para acero; para madera; helicoidales (Fig. 3.7.31); rimas "puente" para acero; cónica, recta (Fig. 3.7.32); de sección variable (Fig. 3.7.33); rimas "estriadas": para acero; en espiral (Fig. 3.7.34); rectas (Fig. 3.7.35), de sección variable, helicoidales, etc.; portabrocas para rimar y taladrar enchufes.

Tambores: para aceite; para agua.

Cables eléctricos: conectores; interruptores, transformadores; alambres.

Esmeriladoras: de motor; de mano; con rueda de repuesto.

Obra falsa: de acero; de madera; placas de conexión (cubrejuntas); rondanas. El material para la obra falsa se diseña para las necesidades y casas específicos y se fabrica en el lugar de la obra o en el almacén. En la lista de materiales se deberán incluir todos los aditamentos que se vayan a suministrar.

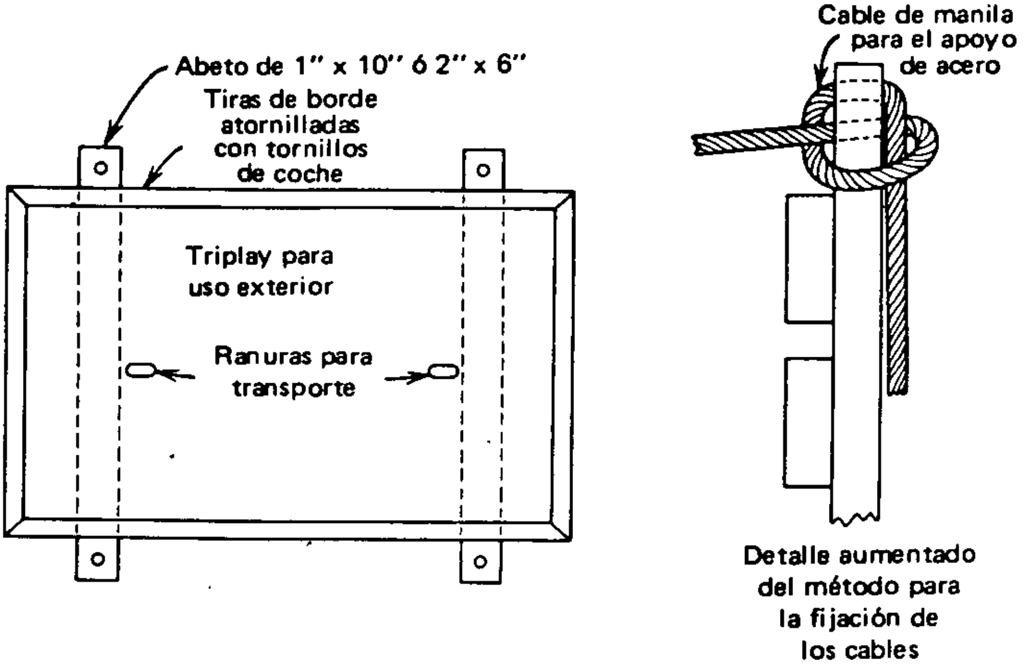


Figura 3.7.37 Andamio de triplay. Los cables se fijan de la misma manera que en los andamios de tablones.

Limas: de mano.

Extinguidores: la clase y el tipo de los extinguidores dependerá de los riesgos que se espere encontrar. Se deberán instalar extinguidores de mano en las cabinas de todos los camiones y grúas, así como también en todos los sitios donde se almacene equipo eléctrico o de combustión interna.

Andamios: de tablones (Fig. 3.7.36); de triplay. Algunos montadores prefieren los andamios de triplay, mientras que otros los prefieren hechos de tablones espaciados entre sí. Ambos tipos se cuelgan de la estructura y sirven para que los atornilladores, remachadores, soldadores, montadores, etc. trabajen en los lugares que no son accesibles desde la estructura misma.

Poste grúa: de acero, de madera, zapata para; empalmes.

Gafas, de seguridad: para todo uso: claras, oscuras; para cortar con soplete: con cubierta clara, oscura; tipo copa: clara oscura; para soldar, claras, oscuras, con pantallas laterales, sin pantallas laterales; para destellos de soldadura: oscura.

Grasa.

Pistola engrasadora: tipo Dot; tipo Zerk; tipo Alemite; de bomba.

Esmeriles: eléctricos, neumáticos, manuales; con rueda de repuesto.

Rueda para esmeril: manual.

Martillo, manual: cincelador (Fig. 3.7.38), de uña; un marro (Fig. 3.7.60), cincelador (Fig. 3.7.39), mandarria, manual (Fig. 3.7.40).

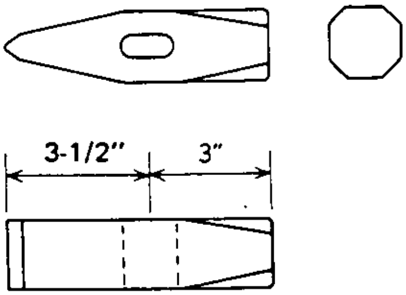


Figura 3.7.38 Martillo cincelador manual.

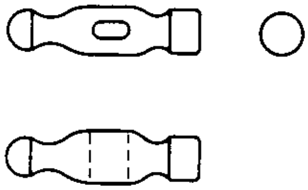


Figura 3.7.39 Martillo cincelador.

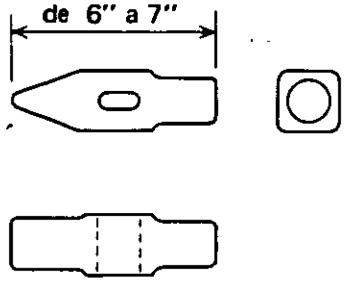


Figura 3.7.40 Martillo manual.

Martillo mecánico: cincelador, eléctrico, naumático; remachador; estándar, con pistón defasado, eléctrico, neumático (Fig. 3.7.41); pistones de refacción. Los martillos para cincelar son similares a los de remachar, excepto que los primeros están diseñados para accionar un cincel o broca más pequeño que la contraremachadora, para un mayor diámetro. En general, el martillo para remachar con pistón defasado tiene un mango invertido para reducir su longitud total.

Mangos: de azuela; de hacha; de botadora de remaches de mano; de rompedoras; de martillo para cincelar; de cortadora; de martillo; de mazo; de piqueta; de marro.

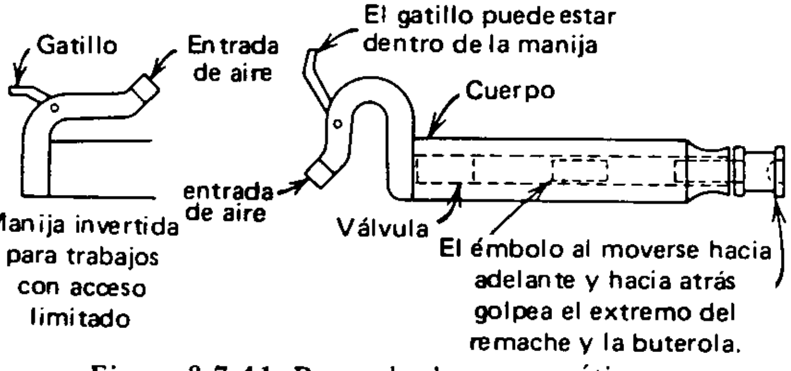


Figura 3.7.41 Remachadora neumática.

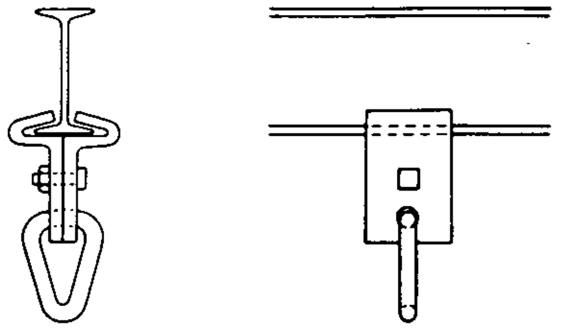


Figura 3.7.42 Gancho para viga.

Hachuela.

Cascos: *Ver* Cascos de seguridad.

Gancho para viga: de viga (Fig. 3.7.42); para colocar columnas (Fig. 3.7.43). El gancho para viga se sujeta al patín inferior de la viga, y de él se sostiene un polipasto que casi siempre se utiliza para manejar las cargas ligeras que se presentan cuando las cuadrillas de detalle montan piezas pequeñas que las cuadrillas de montaje han dejado pendientes.

El gancho sujetador para colocar columnas se utiliza para girar una columna de su posición horizontal, de descarga, a una posición vertical para su montaje. Además, elimina la necesidad de que un operador suba por la columna, una vez ya colocada en su lugar, para quitar la eslinga. También permite sostener la columna más vertical que lo que se lograría utilizando una eslinga para montarla. Además, esto hace que el montaje de la columna sea más fácil, seguro y rápido.

En las placas de empalme del extremo superior de la columna se deberán proveer agujeros de 2 a 2–1/2 plg. o mayores, según el peso que se vaya a levantar y del diámetro del pasador de izaje. Se insertan un par de argollas sobre las placas de conexión y después se aseguran con un pasador largo de acero que pasa a través de los agujeros barrenados en ellas; luego, las dos argollas se conectan a su vez por medio de eslingas a otra argolla sencilla colocada en la parte superior. Para evitar que el pasador se caiga después, se asegura al ojal superior de una de las eslingas principales por medio de una pequeña argolla colocada en el extremo del pasador. Si se prefiere, la eslinga protectora de izaje se puede asegurar directamente a la argolla del gancho de izaje.

El cable guía se fija a la argolla pequeña colocada en el extremo del pasador y una vez que la columna ha sido montada en su sitio, asegurada con pasadores y remachada, se baja ligeramente el gancho sujetador y con un rápido tirón en el cable guía, en la posición más cerca a la horizontal que el lugar permita, el pasador saldrá con facilidad. Esto desengancha las argollas de izaje y desconecta el gancho sujetador de la columna.

Malacate manual: *véase* malacate (winche).

Malacate mecánico: diesel; eléctrico; de gasolina; neumático; de vapor; de tractor; tipo Tugger de tambor sencillo, de tambor doble, de tambor triple, etc.; con mecanismo giratorio; separado para conectarlo al malacate principal. Los accesorios y las especificaciones del malacate, como son el número, tipo, caballos de potencia, cantidad de tambores, tipo de mecanismo giratorio, etc., se deberán definir con base en la potencia de izaje que se requiera. A su vez, éste depende de la velocidad del cable, así como de la longitud máxima del cable que se tiene con el tambor lleno y del diámetro del cable de alambre, así como también del paso del cable en el tambor.

La altura a la que la carga se vaya a elevar está en relación con el tiempo del izaje; mientras más partes haya en las líneas, más lento

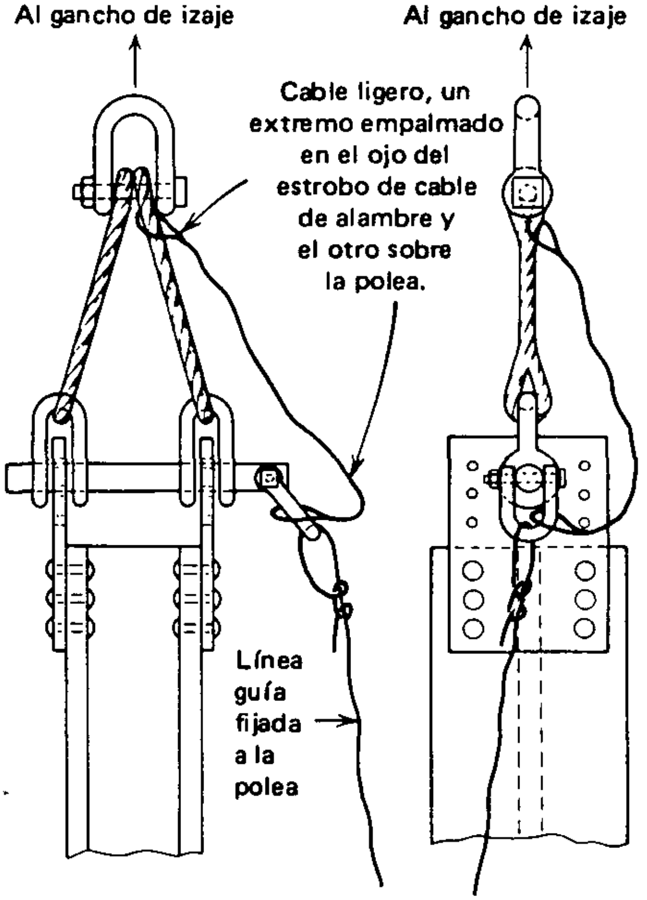


Figura 3.7.43 Gancho para colocar columnas.

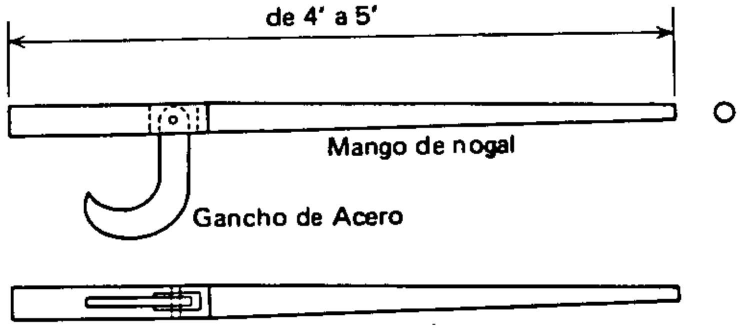


Figura 3.7.44 Gancho de volteo.

será el izaje y por lo tanto se necesitará más cable en el tambor. Pero en cambio, la fuerza de izaje requerida es menor que con menos partes para la misma carga. Si un malacate tiene controles de aire, deberá prevenirse al personal de campo para vigilar la presión y evitar sobrecargas en los controles.

En general, las fricciones se ajustan regulando la presión en las válvulas de control, usando de 40 a 50 libras por pulgada cuadrada para trabajo pesado y de 20 a 25 libras por pulgada cuadrada para trabajo ligero. El ajuste del freno debe ser tal que éste soporte la

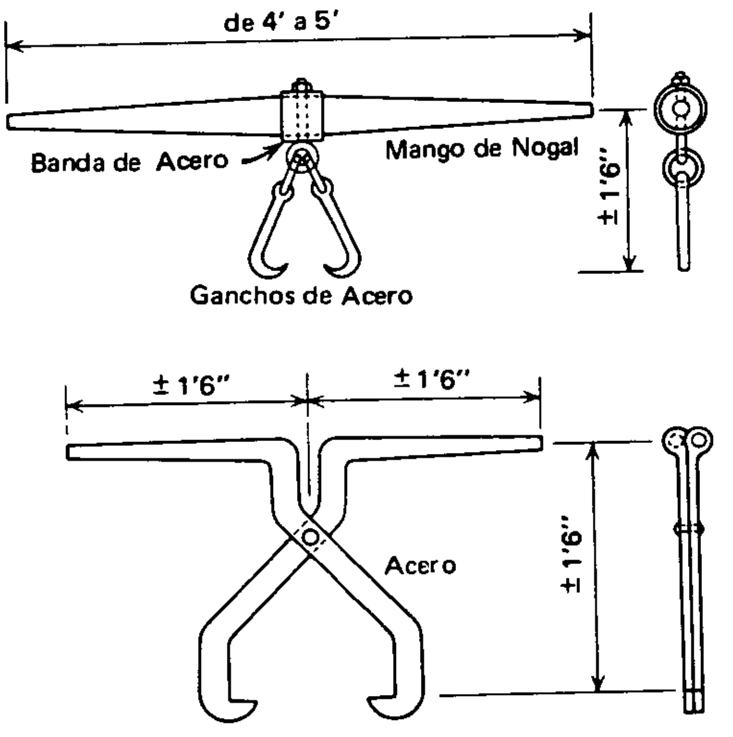


Figura 3.7.45 Ganchos para acarrear madera.

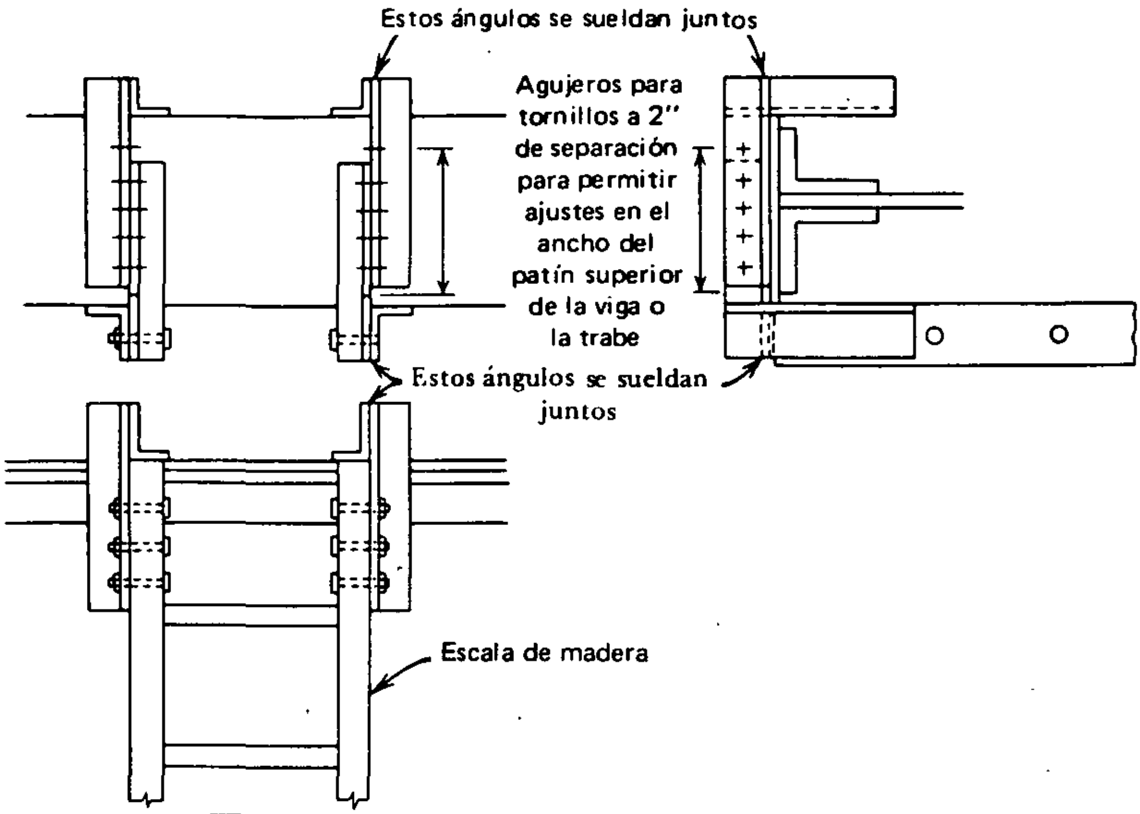


Figura 3.7.46 Gancho ajustable para madera.

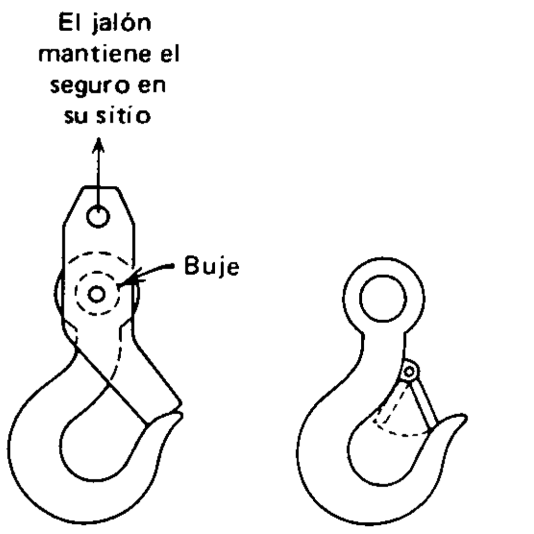


Figura 3.7.47 Ganchos de seguridad.

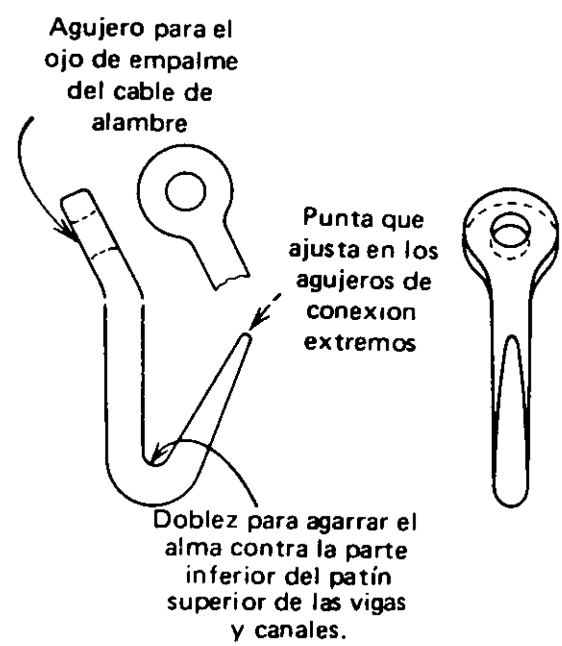


Figura 3.7.48 Ganchos para selección.

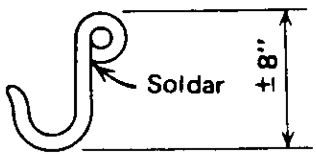


Figura 3.7.49 Gancho para línea.

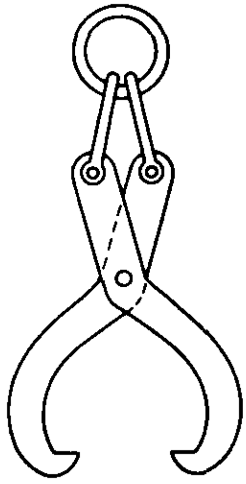


Figura 3.7.50 Gancho para madera.

carga máxima con el pedal hundido hasta la midad. Si los perros del tambor, están trabajando bien, deberán hacer contacto total con los dientes del trinquete cuando se aplican y deberán soltarse por completo cuando se retiran.

El malacate tipo Tugger es un mecanismo pequeño de un solo tambor que puede sujetarse a una columna o a una viga y, en general, se acciona neumáticamente. Está adaptado para utilizarse con cable de alambre de diámetro pequeño, para el montaje de piezas pequeñas, en lugares donde un malacate grande sería innecesario y antieconómico. Un malacate de tractor es un malacate de un solo tambor montado en la parte posterior de un tractor, para operar líneas de capacidad ligera, pero donde se requiera una movilidad excesiva para utilizar el malacate en diferentes lugares.

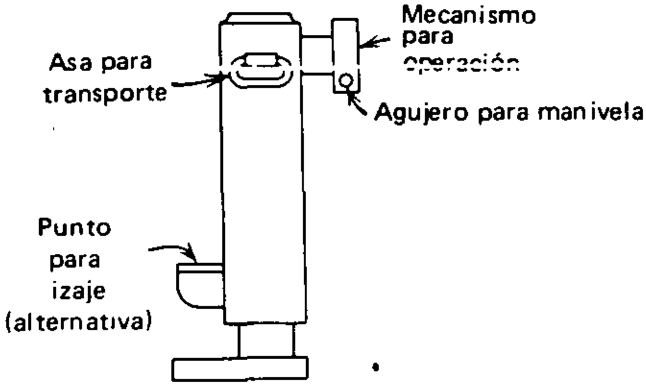


Figura 3.7.51 Gato de puente.

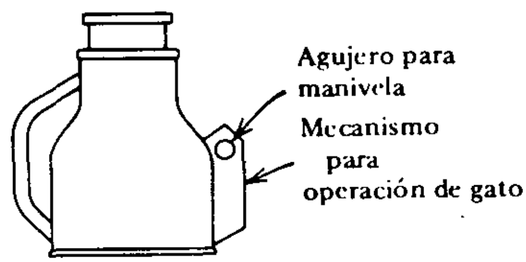


Figura 3.7.52 Gato de "botella".

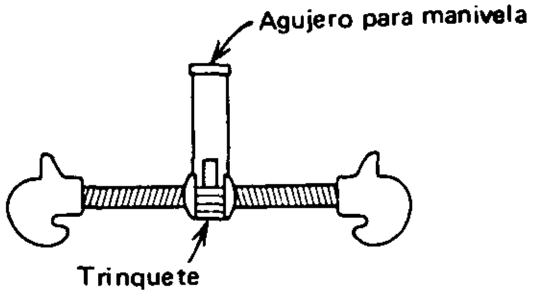


Figura 3.7.53 Gato de "jalar y tirar".

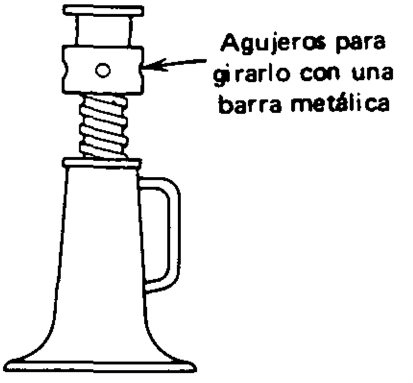


Figura 3.7.54 Gato de tornillo.

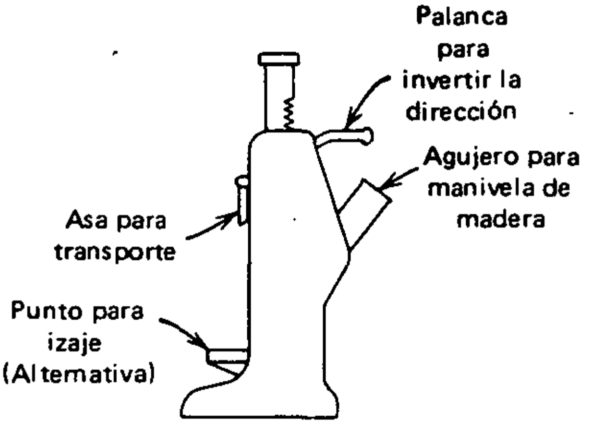


Figura 3.7.55 Gato de vía.

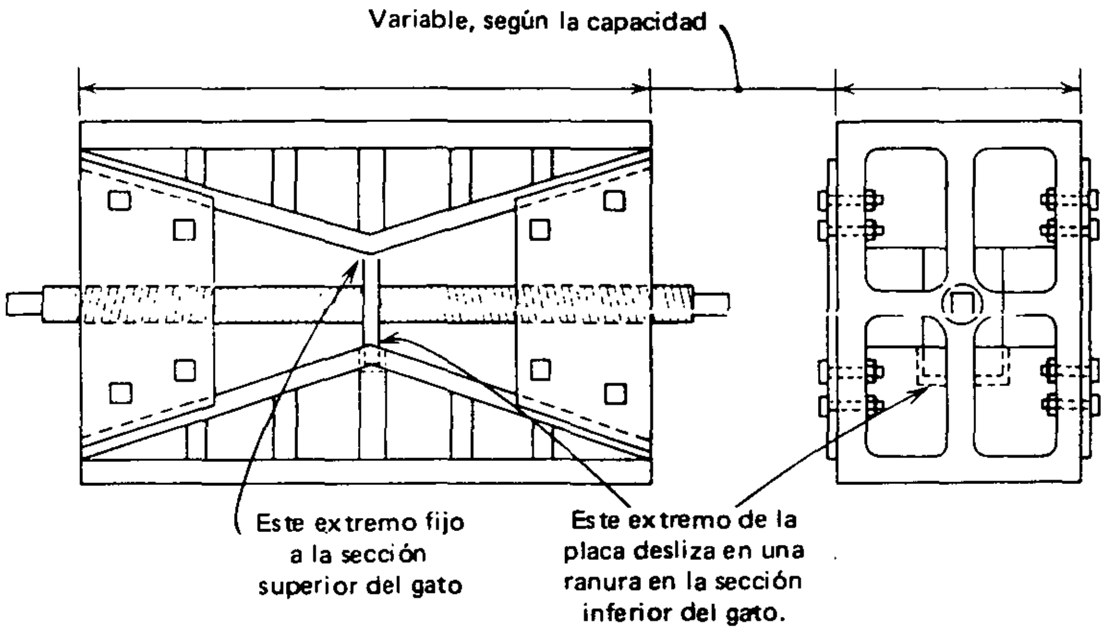


Figura 3.7.56 Gato de cuña.

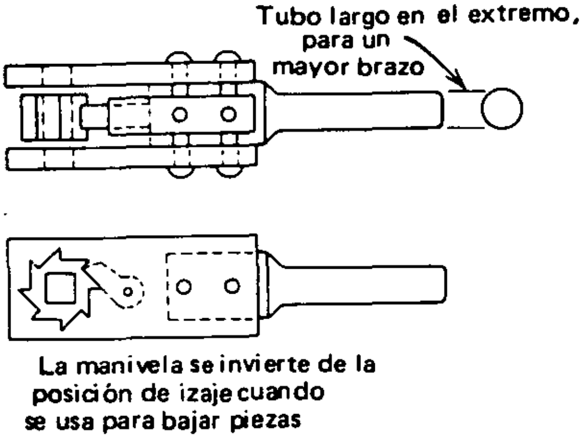


Figura 3.7.57 Manija de un gato de cuña.

Vigas de izaje: de acero; de madera.
Crayón: para marcar.
Escaleras: rectas: de acero, de madera; con extensión; con ganchos.
Linternas: de luz roja; luz clara; focos de repuesto
Lanchas.
Nivel: de burbuja.
Salvavidas: aros, chalecos.
Llaves y cerraduras.
Plataformas: de madera (Figs. 3.7.58 y 3.7.59).

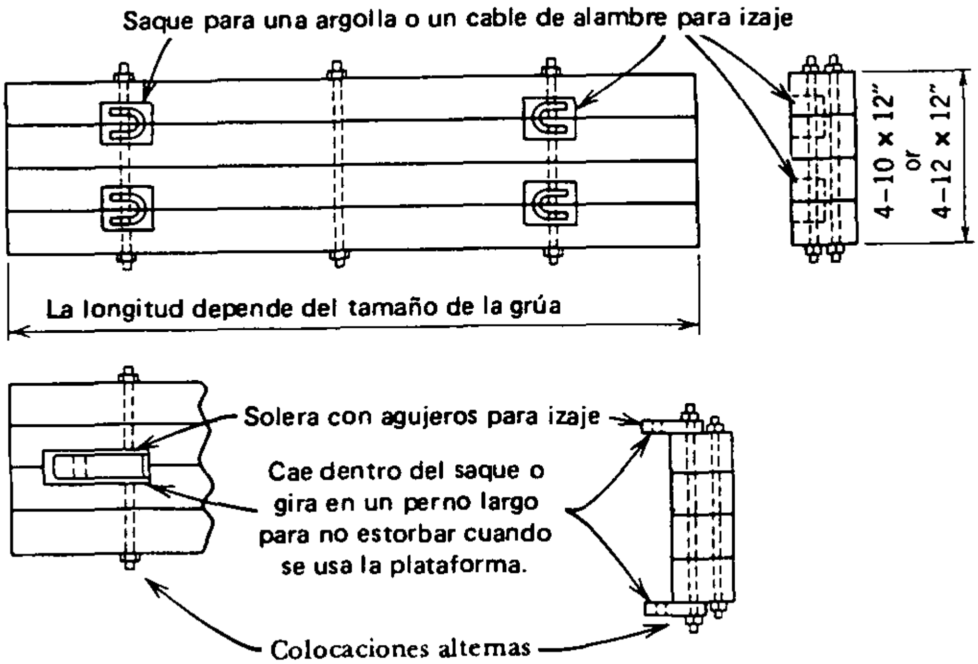


Figura 3.7.58 Plataforma grande.

Ganchos: de volteo (Fig. 3.7.44); para acarrear madera (Fig. 3.7.45); de escalera (ajustable) (Fig. 3.7.46); de seguridad (Fig. 3.7.47); para andamios; para clasificar (Fig. 3.7.48) línea guía (Fig. 3.7.49); para madera (Fig. 3.7.50) Se utiliza un gancho de volteo para acarrear y rodar madera, y un gancho para dos hombres, para llevar madera de un lugar a otro. Un gancho ajustable para escalera permite sostener una escalera corta de madera de la parte superior de una trabe, de una viga o de una armadura de gran peralte, con el fin de que los atornilladores, conectores y ajustadores, etc. alcancen con seguridad y facilidad las conexiones y el patín inferior. El gancho de seguridad cuenta con un pestillo o aldaba para evitar que el lazo del gancho se salga, hasta que éste no se levante o se baje, según el caso.

El gancho para clasificar tiene una punta que puede deslizarse dentro de los agujeros extremos de conexión de las vigas, canales, etc., así como también una parte doblada, diseñada para sujetar el extremo del alma de una viga contra la parte inferior del patín superior. El gancho del cable-guía se conecta al extremo de la carga que se va a manejar, o de la pieza que se vaya a montar, para ayudar a mantener la carga libre de obstrucciones o para guiar la pieza a los conectores que la esperan arriba. El gancho para madera se utiliza para levantar maderos, con equipo mecánico, clavando las puntas del gancho en la madera para sujetarlo con más rapidez.

Manguera: para aire: 1/2 plg de diámetro, 3/4 plg, 2 plg; para vapor; para agua; conectores; empaques, válvulas: Cleco, Thor, de acción rápida, etc.

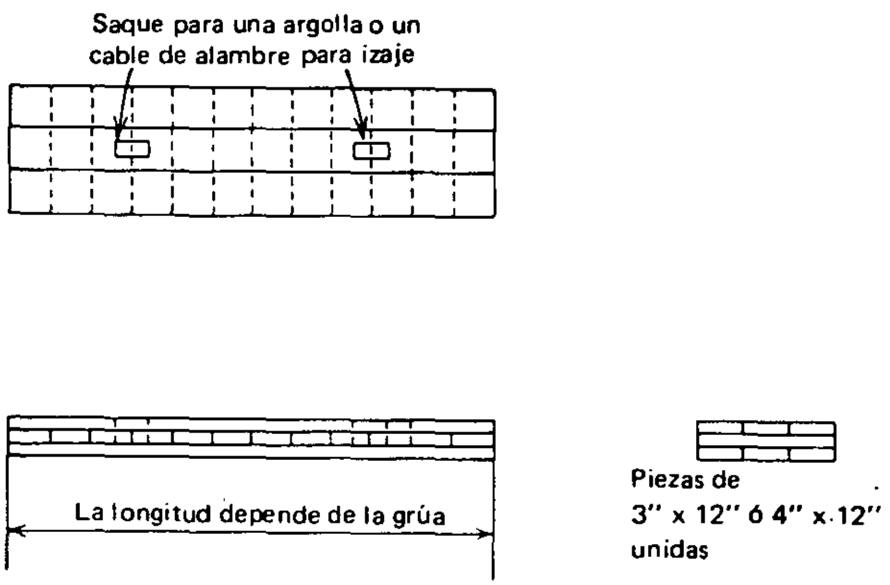


Figura 3.7.59 Andamio pequeño.

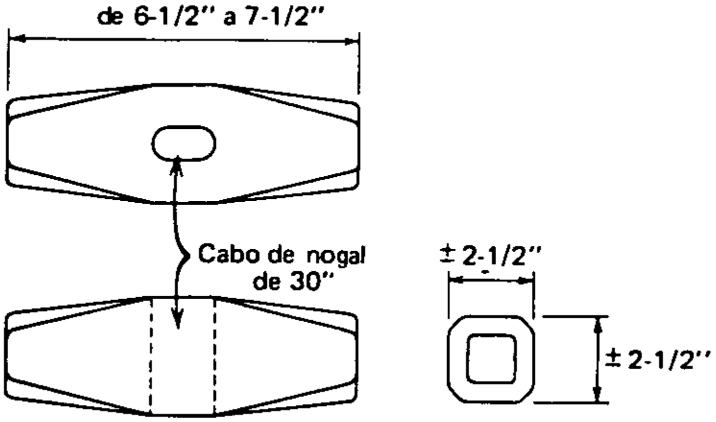


Figura 3.7.60 Marro.

Las mangueras de aire se deberán revisar con cuidado para asegurarse de que están en buenas condiciones y pueden resistir la presión que se les va a aplicar, con suficiente margen.

Remendador de manguera: alambre para remendar.

Llave de impacto: eléctrica; neumática; de acumulador; casquillos de. La llave de impacto hace girar un casquillo para remachar, u otro accesorio, por medio de una serie de impactos neumáticos rotatorios. Se recomienda para apretar tornillos de alta resistencia ya que permite un mejor control del apriete de tales tornillos.

Gatos: de puente (Fig. 3.7.51); hidráulico, accesorios, manómetros, tubería, tipo de tornillo o en forma de botella (Fig. 3.7.52); de

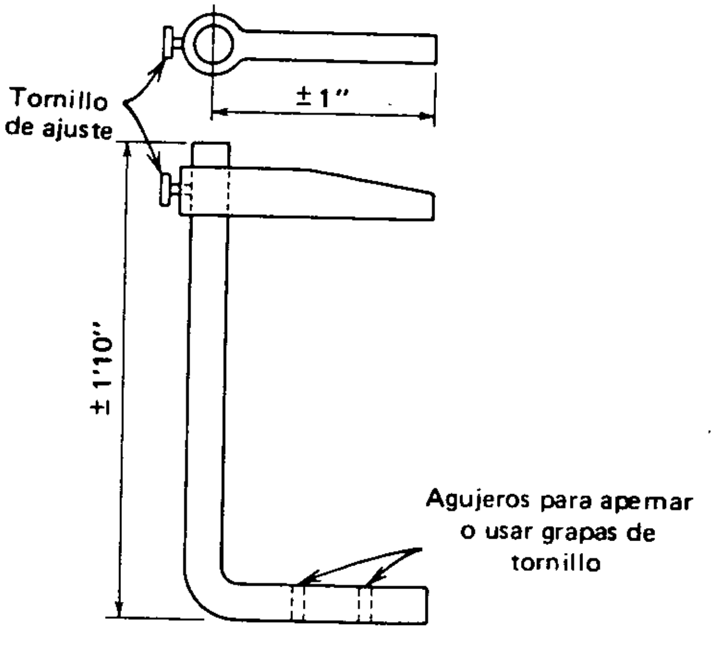


Figura 3.7.61 Abrazadera para taladrar.

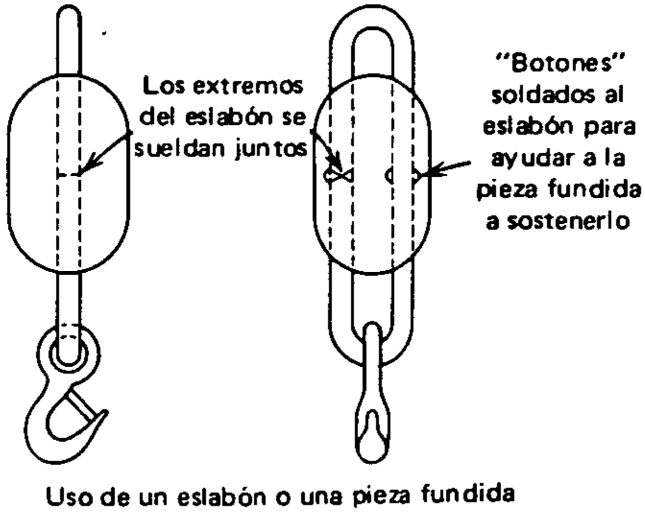


Figura 3.7.62 Contrapeso: de una pieza.

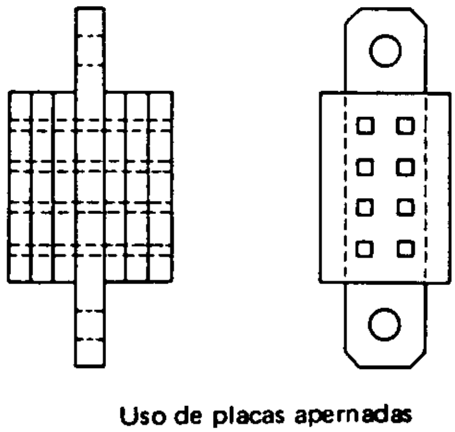


Figura 3.7.63 Contrapeso: ensamblado.

tracción o "de jalar y tirar" (Fig. 3.7.53). Ver también trinquete de manguito; de tornillo (Fig.3.7.54); de vía (Fig. 3.7.55); de cuña (Fig. 3.7.56); de cuña con mango (Fig. 3.7.57).

Mazo: (Fig. 3.7.60) Se usa para empujar pasadores dentro de los agujeros de conexión, para colocar miembros de acero en su lugar, para enderezar material doblado y en dondequiera que se necesite una fuerza que pueda obtenerse por medio de una acción de golpeo. El mazo más común utilizado por los montadores tiene una cabeza de 8 lbs. de peso y un mango de 30 plg. de largo.

Aceite: para martillo neumático, cilindros de; para motores, etc.

Latas de aceite: rectas; de presión.

Abrazadera para taladrar: (Fig. 3.7.61). La base se sujeta o atornilla a la pieza donde se va a taladrar el agujero. El brazo se ajusta a lo largo del taladro y de la broca, con el tornillo de avance en posición retractada. A medida que el taladro penetra dentro del material, el tornillo de avance de la máquina se aprieta contra el brazo de la abrazadera hasta que el agujero ha sido barrenado por completo.

Contrapesos: ligeros; pesados; de una sola pieza (Fig. 3.7.62); ensamblado (Fig. 3.7.63). El contrapeso o "bola" se utiliza para auxiliar en el movimiento de las líneas principales o auxiliares y ayudar a bajar la carga y el gancho de izaje una vez que se ha izado la carga y se ha liberado el gancho. El peso deberá vencer la fricción de las diferentes poleas de la garrucha, así como también la de cualquier polea secundaria o de guía sobre la que corre la cuerda. Las bolsas de contrapeso o pesos separadores normalmente pesan de 500 a 1000 lbs o más, según se requiera.

Cuando se emplea una grúa en un edificio alto y se requiere bajar (acarrear) la polea de carga hasta el piso, para recoger una carga, se necesita un sobrepeso muy grande para vencer el peso del cable-guía comprendido entre el malacate situado en la parte baja y el extremo superior de la pluma, a través del pie del mástil, además de la fricción de las diferentes poleas. Si el peso separador no es lo suficientemente grande para su propósito, el peso del cable guía puede ser lo bastante grande para jalar todas las poleas de carga. Cuando se utiliza una grúa móvil con una pluma demasiado larga, puede presentarse el mismo problema ya que la bola deberá bajar la polea de carga y el gancho de izaje hasta el piso, después de que la carga ha sido elevada a la altura máxima de la pluma y descargada en el nivel superior.

Con mucha frecuencia, cuando se tiene una bola sin el suficiente peso, los trabajadores se ven forzados a jalar el cable-guía hacia arriba, con el fin de separar las diferentes partes de las líneas y permitir así que baje la polea de carga. En este caso los trabajadores realizan un trabajo que debería ser realizado por la bola separadora.

Los contrapesos pueden ser de acero forjado o fundido, en forma de bolas redondas o alargadas, con un eslabón soldado, forjado o fundido dentro de ellas, con unas asas sobresaliendo de la parte superior e inferior de la bola. El eslabón transmite la carga al gancho de izaje a través del contrapeso, y el metal que rodea el eslabón sirve sólo como un peso que ayuda a levantar las líneas de izaje; antes de fundir el eslabón puede colocarse un gancho en su parte inferior. Es más aconsejable usar el tipo de contrapeso sin el gancho, ya que hay ocasiones en que las eslingas y otros tipos de colgadores deben fijarse directamente al asa inferior. Hay otro tipo que tiene pesos atornillados entre sí a lo largo de una placa con agujeros en sus extremos superior e inferior, a los cuales se enganchan la garrucha inferior de carga y el gancho de izaje respectivamente.

Cuando existen muchas partes en las líneas de carga, se puede utilizar una garrucha pesada como garrucha inferior de carga, para reducir el tamaño y peso del contrapeso. Estas son tan sólo garruchas para cable de acero, con placas gruesas atornilladas a cada lado, en lugar de las placas delgadas comunes que se encuentran en las garruchas ordinarias.

Pico.

Piloteadora: neumática; eléctrica; de vapor; de combustión interna; de gravedad; vibradora; sónica; de ariete hidráulico; guías; martillo; puntas; anillo; gancho; extractor.

Pilotes: de acero; de madera.

Pasadores: de alineación (Fig. 3.7.64); de posición. El pasador de alineación lo utilizan los montadores para colocar materiales pesados en su lugar, empujando el pasador de alineación a través de los agujeros correspondientes de la conexión. Después se colocan los pasadores de posición en los agujeros restantes, mientras que el pasador de alineación se retira para usos futuros.

Tubería: para aire, para agua; boquillas; coples; codos; accesorios; niples; tapones; reductores; tes; tenazas, válvulas; prensa de tornillo.

Cortadora de tubo: terraja y dados.

Cuerda para plomeo: de alambre; ganchos ("pata de cabra") (Fig. 3.7.65); accesorios; plomada o peso; placas (Fig. 3.7.66).

Bombas: diesel; eléctricas; de gasolina; neumáticas; de vapor; manuales; para agua; para gatos hidráulicos, para gasolina.

Punzones: marcador (Fig. 3.7.67); de tornillo (Fig. 3.7.68); punzones y dados. El punzón marcador se utiliza para hacer una pequeña incisión como guía, para que el taladro comience a formar el agujero en la posición correcta, y también para marcar líneas de centro y otros puntos de localización en la estructura. El punzón de tornillo manual se utiliza para punzonar agujeros en materiales muy delgados, en donde sería antieconómico emplear un taladro muy potente o un taladro de trinquete manual.

Carro rodante de ferrocarril: una plataforma pequeña con cuatro ruedas de ferrocarril para vía estándar, para rodar sobre vías de ferrocarril. Esta se utiliza para transportar piezas pequeñas, vigas de acero, etc., sin necesidad de hacerlo en los carros de ferrocarril. El carro puede empujarse a mano o jalarse por medio de un polipasto o un calbe conectado a un equipo motríz.

Rieles de ferrocarril: planchuelas; clavos; grapas; tornillos de conexión; tornillos de gancho; rondanas; placas de cambio; durmientes; escantillón; juego de ruedas.

Rimas: ver brocas.

Recipientes de aire: se debe revisar que el recipiente proporcionado cumpla con los requerimientos legales del estado y/o la ciudad en donde se va a utilizar.

Respirador: si se va a quemar, soldar, cortar, calentar, etc., en un espacio cerrado, o sobre un material que al calentarse produzca gases tóxicos, deberá suministrarse un respirador adecuado como

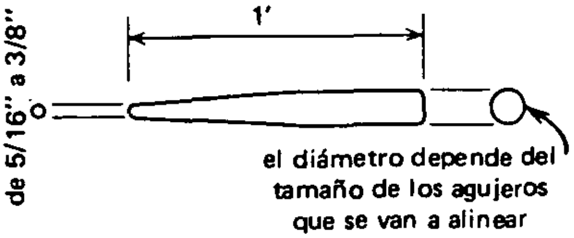


Figura 3.7.64 Pasador de alineación.

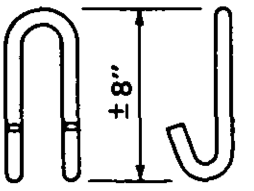


Figura 3.7.65 Gancho para plomeo (pata de cuervo).

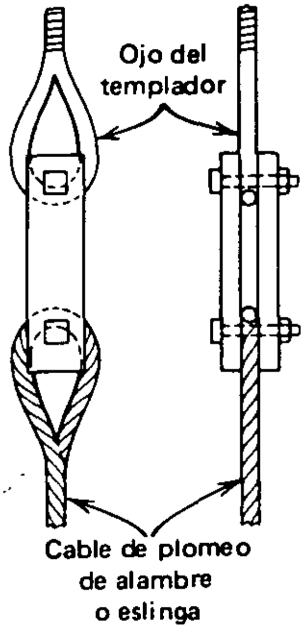


Figura 3.7.66 Placas para plomeo.

Figura 3.7.67 Punzón marcador.

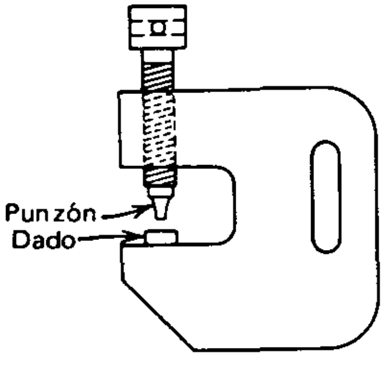


Figura 3.7.68 Punzón de tornillo.

Figura 3.7.69 Anillo de conexión.

protección contra los gases que se producen. Este respirador puede ser una simple mascarilla con filtros y/o tanques adecuados como parte de ella, o un respirador con línea de aire. El alimentador de la línea de aire debe localizarse de tal manera que los gases dañinos no sean aspirados junto con el aire suministrado al respirador.

Anillo, de conexión: (Fig. 3.7.69) una argolla con forma y diseño tal que permita levantar en un punto, dos o más eslingas de sujeción conectadas a la pieza que se está izando.

Corta remaches: (*Ver también* Rompedor manual) neumático; punzón de mano; cincel; retén de cincel; resorte de cierre; con amortiguador de hule; camisa: superior, inferior; brocas; con punta de diamante, etc.; "Hell-dog". La cortadora neumática de remaches ordinaria es lo bastante pequeña para que la maneje un solo hombre y puede romper las cabezas y sacar los vástagos de los remaches o tornillos de ajuste de tamaños razonables; el "Hell-dog" es una máquina romperemaches, larga, pesada y muy potente que necesita dos o tres hombres para manejarla y operarla, y se utiliza para cabezas de remaches de gran diámetro.

Bote para recoger remaches: (Fig. 3.7.70).

Barras para remachar: tipo banjo (Fig. 3.7.71); club; en escuadra (Fig. 3.7.72); excéntrica o cuello de ganso (Fig. 3.7.74); con resorte (Fig. 3.7.75); recta (Fig. 3.7.76); cadena; gancho (Fig. 3.7.73).

Forja para remaches: con abanico; con abanico extra; con tobera de hierro extra. La tobera de hierro es una placa con agujeros que permiten el paso del aire y formar un tiro para mantener fuego encendido. Esta tobera se coloca en la parte inferior de la forja, sobre la abertura a través de la cual el abanico empuja el aire para formar el tiro. En algunas forjas se usa aceite como combustible, el cual se impulsa a presión.

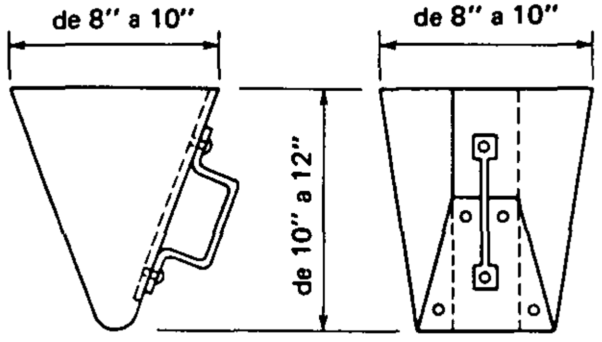


Figura 3.7.70 Bote para "cachar" remaches.

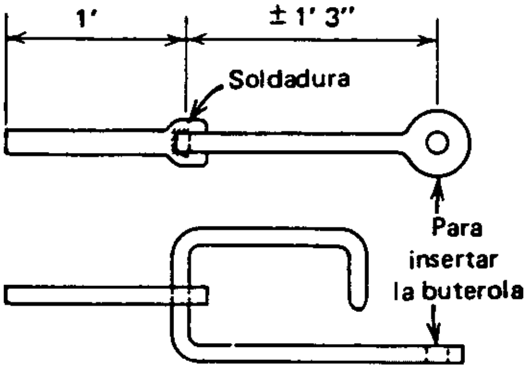


Figura 3.7.71 Barra tipo "Banjo" (número 9).

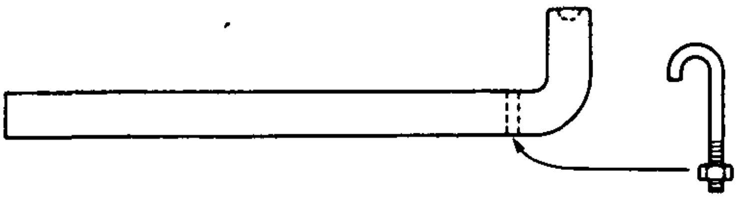


Figura 3.7.72 Barra con doblez a 90°. Figura 3.7.73 Tope o gancho de la barra.

Figura 3.7.74 Barra con cuello de ganso.

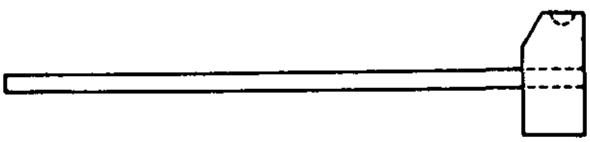


Figura 3.7.75 Barra con resorte.

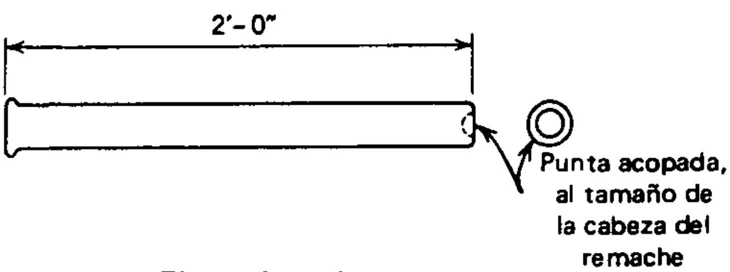


Figura 3.7.76 Barra recta.

En general, los calentadores eléctricos de remaches no son muy satisfactorios para una cantidad apreciable de remaches, así como tampoco para su uso en el campo. Estos pueden emplearse en el almacén para calentar ocasionalmente uno o dos remaches, eliminando la necesidad de prender y extinguir una forja cuando se trata de calentar unos cuantos remaches, lo cual no resulta económico.

Tenazas para calentar remaches: (Fig. 3.7.77); para revolver; con paletas; para recoger (Fig. 3.7.78).

Contraremachadoras (buterolas): cónicas (Fig. 3.7.79*a*); plana o al ras (Fig. 3.7.79*b*): piedra esmeriladora (conformada); calibradores.

Cable, de manila: andaríveles de grúa; para polipastas de mano, líneas para andamios; eslingas, molinetes para cables (para malacate; cable guía.

Cable, de alambre: *ver* Cable de alambre.

Bote de remos: remos; ancla.

Cuerdas y cinturones de seguridad: los cinturones de seguridad deberán tener hebillas de desenganche rápido.

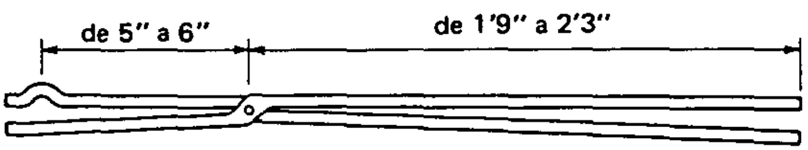


Figura 3.7.77 Tenazas para calentar.

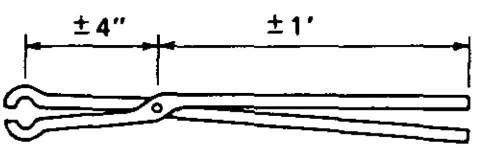
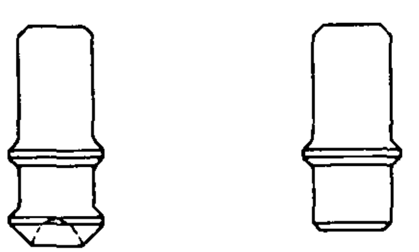


Figura 3.7.78 Tenazas para recoger.



(*a*) (*b*)

Figura 3.7.79 Buterolas (*a*) con copa para remaches con cabeza de botón, (*b*) planas para remaches planos.

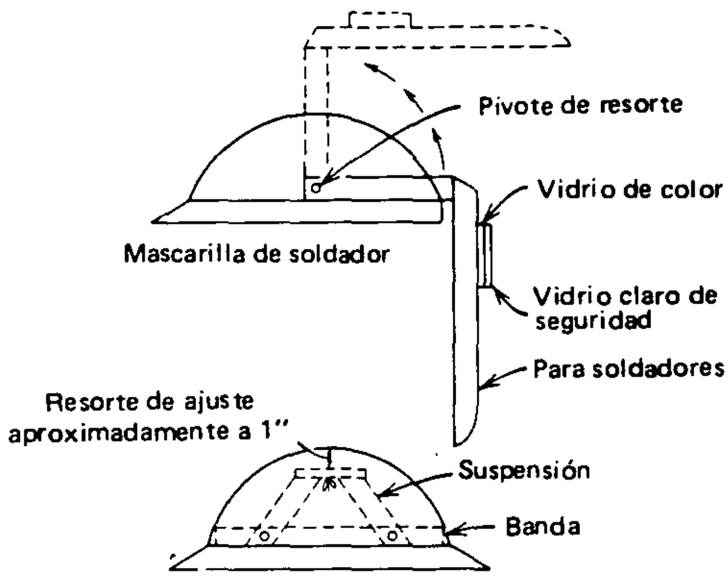


Figura 3.7.80 Casco de seguridad (sin ala frontal).

Cascos de seguridad (cascos duros): con ala (Fig. 3.7.81); sin ala en el frente (para usar caretas de soldador) (Fig. 3.7.80); con bandas extras: dē cuero; tejidas de cuero, de plástico, de hule espuma; revestido para el invierno.

Sierras: para corte transversal: de dos mangos, un mango; de mano; para metal, marco, hojas.

Andamios: silla para control; andamios; de vigas de agujas; para barcos. Los andamios de agujas verticales consisten en dos tablones cepillados de abeto Sitka (o el equivalente en peso y resistencia) de 4 × 6 plg y aproximadamente 26 pies de largo, unidos al centro y cerca de los extremos por medio de cuerdas. Los tablones cortos para andamios, en general de 4 × 6 ó de 2 × 10 plg y de 10 a 16 pies de largo se apoyan transversalmente en maderas de 4 × 6 plg. Los tablones tienen grapas o tornillos doblados en sus extremos para mantenerlos en su lugar.

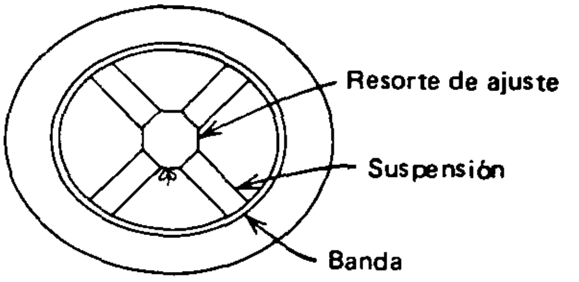


Figura 3.7.81 Casco de seguridad con ala.

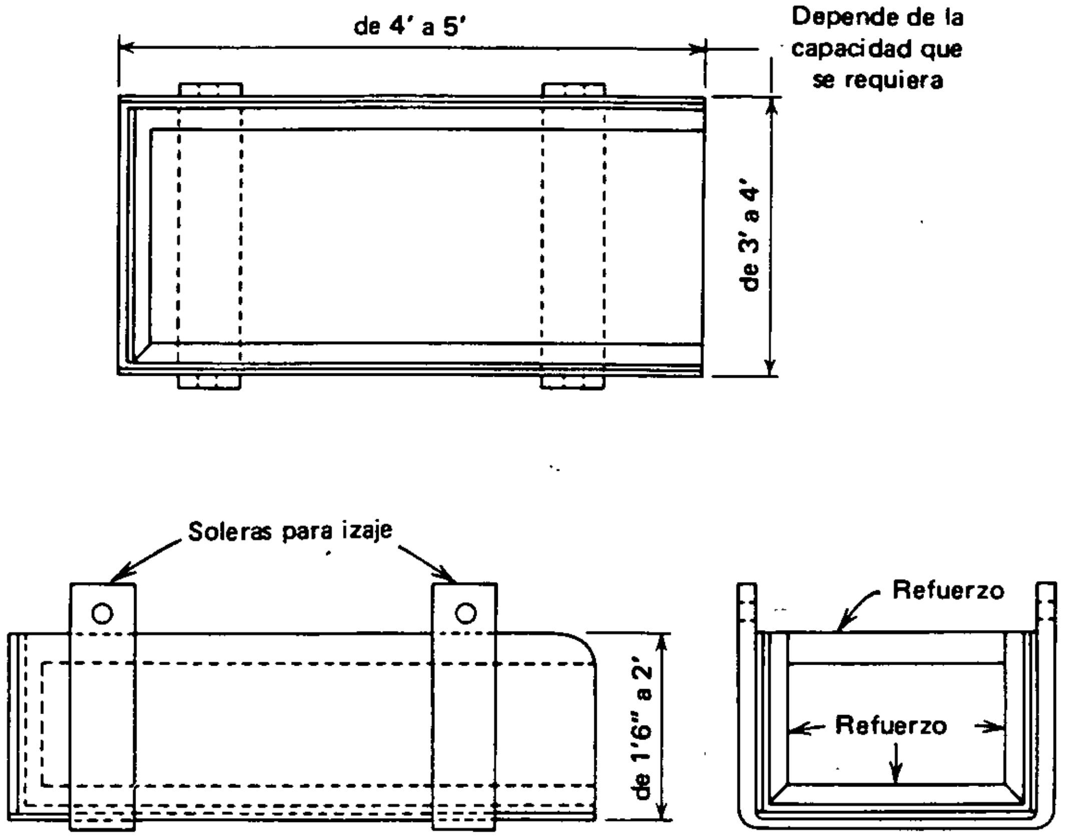


Figura 3.7.82 Caja de izaje de acero.

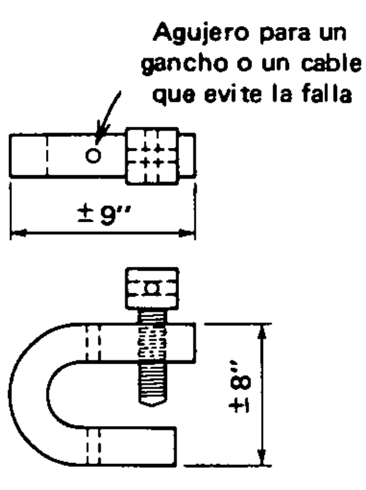


Figura 3.7.83 Mordaza de tornillo (mordaza tipo "C")

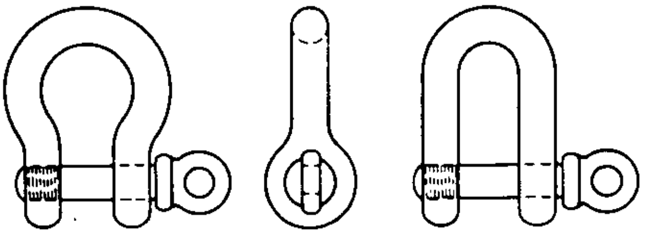


Figura 3.7.84 Argollas con pasador de tornillo. Los diámetros del pasador y la argolla dependen de la capacidad que se necesita.

Caja de izaje: de acero (Fig. 3.7.82), de madera, para manejar barriles o cubetas de tornillos, y otras piezas chicas. Con frecuencia se utiliza para subir a los obreros por medio de una grúa a puntos elevados a donde no es fácil llegar con escaleras.

Abrazadera de tornillo (abrazadera C): estructural (Fig. 3.7.83); cadena; gancho.

Desarmador.

Argollas: con pasador; con tornillos (Fig. 3.7.84), estándar (Fig. 3.7.85).

Cobertizos: para oficina; para almacén o "cobertizo para hombres"; combinación de oficinas y almacén o "cobertizo para hombres"; desarmado (Fig. 3.7.86); portátil de una pieza (Fig. 3.7.87); de remolque: de un eje (Fig. 3.7.88), de dos ejes (Fig. 3.7.89); grande, pequeño.

Lainas o calzas: Se deberá tener a mano una dotación de lainas para colocación de emparrillados, placas base, losas y ángulos guía, para embarcarse de inmediato al lugar de la obra cuando se necesiten. El suministro deberá renovarse subsecuentemente. En general, las lainas son de 3 X 3 ó 4 X 4 plg y de 1/8, 1/4 y 1/2 plg de espesor. En trabajos donde es crítica la nivelación del material base, se deberá disponer también de una dotación de lainas muy

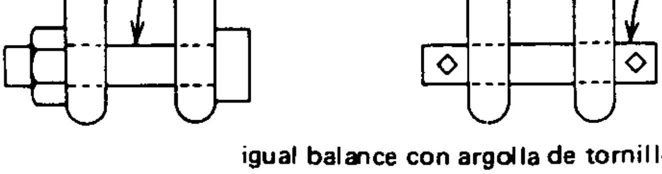


Figura 3.7.85 Argollas estándar de pasador.

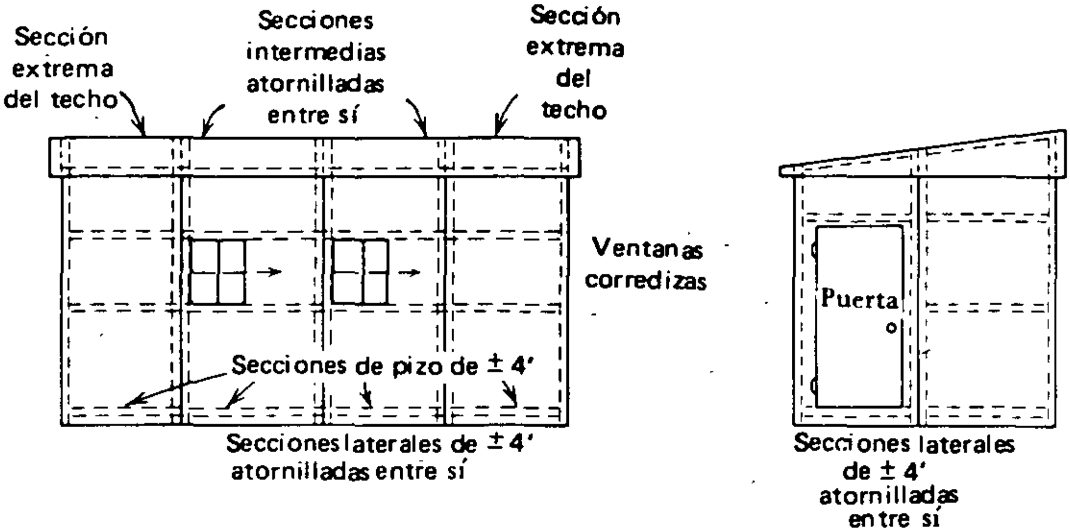


Figura 3.7.86 Cobertizo desmontable.

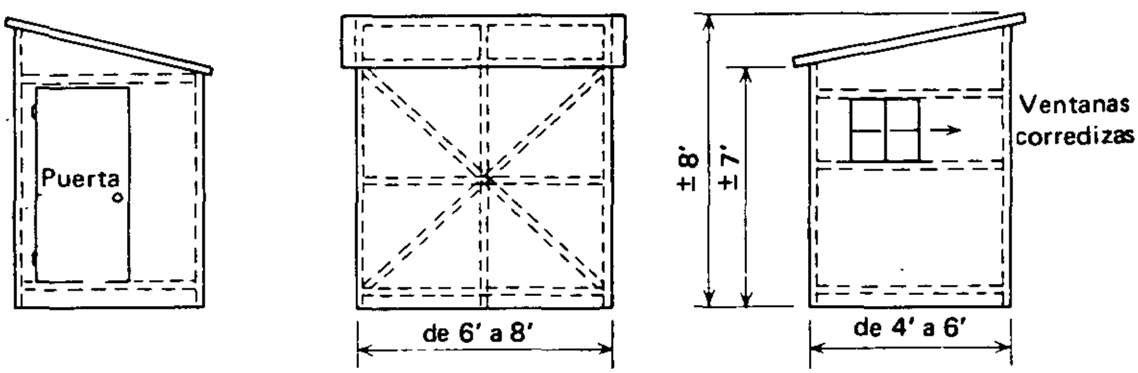


Figura 3.7.87 Cobertizo de una sola pieza.

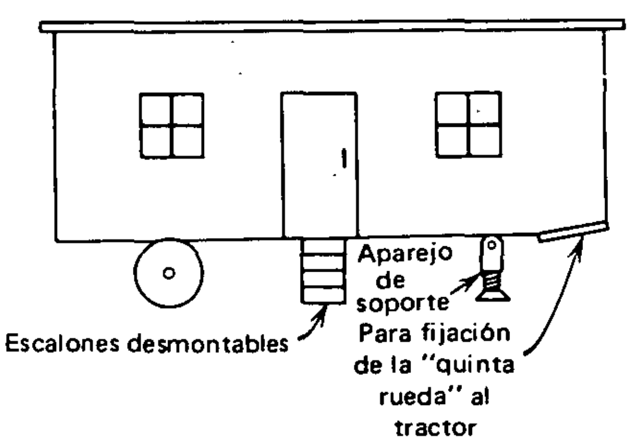


Figura 3.7.88 Oficina montada sobre remolque de un solo eje.

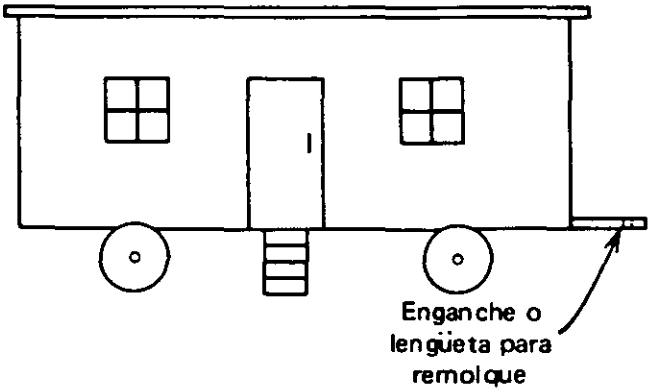


Figura 3.7.89 Oficina montada sobre un remolque de dos ejes.

delgadas. Los ángulos guía se ordenan específicamente para ciertos emparrillados, ya que sus longitudes dependen del ancho del ensamble de la parrilla y pueden requerir lainas de 3 X 5 plg sobre las que se van a nivelar, la argolla y el pasador dependen de la capacidad que se requiera.

Pala.

Campanas para señales: *ver* Campanas.

Sistema de señales: vocal; audífonos; altavoces; transmisores; alambre.

Marro: *ver* Martillo; de mano.

Protector de eslinga: (Fig. 3.7.90); se utiliza en los patines inferiores de trabes pesadas y evitar que las esquinas de los patines corten o dañen la eslinga. Su dimensión depende del ancho del patín y del diámetro del cable de alambre que se utilice.

Carretes: (Fig. 3.7.91) Estos son diseñados para proteger un tirante de alambre, etc., en donde se conecta a un pasador o tornillo. Las dimensiones varían según el diámetro del cable de alambre que se

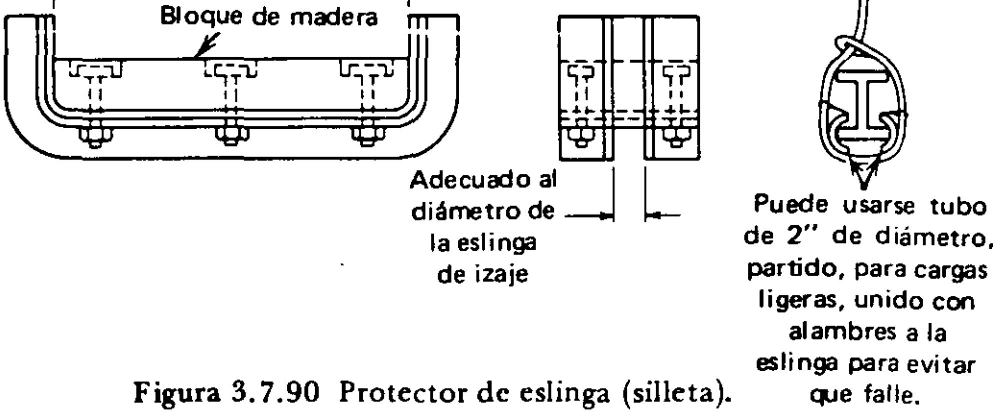


Figura 3.7.90 Protector de eslinga (silleta).

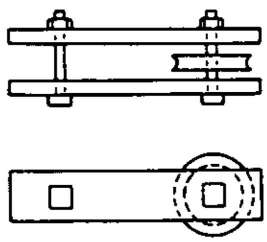


Figura 3.7.91 Polea y placas para fijar los tirantes al ojo del templador.

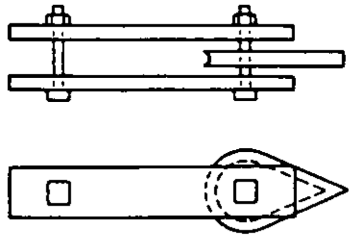


Figura 3.7.92 Guardacabo y placas para fijar los tirantes al ojo del templador.

utilice. Casi siempre se emplean en uniones donde el tirante se dobla sobre un punto de fijación y se hace un lazo temporal en el tirante por medio de clips para cable de alambre. Cuando un ojal en el extremo de un tirante de una sola pieza se asegura a un pasador o a un tornillo, es preferible usar un guardacabo, instalándolo en el momento de conectar el ojal, de manera que no se desprenda.

Trinquete de manguito: (Fig. 3.7.93) se utiliza para jalar y unir dos piezas cuando se requiere una fuerza mucho mayor que la que se puede aplicar por medio de un templador. Sus extremos se unen operando el manguito y el trinquete hacia atrás y hacia adelante. Un gato de tracción o de jalar y empujar es similar, excepto que este último no sólo las jala para unirlas sino que también empuja para separarlas.

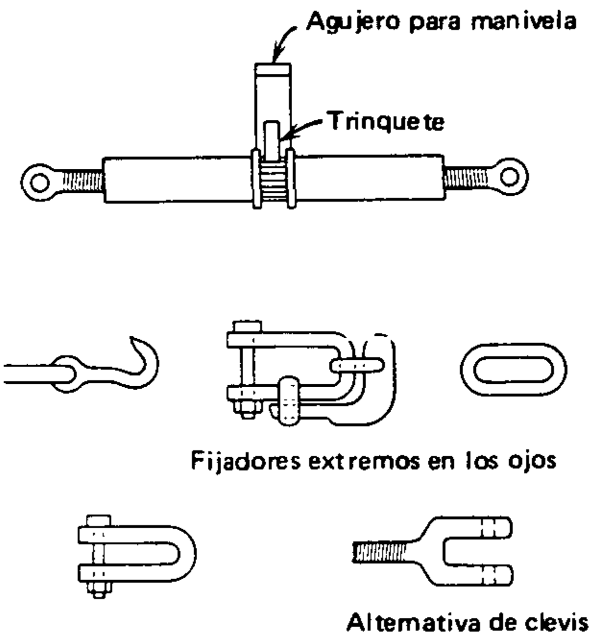


Figura 3.7.93 Trinquete con manguito.

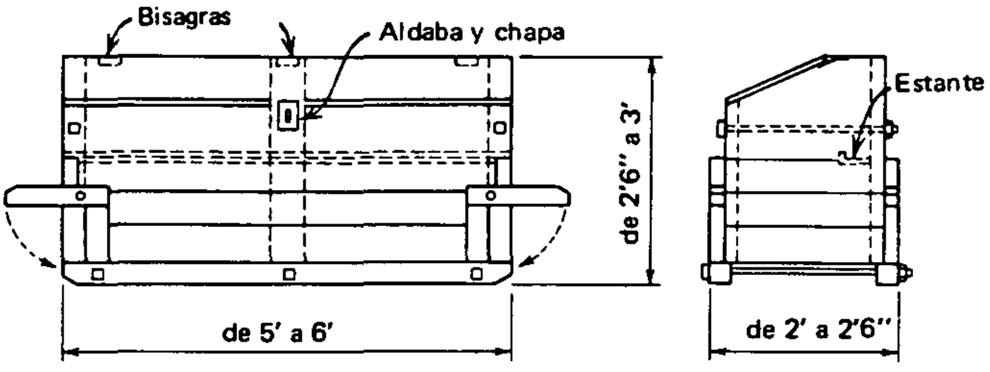


Figura 3.7.94 Caja de herramientas, de madera.

Tarraja y dados: para tornillos; para tubo.
Tanque para agua.
Cinta métrica: de acero; de tela. Por lo general, las medidas se toman en pies, y se cierran a octavos o dieciseisavos de pulgada. Muy raras veces se requerirá una cinta con décimas y centésimas de pie.
Lonas impermeables: se usan para proteger al equipo de los elementos naturales, y del polvo, chispas, escamas, escorias, etc. Es aconsejable que las lonas sean resistentes al fuego y al óxido.
Guardacabos: (Fig. 3.7.92); para tirantes o eslingas especiales. *Ver* Carretes.
Madera: entibado; carretón; obra falsa; andamios (*ver* Andamios); plataformas: grande, pequeña (*ver* Plataformas); vigas tipo aguja (*ver* Andamios); tablones para piso; tablones para andamios; largueros; puntales. En general, los tablones para pisos de trabajo y pisos extra son de 2 X 12 plg X 22 a 24 pies de largo, de abeto Douglas grado estructural o su equivalente en resistencia y peso

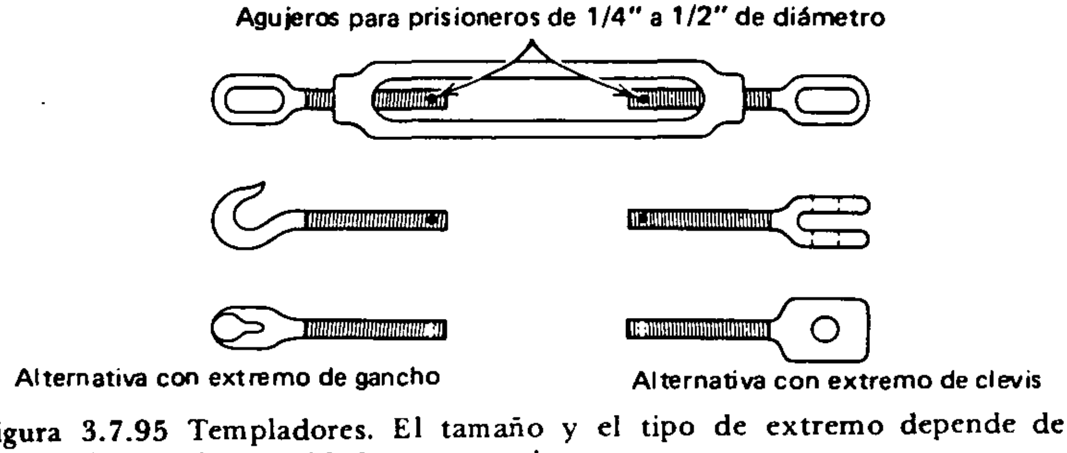


Figura 3.7.95 Templadores. El tamaño y el tipo de extremo depende de la utilización y de la capacidad que se requieren.

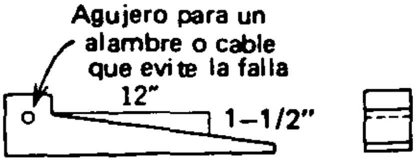


Figura 3.7.96 Cuña de acero.

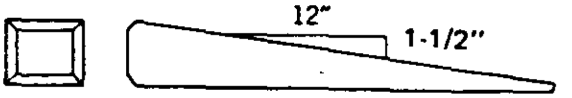


Figura 3.7.97 Cuña de madera.

con acabado áspero. Donde sea necesario y según el claro en que se van a utilizar, en ocasiones se emplearán tablones de 3 X 12 plg. para las cargas que se soportarán, pero deberán reducirse en longitud, cuando sea posible, para que dos o tres hombres los manejen con seguridad. La madera no se deberá pintar ya que se pueden tapar posibles defectos y si se colocan tiras de fijación en ambos extremos se evitan resquebrajamientos.

Los largueros que se utilizan para bajar un cargamente de acero o de piezas individuales de los carros o camiones de transporte, cuando se distribuya, en general son vigas de madera de 4 X 4 plg o también pueden ser dos o tres tablones de 2 X 12 plg colocados uno sobre otro. Estos tablones pueden ser de los que se han desechado para pisos o andamios por no llenar los requisitos de seguridad. La cantidad ordenada depende del área de trabajo, así como de la cantidad de cuadrillas de izaje y de otras, operando al mismo tiempo.

Rodillo para maderos: (Fig. 3.7.27); rodillos; ruedas, (*ver también* Rodillo para madera).

Cajas de herramientas: grande (Fig. 3.7.94); pequeña; de grúa; de grúa torre; para motor; del superintendente.

Almacenes: *ver* Cobertizos.

Tractor.

Marco móvil para grúa de torre con marco tipo A; grúa de torre atirantada; grúa de pies rígidos.

Templadores: (Fig. 3.7.95), para grúa de torre atirantada: tirantes de la pluma, tirantes en los bloques de apoyo, tirantes del mástil; para tirantes de plomeo, con ojos en ambos extremos, con ojo en un extremo y clevis en el otro, con clevis a ambos extremos.

Prensa de tornillo: de banco; de herrero; de tubo.

Rondanas: *ver* Tornillos.

Estona.

Recipiente para agua: *ver* Cubetas: agua.

Cuñas: de acero (Fig. 3.7.96); de madera (Fig. 3.7.97).

Soldadura: mordazas, portaelectrodo, cable de tierra, cable de carga, martillo, careta; vidrio para careta: simple (cubierta), sombreado.

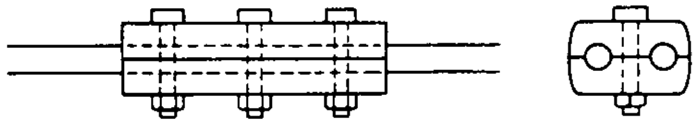


Figura 3.7.98 Mordaza para cable de alambre.

Máquinas para soldar: montadas sobre patines; montadas sobre ruedas; diesel, eléctricas, de gasolina; rectificadores; transformadores.

Silbato.

Mordazas para cable de alambre (Fig. 3.7.98)

Clips para cable de alambre (Fig. 3.7.99): para grúas de tirantes, para tirantes de izaje, para tirantes de mástiles; tirantes de postes-guía; tirantes móviles; para estrobos de línea de carga; para estrobos de líneas de plumas; para estrobos de correderas; para tirantes de plomeo.

Tirantes de cable de alambre: tirantes para izaje del mástil de las plumas; para poste-guía; para mástil de grúas de tirantes; para plomeo.

Amarres de cable de alambre.

Cable de alambre, corridas de: para malacates de aire; para cabrestante o winches; para líneas de las plumas de las grúas; como líneas de carga o como correderas; para líneas de mástiles de grúas, como líneas de carga o como correderas. El cable de alambre debe man-

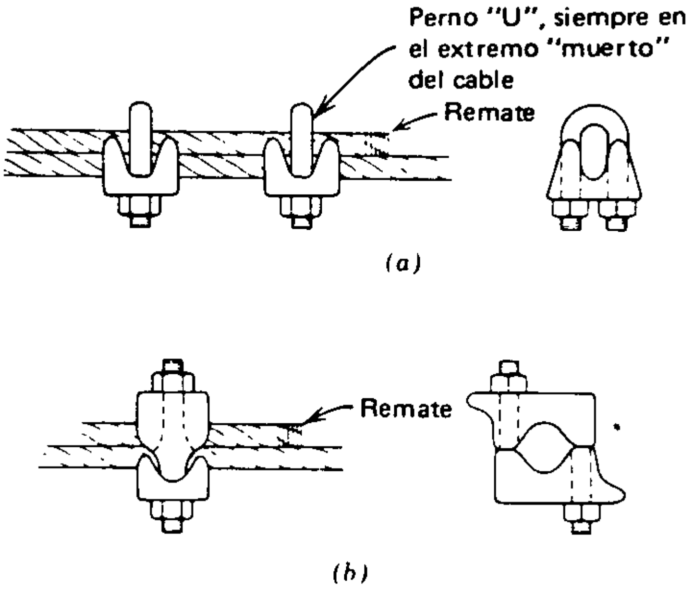


Figura 3.7.99 Clips para cable de alambre (*a*) tipo Crosby, (*b*) tipo "Laughlin" (de agarre rápido).

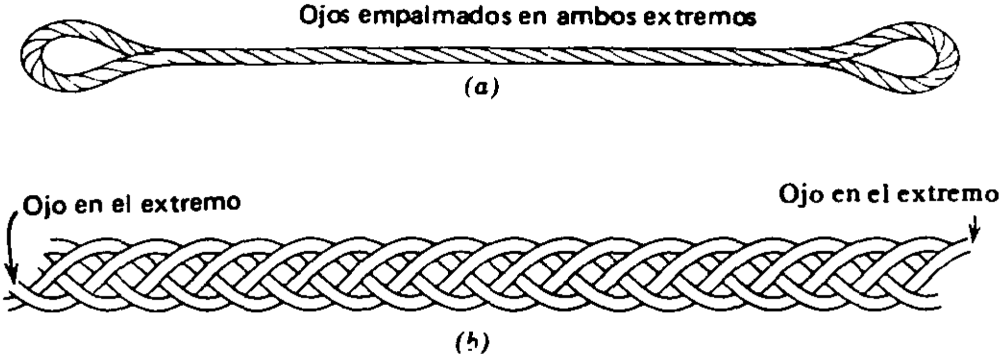


Figura 3.7.100 Eslingas de cable de alambre: (*a*) de una pieza con ojos en ambos extremos (*b*) trenzada, de longitud variable.

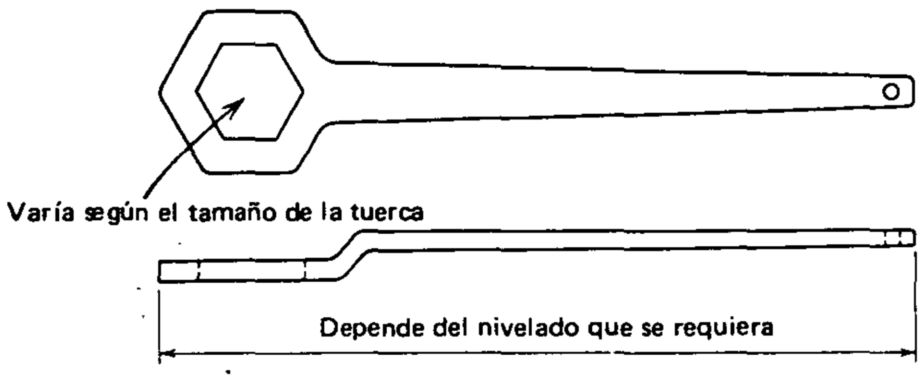


Figura 3.7.101 Llave de caja.

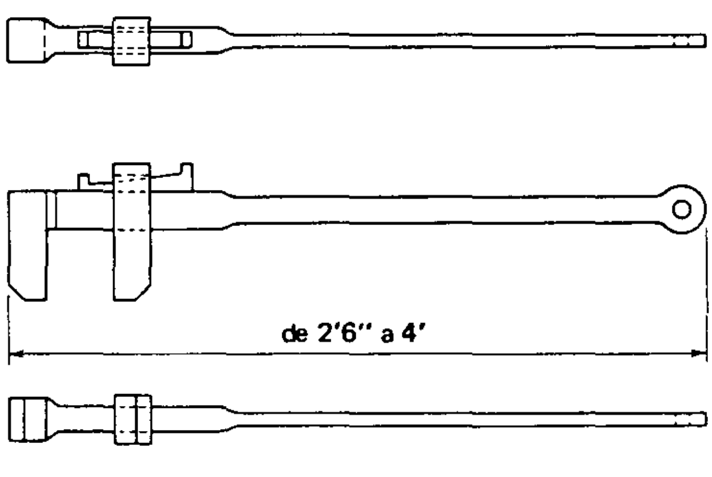


Figura 3.7.102 Llave de "perico".

tenerse bien lubricado y se debe revisar con frecuencia para confirmar si se ha desgastado, si hay alambres rotos, si se ha corroído o deteriorado, etc., quitándolo de inmediato del servicio cuando no esté en condiciones adecuadas o seguras para la resistencia y el servicio que se requiere de él. Debe establecerse un criterio para determinar cuándo debe descartarse el cable usado, con base en la reducción del diámetro ocasionada por el desgaste o rozamiento, o con base en la cantidad de alambres rotos en un cierto cable o torón, cantidad de dobleces, alambres flojos, etc. Por lo general el cable de carga desechado puede usarse en tirantes para amarre o para plomeo, en donde su resistencia reducida es todavía más que suficiente para estos propósitos.

Eslingas de cable de alambre: de una pieza (Fig. 3.7.100*a*); trenzadas, (Fig. 3.7.100*b*); para anclajes de columnas; para los ganchos colocadores de columnas; para anclajes de máquinas; con ojos en ambos extremos: de montaje, de descarga; "eslingas de calle". Al ordenar las eslingas deben especificarse la longitud, el diámetro y el tipo, basándose en el tamaño y el peso del material que se va a manejar. Este tipo de eslingas debe descartarse cuando estén tan dañadas o torcidas que no puedan sujetar de modo adecuado la pieza que se está izando, o cuando algunos alambres rotos no sean ya seguros para manejar la carga y puedan ser peligrosos para los hombres que los manejan. Los extremos empalmados deben rematarse con cuidado para poder manejarlos con seguridad.

Las "eslingas de calle" son eslingas con ojos en cada extremo, lo bastante largas para enredarlas alrededor de las piezas que se están descargando de un vagón o de un camión, pasando uno de los ojos a través del otro y enganchando después el ojo libre a los ganchos de izaje.

Distribuidores para cable de alambre: de izaje, con ojo y gancho pesado de izaje; para distribución, con ojo y gancho ligero de izaje; ganchos de repuesto. Algunos montadores empalman directamente los ganchos al ojo de uno de los dos extremos de las eslingas; otros usan eslingas con ojos en ambos extremos y ensartan los ganchos en los ojos.

Llaves: de caja (Fig. 3.7.101); tipo "crescent"; de perico (Fig. 3.7.102); inglesa; de cola (Fig. 3.7.103); de dado; Stillson.
En el equipo para la oficina de campo por lo general se incluyen alguno o todos los conceptos siguientes:

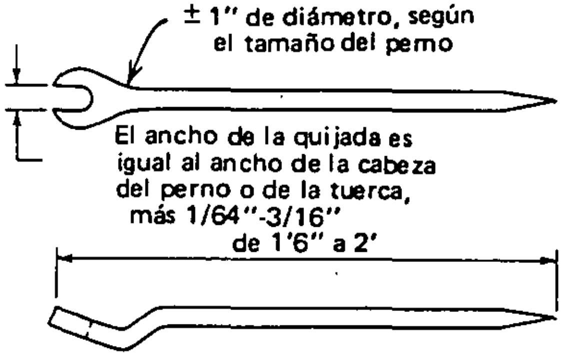


Figura 3.7.103 Llave de cola.

Equipo para la oficina de campo

Máquina sumadora
Tablero de avisos
Protectora de cheques
Calculadora
Archiveros
Equipo de primeros auxilios
Nivel: de constructor; de carpintero; de ingeniero (para topografía)
Barra de nivel (para topografía)
Caseta para oficina; sillas; escritorios, archivero para dibujos y planos
Plomada (de topógrafo)
Estufa para calefacción: grande, pequeña; de carbón; de petróleo, eléctrica, radiador de vapor, de gas propano
Camilla
Tránsito (de topógrafo)
Máquina de escribir
Reglamentos de seguridad
Cartelones de seguridad

FACULTAD DE INGENIERIA U.N.A.M.
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CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

PLAN DE MONTAJE GENERALIDADES

EQUIPOS Y HERRAMIENTAS
ASPECTOS GENERALES

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Capítulo 5

El plan de montaje

5.1 Preliminar

Una vez presupuestada una obra y entregadas las cotizaciones, o bien, después de que se ha obtenido un contrato en un concurso y se han comparado los documentos, especificaciones y dibujos del contrato con los de cotización (en relación con el presupuesto), entonces deben iniciarse de inmediato los trabajos para preparar un plan de montaje seguro, eficiente y económico; dicho plan debe estar dirigido a expeditar al máximo el trabajo de campo, dentro de los límites de la seguridad, considerando los costos adicionales que esto implica en función del ahorro de tiempo. Esto es indispensable si el tiempo que señala el contrato es muy "apretado" y, sobre todo, si en el contrato se incluye una multa si se rebasa ese tiempo.

Si aún es reciente la última visita que se hizo a la obra para preparar la cotización, quizá sea innecesaria una nueva visita, pero hay ocasiones en que ya ha pasado un tiempo considerable, y las condiciones del lugar pueden haber cambiado; tal vez ya se hayan iniciado los trabajos de cimentación, lo que implica excavaciones, y por tanto ya no existe la ruta de acceso que se planeó, el contratista general puede haber montado oficinas o casetas, o puede haber colocado su equipo en sitios que interfieren la entrega del equipo, las herramientas, o los elementos de la estructura; quizá se hayan colocado cables telefónicos que interfieren con el uso del equipo que se especificó al cotizar. Deben tomarse en cuenta todas estas posibilidades, y si existe alguna duda acerca de cambios en las condiciones, es necesario inspeccionar de nuevo el lugar de la obra antes de adelantar demasiado la planeación del montaje; después de revisar el lugar de la obra y las condiciones de los alrededores, se establecerá un plan de montaje que puede, o no ser el que se previó cuando se hizo el presupuesto. Siempre debe hacerse una visita de inspección después de haber establecido el plan de montaje, preparado los dibujos, entregado el programa al fabricante y terminado otros trabajos preliminares.

Por lo general, habrá un tipo especial de equipo que sea el más adecuado para el proyecto que se está estudiando, pero en ocasiones puede haber diferentes equipos de diferentes tipos que pueden ser igualmente seguros, económicos y eficientes; entonces su especificación dependerá de la disponibilidad del equipo y el costo de suministrarlo. Si existe la posibilidad de usar plumas o grúas viajeras para montar una obra, deben compararse también con el uso de grúas móviles y grúas-torre levadizas o fijas. El equipo de mástil corto y mayor capacidad debe compararse con los aparejos de mástil largo y menor capacidad.

Debe considerarse el uso posible de un poste-grúa, un poste-canasta, un poste-guía, una cabria o un aparejo liviano. Algún tipo de trabajo puede ser más adecuado para efectuarlo mediante operaciones manuales simples. Cuando existen vías acuáticas disponibles, debe tomarse en consideración el uso de equipo flotante, como plumas y grúas montadas sobre barcazas; no debe omitirse la posibilidad de combinar dos tipos diferentes de equipo de montaje, tales como plumas con grúas, postes con plumas o grúas u otras combinaciones.

Es necesario estudiar el tipo de energía para el equipo, para decidir si se usa diesel, gasolina, electricidad, o calderas de carbón o petróleo. En trabajos donde se usa equipo manual debe decidirse si se usan malacates pequeños accionados neumática o eléctricamente, cabrias o winches operados manualmente, o cable de manila manejado a mano o por medio de un carrete o un malacate movido por algún tipo de fuerza motriz; esta decisión puede influir en la selección del tipo de equipo auxiliar. Si se usará una instalación eléctrica para un malacate, entonces puede decidirse utilizar compresores, generadores para soldadura, transformadores y rectificadores eléctricos; si no se requiere electricidad, todas las máquinas de soldar, compresores, malacates, etc. pueden utilizar diesel o gasolina. Estas decisiones pueden determinar el peso del equipo escogido, lo cual a su vez puede implicar variaciones a los diseños de las áreas en que operará el equipo.

Cuando se está decidiendo el tipo de equipo que se usará, se requiere hacer un examen de muchas de las características del trabajo; las cimentaciones y las condiciones del terreno pueden ser factores determinantes, ya que en muchas obras el lugar está tan lleno de zanjas o zapatas de cimentación que una grúa no se puede mover con seguridad ni economía sin dañar las zapatas o causar derrumbes costosos en las zanjas o excavaciones. Las normas legales locales pueden prohibir algún tipo o determinar el uso de otro tipo de equipo. Las líneas elevadas de transmisión de corriente eléctrica, que no pueden moverse o desenergizarse pueden restringir el método de montaje. Se tiene que tomar en cuenta la capacidad que se requiere para manejar la pieza más pesada de la estructura.

Por último, deben compararse el tiempo que se requiere, el costo, la eficiencia y la seguridad de métodos de montaje en que se utilice un solo tipo o una combinación de varios tipos de equipo de montaje, en general seleccionando el que dé el resultado que se desea, en el tiempo permitido, por medio de los métodos más seguros y al menor costo. A menudo no se tiene una respuesta definitiva y rápida acerca del equipo a escoger.

5.2 Selección del método de montaje

Por lo general, el estudio de los planos del contrato y una revisión de las condiciones del lugar conducirán a una decisión acerca del equipo y el método que se utilizarán; el método seleccionado depende de la rapidez requerida y del equipo disponible, ya sea propio, o que se tenga que comprar o rentar. Se deben tomar en cuenta los costos relativos de muchos otros factores; el método depende de las condiciones del lugar, de las áreas disponibles para operar el equipo y de los riesgos de un plan determinado en comparación con otro.

Aparte de considerar si las condiciones del terreno permitirán usar grúas, plumas, grúas móviles u otro tipo de equipo, deben estudiarse varios métodos para determinar cuál es el mejor, tomando en cuenta todos los factores; por ejemplo, un edificio bajo, con miembros pesados, puede montarse con una pluma atirantada, una pluma de patas rígidas, una grúa de orugas de alta capacidad, una grúa montada sobre camión, o aun con una grúa móvil.

Se debe comparar el tiempo de instalación de una pluma con el tiempo en que puede entregarse una grúa totalmente aparejada. El costo de entrega de una grúa montada sobre camión, por sus propios medios, por lo general, es mucho menor que el costo de entrega de una grúa de orugas por medio de un transporte; de manera similar, el costo de embarcar, descargar, ensamblar, preparar y, después, desmantelar y devolver una pluma puede contrarrestar las ventajas que puede tener ésta sobre una grúa.

Se necesita mucho espacio para que las grúas móviles puedan moverse en el lugar de la obra; por tanto se reducen las áreas disponibles para la descarga, selección y distribución de la estructura, mientras que una pluma permite utilizar toda el área que la rodea, para los trabajos mencionados. Cuando el montador posee una pluma adecuada, pero tendría que rentar o comprar una grúa con capacidad suficiente (o viceversa) puede no haber duda, ya que la decisión está basada no sólo en el tiempo sino también en el costo, si el montador espera tener utilidades que le permitan seguir operando.

Es necesario tomar en cuenta el tipo, tamaño y altura de la estructura, las posibles interferencias con otras operaciones, el tráfico de carreteras o de peatones que pudiesen demorar la entrega de materiales, o bien restringir el área en la cual pueden entregarse dichos materiales en el lugar de la obra; con frecuencia, las normas legales locales limitan los horarios de entrega de los camiones y entonces es importante contar con equipo de gran capacidad para descargar con rapidez grandes partidas de estructura.

Por lo general, en un edificio alto de varios niveles es mejor utilizar una pluma con tirantes, elevándola piso por piso, que usar una grúa levadiza de menor capacidad o una grúa móvil con un mástil demasiado largo. En algunas ciudades no se permite que una grúa permanezca operando desde la calle, debiendo trabajar entonces dentro de los linderos del edificio; esto requiere dejar de parte de la estructura desmontada, desde el piso hasta el techo, mientras que algunas secciones se montan con la grúa y ésta retrocede para montar otra sección, de abajo hacia arriba y continúa retrocediendo y montando. Esto interfiere con la terminación del edificio, ya que no puede completarse ningún piso hasta que la grúa termine el montaje. Usando una pluma con tirantes o una grúa levadiza, pueden realizarse otras operaciones para ir completando pisos, tan pronto como la pluma o la grúa se haya retirado al siguiente nivel, y las cuadrillas de ajuste, de atornilladores, remachadores o soldadores vayan terminando sus trabajos en cada piso.

Se deben tomar en cuenta el clima, las posibilidades de inundaciones o vientos fuertes; en una excavación profunda, una tormenta súbita o una lluvia constante pueden inundar la excavación de manera tal que una grúa móvil no pueda operar sobre el terreno, mientras que una pluma o una grúa levadiza puede pasarse a un nivel superior y estar lista para trabajar en cuanto cese la lluvia.

Las estructuras circundantes pueden modificar la decisión sobre cómo montar y qué equipo usar. Si el nuevo edificio es angosto y está rodeado por completo de edificios viejos u otras estructuras, el equipo lógico a usar sería una grúa, pues los tirantes de una pluma estarían tan inclinados que no sólo serían inseguros, sino que sería difícil hacer girar el aguilón bajo dichos tirantes. Si es necesario usar una pluma, existe la posibilidad de utilizar un mástil 20 pies (6 m) más alto que el aguilón, en vez del mástil común que es 10 pies (3 m) más largo que el aguilón.

Si el lugar de la obra está en un área donde hay trabajadores experimentados sólo en montajes con grúa, éste es un factor que influirá sobre la decisión de usar una pluma; por otro lado, este factor debe balancearse en relación al costo que representaría transportar hasta el lugar de la obra a personal experimentado en montajes con pluma, para contrarrestar el montaje más lento o más costoso que se realizará con la grúa.

Siempre hay que esforzarse por utilizar el método que implique el menor riesgo para el personal y el equipo; la prevención de accidentes es de gran importancia puesto que una relación mínima de accidentes propicia una producción máxima y un costo mínimo. La velocidad de montaje que se espera lograr debe estar en relación con la velocidad a la que el fabricante podrá producir y cargar, así como con la velocidad a la cual el transportista podrá entregar el material fabricado y con la velocidad de descarga y de montaje que se tendrá con el equipo del montador.

5.3 Montaje con grúa

Las grúas para montaje se pueden seleccionar cuando en el lugar de la obra se espera encontrar un terreno con condiciones adecuadas para la operación de grúas móviles, ya sea con o sin pisos de madera, tablones o caminos de troncos a través del área. En caso de que existan zanjas o aberturas, es necesario asegurarse de que se puedan construir pasos o puentes para soportar la grúa; también, es necesario confirmar si habrá zapatas, cimentaciones o muros que puedan interferir con los movimientos de las grúas y si habrá obstáculos elevados; todas estas preguntas deben contestarse. Este equipo se podrá utilizar si la estructura no sobrepasa el alcance de los mástiles de las grúas disponibles, de orugas o montadas sobre camión, y si el peso de las piezas que se izarán a las diferentes alturas está dentro de la capacidad de dichas grúas.

Por lo general, una grúa montada sobre orugas debe entregarse mediante carros de ferrocarril o mediante camión, ya que si se mueve sobre sus propias orugas, puede dañar las carreteras, y a menudo es necesario desmantelarla para restringirse a los anchos y alturas libres que se encontrarán durante el trayecto; por lo tanto, debe considerarse el costo por desmantelarla, cargarla, descargarla y ensamblarla de nuevo. Debe confirmarse si, utilizando un transporte, debe desmantelarse el mástil parcial o totalmente o puede entregarse ensamblado por completo y montado sobre la grúa.

En general, las grúas montadas sobre camión pueden circular por los caminos, por sus propios medios; sólo se requiere quitarles los contrapesos, pero puede ser necesario desmantelar el mástil si el peso de éstos excede los límites legales.

Cuando el ferrocarril que se usará para las entregas de material cuenta con vías dentro del lugar de la obra y especialmente cuando se van a instalar vías permanentes dentro del área de trabajo, el tipo lógico de equipo a usar puede ser una grúa locomotora, si las vías pueden cruzar las posibles obstrucciones; pueden usarse vigas, temporalmente, para soportar las vías a través de las zanjas y, si los muros o columnas no son muy altos o están muy cercanos como para estorbar el giro de la grúa, ésta puede ser el mejor tipo. La mayoría de las grúas locomotoras son de altas capacidades, aun con mástiles largos.

Antes de tomar la decisión sobre el uso de grúas en vez de plumas, o acerca del tipo de grúa que se usará, se debe estudiar la forma general de la estructura, los pesos de los miembros, las condiciones del terreno, las obstrucciones elevadas y las interferencias con el tráfico de vehículos y de peatones.

Las grúas-torre se usan vez más como equipo de montaje para ciertos tipos de estructura; en la actualidad se encuentran disponibles con bajas capacidades, a su máximo alcance, pero con mayores capacidades en un radio mínimo. Una grúa-torre del tipo estático o fijo requiere de un anclaje excepcional, ya que debe contar con grandes contrapesos o tirantes, para compensar el excesivo momento de volteo.

En el comercio existen grúas montadas sobre camión, con capacidades máximas para un mástil básico (cerca de 60 pies), o capacidad de más de 125 ton con un radio mínimo, equipadas con un mástil y un aguilón que alcanzan hasta 330 pies manejando cargas ligeras. Debe considerarse su peso, ya que las condiciones del terreno pueden no ser adecuadas para que operen con seguridad.

Existen grúas de orugas que pueden levantar 165 ó 200 ton con un mástil corto, pero pueden usar mástiles y aguilones con los cuales pueden llegar hasta 300 ó 400 pies con cargas ligeras; este tipo de grúa también es muy pesada y es necesario comparar las condiciones del terreno en que se usarán, ya que en general la concentración de carga en las ruedas de una grúa montada sobre camión es mayor que la concentración de carga de las orugas.

Para seleccionar una grúa-torre en vez de una grúa montada sobre camión o una grúa de orugas, o aún una grúa torre montada sobre un camión, la estructura debe poderse adaptar al uso de una grúa torre de tipo fijo o levadizo. En el caso de una estructura larga, debe contarse con espacio suficiente a todo lo largo, para poder utilizar una grúa-torre montada sobre una plataforma que se deslice o ruede sobre rieles tendidos sobre el piso.

Para tomar una decisión acerca del uso de grúas móviles en el montaje de una estructura alta es necesario considerar el riesgo extra que representa para el personal que trabaja en lo alto; si los elementos estructurales se izan desde el piso hasta su posición en lo alto del edificio, a través de áreas montadas previamente, no existe ninguna cubierta protectora debajo del personal que realiza las conexiones. Usando una pluma con tirantes, una grúa fija o una grúa levadiza, existirán pisos de tablones por debajo del personal, máximo dos o tres niveles más abajo.

Como una alternativa, puede escogerse el material cuya posición no esté cercana a la estructura previamente montada y entregarlo con la grúa a las cuadrillas de conexión; con esto se permite el uso de tablones cerca de las áreas donde se está trabajando y donde después trabajarán las cuadrillas de ajuste, las de atornillado, remachado o soldadura; pero es un método lento. En vez de esto, pueden colocarse pisos de tablones en áreas pequeñas e izar paquetes de elementos estructurales para depositarlos en dichas áreas, seleccionarlos y montarlos pieza por pieza desde el piso, dejando dicho piso en su lugar hasta que se hayan terminado todos los trabajos necesarios directamente por encima de él.

El montaje de algunas estructuras, como las situadas dentro de una excavación profunda, se presta al uso de una grúa móvil colocada en la calle, la cual se instala montando un panel o una nave a través del frente, hasta el nivel de la calle (una nave es una serie de paneles a través de un edificio). La grúa se mueve después hacia la estructura ya montada, y se coloca sobre soportes adecuados, para montar el siguiente panel o nave del edificio; a continuación, la grúa se usa para montar repetidas veces desde el nivel de la calle y desde la estructura que va montando. Tan pronto como la grúa llega a la parte trasera del edificio, puede moverse hacia la calle sobre la estructura ya montada, usando plumas para montar las secciones superiores de la estructura mientras la grúa va retrocediendo; también puede usarse para montar una parte de la estructura del nivel superior mientras retrocede hacia la calle.

Para usar de esta manera la grúa, la estructura permanente del edificio se debe revisar para asegurarse de que sus miembros son adecuados para soportar la carga de la grúa, ya sea mientras se mueve o mientras se monta. Si algunos de los miembros o de las conexiones deben ser más fuertes de lo indicado en el diseño original, se debe notificar al fabricante con suficiente tiempo para cambiar los dibujos de detalle y antes de que haya hecho el pedido de los materiales; al mismo tiempo, es necesario determinar el costo extra del refuerzo para llegar a un acuerdo y decidir si lo pagará el montador, el fabricante, el cliente o el propietario.

Si se usa obra falsa o puntales por debajo de los miembros definitivos, puede eliminarse la necesidad de reforzar las piezas que soportarán el equipo de montaje.

Cuando la estructura se inicia a nivel, de manera que la grúa se puede mover desde la calle al lugar de montaje y si las condiciones del terreno son favorables, puede usarse una grúa móvil, que situada en la parte trasera comience a montar naves a través de la estructura y de piso a techo; a continuación, retrocediendo y montando repetidas veces, puede montar nave tras nave, saliendo por último a la calle para montar la última nave del frente del edificio. Por supuesto, con este método no pueden completarse pisos hasta que la grúa haya terminado el montaje de toda la estructura.

Como alternativa, en el caso de una grúa torre montada sobre camión, si el edificio no es muy alto, si la torre es lo bastante alta para librar la parte superior de la estructura terminada y si el aguilón es lo bastante largo y tiene capacidad suficiente a la distancia requerida, la grúa puede localizarse justo fuera del límite exterior del edificio. Si la grúa gira hacia la calle para descargar la estructura de los transportes y la lleva después al área que le corresponde, puede proceder con el montaje de un nivel completo y continuar montando los niveles superiores uno a uno, llevando las piezas de la estructura desde el punto de descarga hasta el último nivel terminado, montando de manera similar a una pluma con tirantes o una grúa levadiza; en vez de cambiar de nivel como debe hacerlo una pluma con tirantes, la grúa-torre continúa el montaje con la torre y el aguilón colocados en la misma posición inicial, en el exterior del área del edificio.

Se debe estudiar el costo de embarque y ensamble de cada tipo de grúa, junto con las diferencias del tiempo resultantes de las velocidades de montaje. La necesidad de fijar las ménsulas de apoyo de las grúas montadas sobre camión, las grúas locomotoras, y las grúas-torre montadas sobre camión, cuando se manejan cargas cercanas a su capacidad, se debe comparar con el ahorro de tiempo que representa usar otro tipo de grúas que no requieren de ménsulas de apoyo.

Por medio de un estudio se determinará si el ahorro de tiempo que se obtiene al usar una o más grúas para descargar y distribuir materiales, mientras que una o más grúas los montan a continuación, justifica el costo extra de esta solución.

Las grúas que se usan para descarga y montaje pueden realizar sus labores respectivas y pueden combinarse después para levantar piezas demasiado pesadas para una sola de ellas. Una grúa de capacidad más ligera puede maniobrar y montar con más facilidad que una grúa pesada; con este arreglo también se puede tener una grúa ensamblando armaduras, trabes o subensambles antes de que se necesite montarlas; también permite que la fijación permanente (atornillado, soldadura, etc.) de estos ensambles se haga cerca del piso en vez de hacerlo en lo alto, reduciendo así el costo y el riesgo.

Cuando en una estructura existen algunas piezas excepcionalmente pesadas y el resto son partes ligeras, puede ser mejor elegir dos grúas de igual capacidad para descargar y colocar entre ambas los miembros pesados, para montar después por separado los elementos ligeros de las áreas adyacentes; aunque con esto se expedita el montaje y se requiere menos capacidad de soporte en el piso, también significa que los costos de embarque, descarga, ensamble y después los de desmantelamiento, carga y embarque serán el doble de los que representaría usar una sola grúa de mayor capacidad. En un arreglo más eficiente se podría tener una grúa montada sobre camión trabajando en conjunto con una grúa montada sobre orugas, para manejar las piezas pesadas, utilizando también la primera para descargar todos los materiales ligeros y la grúa de orugas para montarlos.

En el montaje de armaduras, una grúa montada sobre camión puede sujetar la primera armadura de la serie, después de que se monta y, con grúas de mayor capacidad, se montan la siguiente armadura y el suficiente arriostramiento para que ambas armaduras sean estables y se detengan solas; una vez hecho esto puede quitarse la grúa montada sobre camión. De esta manera se elimina el costo y la necesidad de colocar tirantes en la primera armadura para que la grúa de montaje pueda soltarla con seguridad; a veces es difícil obtener los anclajes para estos tirantes y si éstos se reemplazan con una grúa extra, toda la operación es más segura, rápida y a menudo más económica.

5.4 Montaje con pluma de tirantes o grúa torre, fija o levadiza.

Si las condiciones en el lugar de la obra no son favorables para el montaje con grúas montadas sobre camión, sobre orugas o grúas-torre montadas sobre camión, el equipo que se seleccione puede ser una pluma de tirantes, una pluma de patas rígidas, una grúa viajera, o una grúa-torre, fija o levadiza. La selección lógica puede ser una grúa de tirantes si el edificio es muy alto para utilizar las grúas de orugas, sobre camión, o grúas-torre disponibles, o si las cargas están fuera de su capacidad.

En edificios muy altos, muchos montadores levantan el primer nivel de la estructura con una grúa montada sobre un camión o una de orugas, si éstas se pueden rentar en la localidad, de una manera económica y listas para trabajar; encima de esta estructura, la grúa coloca plumas de tirantes con las cuales se monta el resto de la estructura. Un procedimiento eficiente es usar una grúa para descargar y ensamblar el equipo, colocar placas base o emparrillados y montar después el primer nivel; con esto se elimina la instalación de anclajes para los tirantes de la grúa y el tiempo que se requiere para cambiar la longitud de los tirantes después de cambiar de nivel la pluma para salir de la posición inicial. Cuando los trabajos de cimentación se han terminado antes de conceder el contrato de montaje, se obtiene un gran ahorro al eliminar los anclajes que habría que colocar para los tirantes.

Para elegir el tamaño y la localización lógica de una pluma, una grúa levadiza o una grúa torre fija, es necesario dividir la estructura en áreas y niveles y fijar las alturas de los niveles; a continuación, si el edificio no es muy ancho y el equipo puede trabajar de un lado a otro, se estudia la estructuración y se escoge un punto aproximado para localizar la grúa o la pluma. Este punto debe estar situado aproximadamente a la mitad de la distancia entre el sitio en que se entregarán los materiales cerca de la estructura y la parte posterior del edificio, del lado opuesto al punto de entrega; de esta manera, el mástil puede alcanzar el punto de descarga en la calle y puede montar los elementos de la parte posterior de la estructura. Sin embargo, con esta solución pueden tenerse tirantes de diferentes longitudes si se usa una grúa atirantada, lo cual no es aconsejable, y en este caso deben balancearse todos los factores.

La pluma, grúa fija, o grúa torre levadiza no deben localizarse en un tiro para elevadores o un pozo para escaleras, ya que esto interfiere con la instalación de los elevadores y escaleras, que deben hacerse lo más pronto posible después del montaje de la estructura, para reducir el número de pisos que el personal debe subir por medio de escaleras provisionales para llegar al piso de trabajo. La localización de la grúa debe ser tal que las líneas del aguilón y de la carga no interfieran con los elementos de los niveles inferiores al ir cambiando de nivel todo el aparejo.

El equipo debe localizarse de manera que libre la estructura permanente al cambiar de nivel; de otra manera tendrían que omitirse muchas partes hasta que la grúa se haya cambiado de nivel. Cuando sea factible, la localización debe ser tal que el aguilón pueda llegar por encima de los cabezales en los paneles situados alrededor del área donde el aparejo quedará encerrado antes de cambiar de nivel. Si estas piezas no se montan antes de cambiar de nivel la grúa, puede correrse el riesgo de que la estructura permanente no tenga el soporte lateral necesario que le dan las vigas que se supuso estarán colocadas cuando se revisó la resistencia que debía tener la estructura para soportar el aparejo.

Se debe seleccionar un área razonable en la otra dirección (en el sentido longitudinal del edificio) para decidir si con una sola pluma, una grúa levadiza o una grúa fija con un mástil largo puede manejarse todo el piso, o si deben usarse dos o más aparejos con mástiles más cortos.

Se deben balancear las áreas que cubren varios equipos, para que cada uno de ellos tenga aproximadamente la misma cantidad de trabajo; de otra manera, uno de ellos se adelantará al resto, con las complicaciones resultantes.

Es necesario revisar la estructura de apoyo para las vigas de levantamiento que soportan una pluma de tirantes o una grúa levadiza (Fig. 5.4.1) y, si la resistencia de la estructura permanente no es suficiente para la carga, deben diseñarse soportes temporales, de ordinario conectando los miembros especiales directamente a las columnas cercanas a la posición de la pluma.

Si se escoge la pluma de tirantes para el montaje, una vez que se han determinado la cantidad, localización y el tipo particular de pluma a utilizar, con base en la capacidad y la longitud del mástil y del aguilón, debe decidirse la localización de los tirantes, que deben ser ocho, de preferencia, así como las columnas a las que se fijarán;

dichas columnas deben estar espaciadas lo más regularmente posible en sentido angular y tan lejos de la base de la pluma como sea necesario para que las distancias de los tirantes sean las adecuadas, de acuerdo con las tablas de capacidad. Debe hacerse un esfuerzo para que los tirantes de dos plumas adyacentes no se interfieran en su operación. Los tirantes deben localizarse de manera que cuando menos dos de ellos trabajen detrás de la pluma cuando se descarga material y dos cuando se coloque la pieza más pesada. Debe evitarse tener tirantes cortos en un lado de la pluma y tirantes largos en el otro, ya que esto tiende a dificultar el giro de la pluma.

Si algunos tirantes no quedan lo bastante alejados de la base de la pluma como para manejar la carga con seguridad, debido a limitaciones de la estructura, pueden usarse ménsulas de apoyo; éstas consisten en vigas de acero o maderos pesados, amarrados con firmeza a la estructura y volados por fuera del borde del edificio. El tirante se fija al extremo exterior de la ménsula, el cual se fija a su vez diagonalmente a la estructura de la parte inferior, para tomar la fuerza vertical del tirante por debajo del piso de trabajo.

La estructuración del piso de trabajo, debe analizarse para asegurar que transmitirá las fuerzas horizontales de los extremos inferiores de los tirantes hasta la base de la pluma; en ocasiones es necesario usar puntales temporales para este propósito, ya que algunos edificios tienen paneles grandes y abiertos, sin arriostramiento entre columnas en la dirección necesaria para transmitir las fuerzas de los tirantes hasta la base de la pluma.

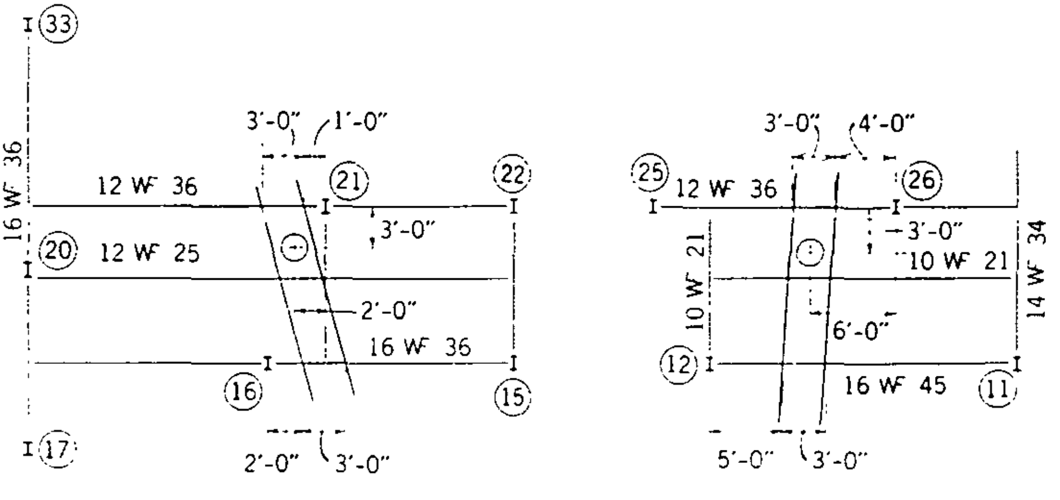


Figura 5.4.1 Croquis típico, mostrando la localización de las vigas de levantamiento. ⊗ indica la localización de la pluma.

El tamaño y peso de las zapatas de las columnas que se usarán para los tirantes se tienen que revisar para asegurarse de que podrán soportar las reacciones horizontal y vertical de los tirantes de la pluma colocada en su posición inicial sobre el piso (o en una torre de obra falsa si se requiere debido a interferencias con los arriostramientos colocados a través del área). Si las zapatas son satisfactorias, se hace un croquis (Fig. 5.4.2), una copia fotostática, una copia del dibujo del diseño de la cimentación, o simplemente un diagrama de la localización de las columnas, mostrando la situación de los anclajes que deben ahogarse para fijar los tirantes, así como la localización de la pluma. Este se envía al contratista de la cimentación o al contratista general para que se instalen los anclajes. Debe darse una copia al superintendente de campo para que pueda revisar si se instalaron en forma correcta; en este croquis debe mostrarse la manera de colocar los anclajes de horquilla, si se va a utilizar este tipo.

Si se usarán amarres en vez de anclajes de horquilla, debe indicarse en el croquis la manera de instalarlos. En general se deja una barra de refuerzo en la parte inferior del lazo de amarre, en la zapata. Se debe avisar al almacén tan pronto se haya determinado cuándo se deben entregar las anclas o los amarres; en el caso de los amarres es conveniente preparar las diferentes vueltas dándoles su forma final y amarrándolas con alambre ligero en varios puntos, para mantener todas las vueltas alineadas exactamente unas con otras, ya que todas ellas recibirán su parte de la carga por igual. Esto debe indicarse en un croquis por separado al superintendente del almacén, dando el tamaño del alambre, el número de vueltas y las dimensiones del lazo de amarre.

Si se encuentra que las zapatas no son adecuadas, y si en los lados de la excavación o en el piso se encuentra roca sólida, pueden usarse anclas de extremos abiertos. De nuevo, es necesario enviar un croquis al contratista general y al superintendente de campo, mostrando la localización, la pendiente o dirección y el tamaño de los agujeros que es necesario barrenar. su profundidad y el método exacto de instalación de las anclas, con sus cuñas colocadas en los extremos inferiores. Es más seguro que el contratista de la cimentación sólo barrene los agujeros y los selle temporalmente, usando después las cuadrillas de montaje para fijar las anclas en su sitio y probarlas. De esta manera el superintendente de montaje sabrá que se instalaron en forma correcta.

Si no hay rocas en las cercanías y las zapatas no son adecuadas, puede ser necesario agregar contrapesos a las zapatas. Si se va a usar una grúa para colocar los emparrillados y losas, o para armar la

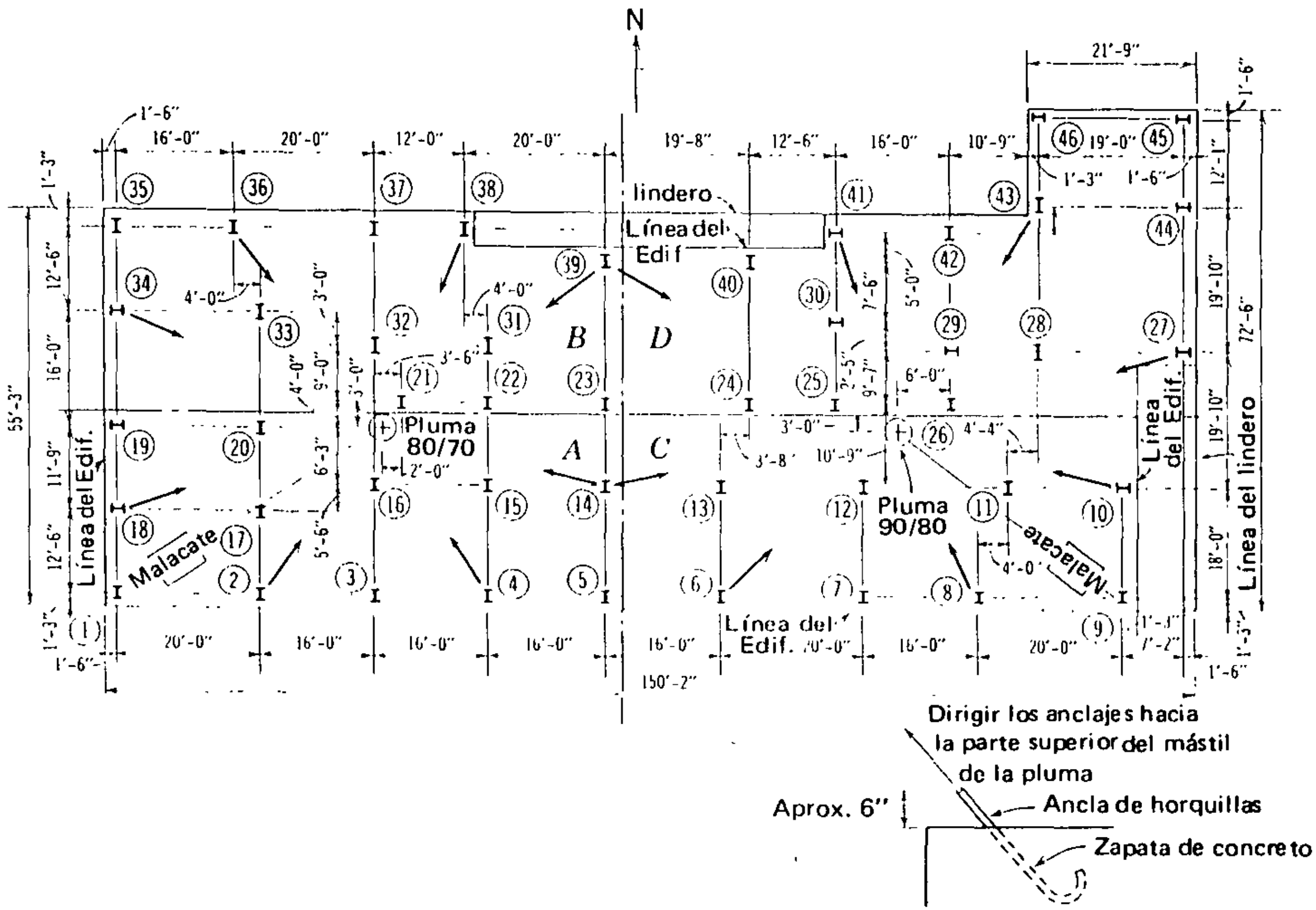


Figura 5.4.2 Croquis típico para la colocación de anclajes para una grúa de tirantes.

pluma, también puede usarse para colocar los contrapesos, que pueden consistir en algunos de los miembros estructurales más pesados, colocados de manera que no estorben la colocación de los emparrillados o losas en su posición final. Debe informarse al fabricante cuáles serán las piezas que se requiere tener desde antes en la obra, para usarlas como contrapesos, ya que puede ser necesario que las fabrique fuera de programa para tenerlas listas y embarcarlas en el momento adecuado.

Como último recurso, pueden hincarse estacas en el terreno para usarlas como anclas. Esta solución es algo precaria, ya que no hay manera de saber qué fuerza resistirán, dependiendo de la consistencia del terreno y de la profundidad y tamaño de las estacas.

La carga que se tendrá bajo la pluma o la grúa levadiza se calcula para asegurarse de que es adecuada la estructura permanente de cada piso de trabajo. Esta reacción debe tomar en cuenta no sólo el peso de la pluma o de la grúa, sino también el tirón de las líneas del mástil y de la carga, el peso de las vigas de elevación y cualquier piso de tablones, miembros estructurales, etc., en el área cercana. En el caso de una grúa de tirantes deben incluirse también las fuerzas verticales que los tirantes transmiten al mástil, ocasionadas por las cargas críticas al alcance máximo.

También se debe analizar la estructura permanente para revisar su resistencia para soportar algunas estibas de material que se puedan depositar sobre ella antes de distribuirlas en todo el piso; si esto es crítico, en las instrucciones al campo debe incluirse información detallada. Los dibujos del plan de montaje deben mostrar dónde puede descargarse material con seguridad. Las estibas de material se descargan en "paquetes" concentrados y, debido a la incertidumbre en cuanto al sitio en que las distribuirá el capataz de la cuadrilla de izaje, en los cálculos debe usarse una carga uniforme basada en un total de una vez y media al peso de la estiba de acero considerada, más el peso de los tablones del piso, las cajas de herramientas y el personal.

La estructura que soporte las vigas de elevación colocadas debajo de la pluma o grúa levadiza se revisa por momento, esfuerzo cortante, aplastamiento del alma y también se revisa la resistencia de las conexiones por cortante y aplastamiento. Quizá en esta etapa sea necesario relocalizar las vigas de elevación (Fig. 5.4.3) (y todo el aparejo) más cerca de las columnas de cualquiera de los extremos, debido a que el momento es muy grande, colocándolas donde se planeó originalmente. Si las vigas de apoyo opuestas de ese panel son todavía muy débiles aun moviendo las vigas de elevación, estas vigas pueden

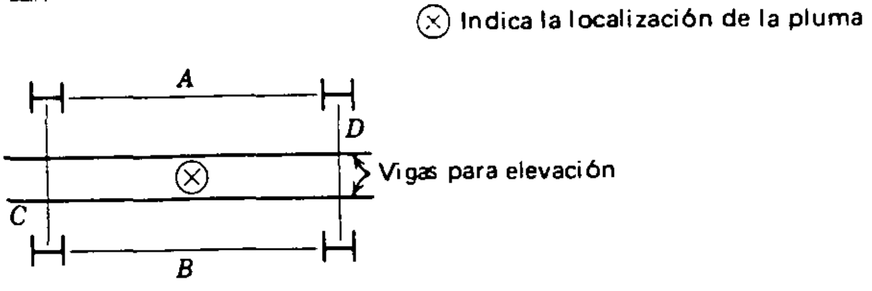


Figura 5.4.3 Colocación de las vigas para elevación. Arreglo normal; las vigas *C* y *D* son adecuadas para soportar carga en su parte central.

colocarse diagonalmente (Fig. 5.4.4) de manera que un extremo de cada viga de elevación descanse sobre cada una de las cuatro vigas que se conectan a las columnas diagonalmente opuestas en el panel. O bien, después de revisar las vigas de soporte en conjunto con las situadas un piso por debajo, puede ser mejor planear la colocación de puntales entre las vigas permanentes de los dos pisos, en vez de mover las vigas de elevación; o puede usarse un par de vigas de elevación cerca de las columnas (Fig. 5.4.5) en los extremos opuestos de los miembros de apoyo en cualquiera de los lados del panel, colocando un segundo par de vigas de elevación en angulo recto y por encima de ellas, lo bastante cercanas unas a las otras como para soportar el bloque inferior de la pluma o la grúa levadiza. Puede ser conveniente suministrar un par de vigas conectadas a las columnas, fabricadas con vigas que en general se tienen en existencia en el almacén o como parte del material de la obra falsa.

Al revisar los elementos de apoyo es necesario considerar las longitudes sin soporte de los miembros, reduciendo los esfuerzos permisibles de acuerdo a éstas. Esto es de particular importancia al revisar las vigas de apoyo que quedan situadas en ángulo recto con la dirección del mástil cuando se efectúan maniobras de descarga y

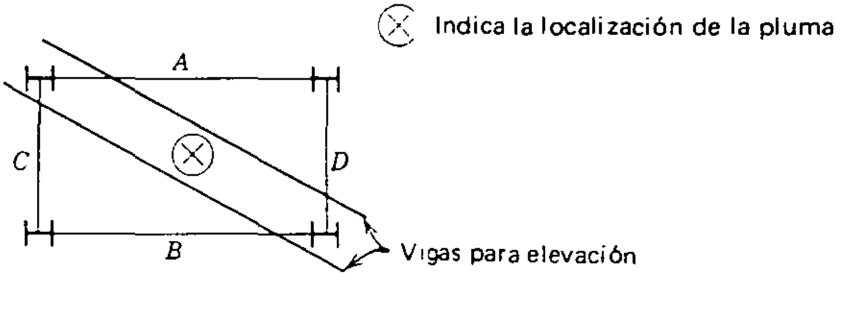


Figura 5.4.4 Colocación de las vigas para elevación. Las vigas *A, B, C* y *D* no son adecuadas por flexión pero sí por cortante cerca de las columnas.

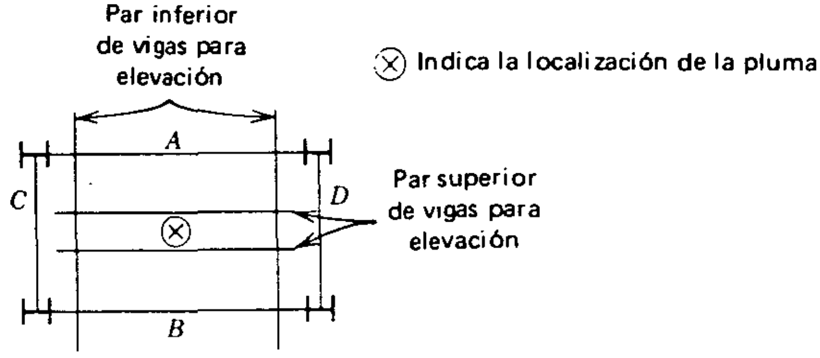


Figura 5.4.5 Colocación de las vigas para elevación. Las vigas A y B son adecuadas si las vigas de elevación se colocan cerca de las columnas; C y D no son adecuadas para soportar cargas.

cuando se colocan las piezas más pesadas; además es necesario porque siempre existe la posibilidad de que giren las vigas, aunque los tirantes de la base de la pluma estén colocados y se suponga que mantienen fija dicha base.

El giro de las vigas puede afectar el valor de soporte de los tirantes de la base. A partir de este análisis se revisa que los miembros de apoyo y sus conexiones trabajen con seguridad, y es en esta etapa cuando es importante notificar al departamento de ingeniería del fabricante las cargas que soportarán las conexiones de los extremos, de manera que puedan detallarse y fabricarse de acuerdo a éstas.

Cuando es necesario conectar miembros temporales a las columnas permanentes, debido a que las vigas permanentes no son del tamaño adecuado, debe informarse de esto al departamento de ingeniería para que puedan detallarse en las columnas las conexiones para los miembros temporales, antes de enviar los dibujos al taller para que se fabriquen las piezas. Si el fabricante debe suministrar los miembros temporales, debe recibir los detalles a tiempo para embarcarlos oportunamente para su colocación.

Se debe revisar la estructura del nivel más alto, una vez que se ha decidido cómo se desmantelarán y quitarán los aparejos de montaje. Si se va a utilizar una pluma tipo Chicago, se estudia la columna a la que se conectarán; de manera similar, se debe revisar el posible esfuerzo de la estructura para soportar un aparejo liviano o una grúa de pescante ligero. Si la estructura permanente se va a usar para desmantelar los aparejos, es necesario analizar los miembros que pueden ocuparse y seleccionar aquellos que puedan servir para el propósito de desmantelarlos desde arriba.

5.5 *Plataformas de colocación*

Si se usan plumas de tirantes, las plataformas de colocación se deben revisar para asegurarse que las plumas pueden continuar operando en la misma localización, ya sea en el nivel superior o en el inferior. Si el uso de las plataformas de colocación implica la eliminación de la estructura a la cual se fijaron los tirantes, debe tomarse una decisión entre mover lateralmente las plumas dentro del edificio a una posición que permita colocar con seguridad los tirantes, o bien acortar el mástil y el aguilón, dejando las plumas en las mismas posiciones relativas. En cualquiera de estos casos, si el mástil es ya muy pequeño y no alcanza hasta la calle para descargar materiales, debe usarse un aparejo auxiliar para colocar dichos materiales en la plataforma; debe revisarse la resistencia de ésta, para confirmar que soportará la carga de los elementos que se izarán desde la calle y se depositarán temporalmente en ella, más la carga de cualquier otro aparejo que se utilice para la descarga. Por lo general se tendrá una concentración de elementos estructurales mucho mayor que en la parte inferior de la estructura, debido a que son menores las áreas disponibles. También debe tomarse en cuenta el suministro de energía al malacate de esta pluma auxiliar, el cual casi siempre se localiza en el mismo nivel de la plataforma.

La pluma auxiliar puede ser de tirantes o de patas rígidas; si es del primer tipo, se debe diseñar en forma triangular, horizontal, para conectar la parte superior del mástil a la cara de la estructura que se montará por encima de ese nivel, ya que existirá poco espacio entre el borde del edificio y la estructura que se montará por encima de la plataforma, previendo además que el pasador de émbolo se pueda fijar a dicha estructuración en vez de conectarlo a la estrella y a los tirantes. Además, debe revisarse la columna a la que se fijará la estructura triangular que soportará el pasador, para asegurarse de que resistirá las fuerzas horizontales que se le aplicarán.

Si se tiene disponible, puede usarse una pluma ligera de patas rígidas, ensamblándola en la plataforma con la pluma de montaje antes de que ésta haya colocado la estructura definitiva. Esta pluma de patas rígidas debe fijarse bien a la estructura, para que resista las fuerzas de levantamiento que se presentarán al izar cargas desde la calle y colocarlas en la estructuración de la plataforma.

En los planos que se suministrarán al superintendente y a sus capataces debe indicarse si la grúa de tirantes se va a mover lateralmente o si se colocará una grúa auxiliar en la plataforma en vez de utilizar una grúa colocada en el piso, o si se reforzará la plataforma para utilizarla como un área de descarga, o si el malacate se va a mover a un nivel superior o a la plataforma.

Se deben analizar las columnas que están situadas por encima y cerca del área, si es que se usarán para fijar un mástil tipo Chicago para desmantelar y cargar la grúa auxiliar y su malacate. Tal vez éste sea el método más seguro y más económico.

5.6 Motores para malacates

Si el edificio es muy alto, el motor del malacate debe situarse en uno de los pisos superiores, para reducir el peso de los cables; de otra manera habrá mucho cable en el tambor, sobre todo cuando se están montando los pisos inferiores. Reduciendo el peso del cable que queda entre el pie de la pluma y el malacate, también puede reducirse el peso de las bolas de contrapeso; estas bolas se necesitan para que los cables de carga puedan bajar hasta el nivel de la calle cuando no llevan ninguna pieza. En cualquier caso, tan pronto se haya montado la estructura del nivel de la calle, el malacate debe sacarse de la excavación, en donde por lo general se sitúa al iniciar los trabajos, hasta el nivel de la calle, ya sea dentro de la estructura o fuera de ella si así lo permiten las normas locales o las reglas de la policía; esto facilitará mucho el sacar el malacate al terminar la obra, moviéndolo sobre rodillos, en vez de tratar de sacarlo del "agujero".

El malacate que se usará se selecciona escogiendo la tracción que se requiere en la línea, así como la velocidad de los tambores, con la cantidad de líneas que tendrá la pluma, las líneas principales de carga y el impulsor, si es que se va a utilizar. Debe tenerse cuidado de que cuando todo el cable esté en el tambor, con la pluma situada en el nivel más alto por encima del malacate, éste tenga todavía suficiente potencia sobre el cable para operar las líneas del mástil y las líneas de carga cuando se levante la carga máxima a esa elevación. Hay que tener en cuenta que tales cargas se izarán usando tres o más cables en las líneas de carga, al tomarlas desde el nivel de la calle, o bien desde una plataforma si las piezas se han llevado a otro nivel; si se tienen tres cables, habrá una longitud de cable igual a tres veces la altura de la parte superior del mástil al nivel de la calle, la cual tendrá que enrollarse en el tambor cuando se lleve una carga de estructura hasta el nivel de trabajo. Esta es otra razón para mover el malacate a un nivel superior cuando se está montando un edificio alto. También debe revisarse que la línea de carga tenga suficiente tracción cuando el tambor tiene todo el cable, cuando la pluma se coloca en su posición inicial.

5.7 Montaje con pluma viajera y pluma de patas rígidas

Una estructura que se va a montar en una excavación profunda puede montarse usando una grúa móvil según se ha descrito, pero en este caso una pluma viajera puede ser más aconsejable que una grúa. La pluma viajera se ensambla al nivel de la calle, fuera de la excavación y con contrapesos adecuados, de manera que pueda ir montando una nave por delante desde la excavación hasta el nivel de la calle, o bien se usa una grúa para montar la primera nave hasta el nivel de la calle y se ensambla la pluma viajera sobre esta estructura; después, la pluma se fija a la estructura y se continúa montando hasta el nivel de la calle, montando repetidamente nave a nave y moviéndose hacia adelante. Al llegar al extremo lejano del edificio, la pluma viajera monta la estructura del siguiente nivel, retrocediendo según se va completando cada nave, hasta que por último llega de nuevo a la calle, donde se puede desmantelar y sacar de la obra. Cualquier necesidad de reforzar la estructura debe comunicarse al fabricante antes de que se procesen los miembros que se han de modificar.

Los hangares, los cobertizos para trenes, los salones para convenciones y estructuras similares se prestan para ser montados con pluma viajera en vez de usar grúa; en caso de utilizar grúas, algunos tipos de plumas viajeras pueden sustituir a la obra falsa que se requeriría como soporte temporal de las armaduras de techo, trabes o arcos; sin embargo, aunque se necesite obra falsa, la facilidad de movimiento de las grúas debe sopesarse en función del costo de ensamble de una pluma viajera y las posibles dificultades que representa el moverla.

La pluma de patas rígidas es el tipo más adecuado para montarse sobre una plataforma viajera. Su uso será muy satisfactorio cuando se puede instalar en una posición desde la cual pueda montar toda la estructura, en vez de colocarla sobre una plataforma. Lo mejor para estas plumas es usar tirantes y anclajes adecuados, pero si no se puede disponer de ellos, deben suministrarse suficientes contrapesos para resistir los tirones de las cargas que se manejarán; la mayoría de las plumas de patas rígidas tiene mayor capacidad a distancias grandes que las plumas con tirantes, pero les falta la movilidad de una grúa de alta capacidad y mástil largo. Se debe comparar el costo de embarque, manejo, instalación y desmantelamiento, así como el tiempo requerido para tenerla lista para trabajar, con los costos de una pluma de tirantes o una grúa.

Con frecuencia, una pluma de patas rígidas puede montarse sobre una torre lo bastante alta como para que el mástil "libre" por completo la estructura terminada. Si se cuenta con un mástil lo bastante largo como para abarcar toda el área, y si la pluma esta conectada perfectamente a la torre y si ésta se tiene fija por medio de contravientos o contrapesos, un aparejo de este tipo puede montar una estructura sin imponer ninguna carga de montaje a la estructura permanente (ésta sería la misma condición si se usara una grúa-torre fija de capacidad comparable). En el caso de un edificio con conexiones soldadas, esto puede ser ideal, pues si se piensa montar este tipo de estructura con una grúa de tirantes o una grúa levadiza, cada uno de los pisos ya montados debe estar bien ajustado y soldado, antes de que la grúa se pueda cambiar de nivel con seguridad, para montar el siguiente nivel. Con la pluma de patas rígidas colocada sobre una torre se permite el montaje de un nivel completo y comenzar el montaje del siguiente, tan pronto como se hayan soldado suficientes conexiones, o se hayan colocado suficientes tornillos de montaje para soportar sólo las cargas de la estructura del siguiente nivel.

Si el mástil puede alcanzar toda la altura de la estructura, pero no toda la longitud del edificio, puede montarse la parte completa a la cual tiene acceso, y pasar después la torre y la pluma a una nueva posición desde donde pueda montar otra sección completa, del frente a la parte posterior, moviendo repetidas veces la torre después de que se ha montado cada sección, hasta terminar toda la estructura. Una grúa-torre viajera se puede usar de modo similar si son adecuadas la longitud y capacidad del mástil.

5.8 Montaje con equipos varios

Si se planea usar una sola grúa y se sabe que en la estructura sólo existen una o dos piezas con peso excesivo, una grúa que tenga capacidad para manejar todo el resto de la estructura puede ser la solución más económica, utilizando junto con dicha grúa un poste-grúa para manejar las piezas pesadas.

Cuando la estructura está situada en una localidad demasiado alejada, pueden ser factores decisivos los malos caminos que conducen al lugar de la obra, o los puentes que sean inadecuados para soportar equipo pesado; o bien, la cantidad de estructura no justifica el envío de grúas, plumas o equipo similar. En este caso, el aparejo más adecuado puede ser un poste-grúa.

Ya sea que un poste- grúa se use solo o junto con otro equipo, los dibujos del plan de montaje deben mostrar o señalar la localización del sitio de ensamble o de colocación del poste-grúa, así el lugar donde se instalarán los anclajes para cambiarlo de lugar la primera vez y, si son necesarios, los anclajes para bajarlo después de terminar el montaje. Se deben indicar todos los movimientos del poste-grúa o agregar una nota para que se use el equipo principal de montaje para colocar el poste-grúa y después cambiarlo de lugar y colocarlo de nuevo para la siguiente operación.

Se deben indicar con claridad todos los detalles del uso del poste-grúa, ya que muchos superintendentes de montaje no están muy familiarizados con éste y se les debe guiar de modo correcto durante las operaciones.

El uso de un poste-grúa se debe limitar a estructuras de poca altura y con materiales ligeros. El diseño del aparejo debe ser tal que todos los miembros puedan ser manejados por uno o dos hombres y que una cuadrilla pequeña pueda mover todo el aparejo ensamblado.

Cuando se tienen pocas piezas pesadas, el peso de un embarque por medio de camión puede exceder el límite legal de las carreteras; entonces la entrega se hace por ferrocarril hasta un punto que tal vez no sea el que conviene para el aparejo con el que se montarán dichas piezas. En este caso, al preparar el plan de montaje debe establecerse el método de descarga. Con frecuencia, el encargado de embarques enviará varias trabes, armaduras o vigas pesadas en una sola carga y el montador puede no necesitar todas esas piezas a la vez; puede necesitarlas a intervalos y no cuenta con suficiente espacio para almacenarlas cerca de donde se utilizarán en la estructura. También, el equipo que se usará para descargar el grueso de la estructura puede no tener la capacidad para manejar estas piezas pesadas; en tales casos, una cabria puede ser el mejor equipo para descargarlas y almacenarlas hasta que se necesiten.

En la actualidad existen helicópteros con potencia de izaje moderada. Cuando se hace una modificación o una adición en el techo de un edificio alto existente, puede usarse con eficacia para entregar y colocar los miembros de la estructura, en caso de que el peso de éstos quede dentro de su capacidad de izaje. En el caso de una adición a la parte superior de una estructura ya en uso, con esto se elimina la colocación de equipo de montaje que puede ser muy difícil y costosa. La protección de un techo existente para soportar un equipo de montaje puede ser un problema grave.

5.9 Estudio de los dibujos y de las especificaciones del contrato

Las especificaciones se deben estudiar para tomar en cuenta cualquier restricción al preparar el plan de montaje y seleccionar las herramientas y el equipo. Debe confirmarse si existen algunas medidas relacionadas con la secuencia de montaje, la coordinación con otros gremios, si hay demoras durante el transcurso del montaje para permitir que se hagan algunos otros trabajos necesarios para seguir avanzando, si se indica la localización de los malacates, compresores y máquinas de soldar. El contrato se debe revisar para confirmar si existen conceptos relacionados con la colocación física de los miembros de la estructura y con los trabajos subsecuentes, como pudiera ser algún requisito en que el trabajo de algunos otros gremios debe terminarse en algún punto de la secuencia de montaje.

Al revisar el contrato y las especificaciones, debe anotarse todo lo que requiera la atención del superintendente de campo, aunque no modifique directamente la preparación del plan de montaje. En esto debe incluirse los requisitos de nacionalidad de los trabajadores, el lugar de residencia, si serán o no trabajadores sindicalizados, los salarios mínimos que se pagarán y si se pagarán en efectivo o con cheque, las limitaciones en las horas laborales por día y los días laborables por semana, los requisitos de pago por tiempo extra, la inspección requerida, el uso de electricidad, gasolina o diesel, el uso de sopletes de corte, las instalaciones o servicios que serán suministrados por otros, como luz, calefacción y servicio de vigilancia. Después de anotar toda esta información detallada deben estudiarse con cuidado los planos que formen parte del contrato, confirmando los tamaños de las piezas poco usuales y los pesos de las piezas pesadas, así como estudiando el área, para dividirla para efectos de embarque y para calcular la capacidad del equipo que se usará.

En esta etapa se determinan la pieza más pesada que se colocará, así como las cargas más pesadas para efectos de descarga, y se confirma si se dispone de una pluma o una grúa de la capacidad suficiente para izar tales cargas a la altura requerida, con un mástil de la longitud calculada; si no se tiene, puede necesitarse un mástil más corto para manejar las cargas y algún equipo adicional, reduciendo así el área que se pensaba cubrir con un aparejo. Si dos aparejos pueden descargar y colocar juntos las piezas pesadas, ésta será la solución más económica, ya que mientras más ligera sea la pluma o la grúa que se use, más rápido puede operar y puede montar más estructura con un cierto tiempo. Sin embargo, el costo de embarcar, manejar, ensamblar, ajustar y desmantelar dos grúas o plumas ligeras puede ser mayor que en el caso de un equipo más pesado y de mayor capacidad, con un mástil más largo; entonces debe compararse el costo de instalar dos equipos y tener un montaje más rápido, con el costo más bajo de instalar un solo equipo, pero con un montaje más lento. Al estudiar la estructura, en caso de que ésta tenga que soportar una pluma o una grúa levadiza, puede descubrirse que soportará mejor dos aparejos situados en diferentes posiciones, que un solo aparejo pesado.

Los planos se revisan también para ver qué piezas se deben ensamblar previamente en el taller; estos ensambles pueden estar restringidos debido a las instalaciones del fabricante, a sus limitaciones para cargar el material, las alturas y anchos libres de trenes, camiones o barcazas, así como a sus capacidades, al equipo del transportista y a la capacidad del equipo seleccionado para la descarga y el montaje de los ensambles. Las armaduras deben ensamblarse en el taller en forma tan completa como sea posible aun en el caso de que con esto se haga necesario usar un vagón especial de ferrocarril o una maniobra especial por camión. Deben considerarse tanto la baja velocidad de un ferrocarril con vagones especiales, como el gasto que representa una maniobra especial con camión, la que a menudo requiere de una escolta policiaca o de otro tipo, en función de las ventajas que representa la facilidad de colocar en su sitio una sola pieza pesada, en lugar de ensamblar, ajustar y atornillar, remachar o soldar en el lugar de la obra muchas piezas pequeñas; a menudo, este tipo de ensambles en campo requieren de obra falsa, con el consiguiente gasto adicional, el tiempo y el peligro que esto implica.

Las trabes muy peraltadas deben estudiarse; algunas de ellas pueden ser demasiado peraltadas o muy largas para transportarlas con seguridad, y requerirán un empalme vertical u horizontal, según sea el caso. En otras palabras, deben determinarse los conceptos que no pueden ensamblarse en taller; se manejarán, ajustarán y conectarán en forma permanente piezas adicionales, todo lo cual puede aumentar los costos de montaje y demorar la terminación de la obra.

Además de revisar los planos o dibujos para localizar los ensambles que se harán en el taller o en el campo, debe prestarse atención a las piezas pesadas, peraltadas y de forma poco usual, para decidir si el fabricante necesita colocar algunos ángulos auxiliares, soldar algunas placas para levantar dichas piezas, o bien omitir estas piezas y que el montador use estrobos u otro tipo de equipo de izaje; en losas pesadas pueden necesitarse ángulos o placas, a menos de que los ángulos de conexión para las columnas estén colocados en la losa, en cuyo caso será suficiente barrenar agujeros en los patines salientes.

En general, las losas pequeñas se sueldan a las columnas, en taller, siempre y cuando los pernos de anclaje estén ya colocados; de otra manera, puede ser peligroso montar una columna, sin soporte, conectada a una losa. Los emparrillados deben ensamblarse en taller de modo tan completo como sea posible, dejando suelta la losa del emparrillado en caso de que el peso del ensamble sea muy grande al colocarla y el equipo con que se cuenta no pueda manejarla. Se debe confirmar si las vigas dobles requieren de algún ensamble en el taller o en el campo y debe decidirse si los dinteles de las vigas de antepechos se embarcan ensamblados o si es posible que se dañen en tránsito, de tal manera que sea mejor embarcarlos sueltos y ensamblarlos en el lugar de la obra, antes de montar las vigas de las que se suspenden.

Si se van a usar sujetadores para colocar columnas, deben darse al fabricante los tamaños de los agujeros para el pasador de las placas de empalme, que se deben colocar entonces en el extremo superior de las columnas; por lo general, el diámetro de estos agujeros variará entre 2 pulgadas (5.1 cm) para las columnas ligeras, hasta 2 1/2 pulgadas (6.4 cm) para columnas pesadas. En algunos diseños se indican las placas de empalme en los extremos inferiores de las columnas, lo cual evita el uso de este equipo de izaje, pues sería necesario voltear las columnas para montarlas. En estos casos debe usarse un madero de algún tipo, de manera que enderezando la columna sobre el madero, las placas de empalme no dañen el piso sobre el que se está volteando la columna; si se va a aplicar esta solución, se debe indicar en las instrucciones, y el madero debe incluirse en el embarque de las herramientas que se envíen a la obra.

En algunos empalmes muy ajustados para los cuales no se suministran placas de relleno, es conveniente que en la fabricación se omitan una o dos de las hileras superiores de tornillos, o parte de la soldadura, de modo que las placas de empalme puedan abrirse ligeramente para facilitar la conexión; de manera similar, en el empalme de una trabe o de una armadura, si se deja al campo la terminación de la primera hilera de remaches o tornillos, o de parte de la soldadura, esto ayudará al montador, aunque tenga que efectuar algún trabajo que en principio debiera hacerse en el taller. Si el montador realiza algún trabajo que debería haberse hecho en el taller, esto se podría compensar cuando el fabricante haga algunos trabajos adicionales que solicite el montador, como suministrar ángulos para izaje o asientos para montaje.

Cualquier empalme complicado se debe revisar para asegurarse de que el material de dicho empalme esté sujeto a una de las partes de la armadura o de la trabe, de manera que las piezas puedan ensamblarse en campo sin mucha dificultad. Cuando sea factible, se debe hacer el empalme de manera que parte del material esté en uno de los lados de una pieza y el resto en el otro lado de la pieza adyacente; entonces puede hacerse la conexión colocando de lado y en su lugar, cualquiera de las piezas. En algunas ocasiones, para facilitar la conexión es aconsejable embarcar todo el material del empalme atornillado a uno de los lados del punto de ensamble, aunque esto signifique trabajo adicional para el campo, ya debe fijar en forma permanente el material que en principio debería haberse ensamblado en taller.

5.10 Estabilidad lateral

Debe revisarse la estabilidad lateral de las armaduras y las trabes cuando se izan por el centro, por los extremos, o por dos puntos intermedios, según sea el caso. Puede encontrarse excelente información técnica para revisar la estabilidad lateral, en un artículo sobre "Resistencia de vigas, determinada por el flambeo lateral" (Strength of Beams as Determined by Lateral Byckling) de Karl de Vries F. de la ASCE, publicado en el boletin No. 2326 de las *American Society of Civil Engineers Transactions* Vol. 112, 1947, págs. 1245 a 1320.

Si son inestables lateralmente, debe decidirse si se refuerzan los miembros, si se agrega un arriostramiento horizontal temporal hecho de ángulos o canales, si se usa un atirantamiento a base de soleras, varillas o cable de alambre, o si se fija una viga provisional al patín superior de la pieza en cuestión, colocándola con el alma sobre dicho patín superior. El atirantamiento (Fig. 5.10.1) se hace uniendo los dos extremos de una pieza por medio de varillas, soleras, cable de alambre o material similar, colocados sobre unos puntales intermedios que sobresalen del patín de la pieza que está a compresión al montarla; sobre estos soportes o puntales se fijan las varillas, roscadas

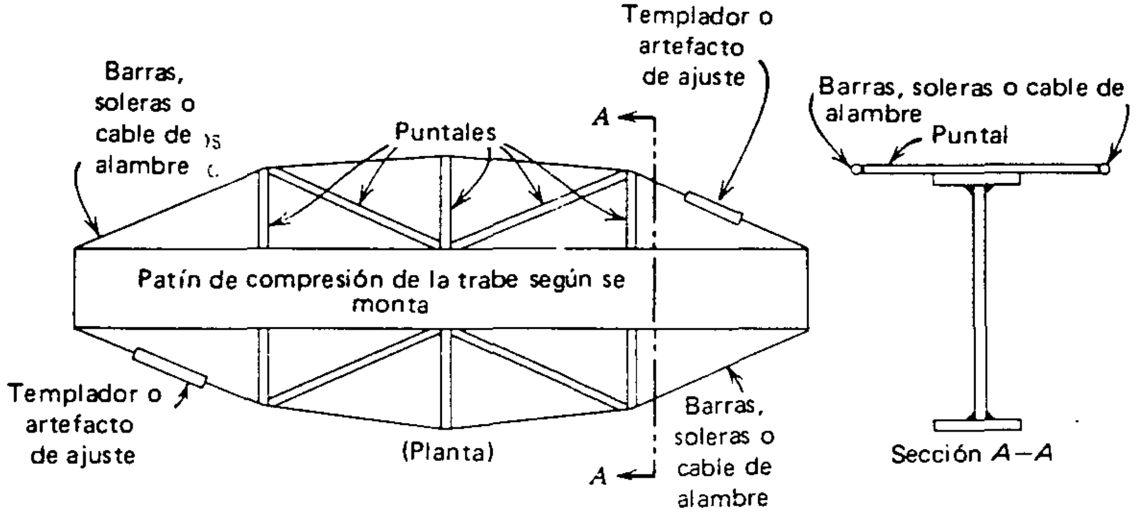


Figura 5.10.1 Atirantamiento. Distribución esquemática, mostrando el atirantamiento de una trabe lateralmente inestable.

para permitir su ajuste, o las soleras con algún medio para cambiar su longitud, o los cables de alambre con templadores. En realidad, éste atirantamiento forma con la pieza una armadura lateral horizontal. No se recomienda el cable de alambre para este fin, porque existe el peligro de que se ajuste de manera desigual en uno de los lados, sobre todo si el cable es nuevo; esto puede ocasionar que la viga se doble en el centro, en uno o ambos extremos, o que falle por completo. Las trabes y armaduras deben revisarse para confirmar si deben embarcarse o acarrearse en posición vertical o si pueden "acostarse" sin tener deformaciones o deflexiones excesivas durante el transporte, o cuando se voltean para montarlas.

5.11 Detalles de conexiones

Algunas veces, en los planos de diseño se muestran conexiones que son tan difíciles de hacer, que un pequeño cambio puede facilitar el montaje y a veces hacer que la fabricación sea más económica. Es necesario estudiar los detalles para facilitar la construcción sin incrementar los costos del fabricante, y debe pedirse al departamento de ingeniería de éste que modifique los detalles complicados si con ello se puede simplificar y expeditar el montaje.

Por ejemplo, una conexión con ángulos dobles por un solo lado (Fig. 5.11.1), colocando dichos ángulos espalda con espalda sobre una viga cabezal, permitirá balancear con facilidad una viga secundaria para colocarla en su sitio; ésta es una conexión mejor, si se diseña en forma adecuada, que un par de ángulos de conexión colocados en el alma de la viga (Fig. 5.11.2), y su montaje es más fácil que el de una conexión en cuchilla. En cualquiera de los casos, el trabajo de taller puede ser el mismo.

Con la conexión en la viga secundaria, el personal debe alinear los agujeros de los ángulos con los del alma del cabezal. Cuando se conecta la viga que va por el lado opuesto del alma, los tornillos de montaje normalmente se deben jalar hacia atrás lo suficiente para que los ángulos de conexión de la segunda viga libren los extremos de los tornillos, de manera que la primera viga cuelga de los tornillos sin tuerca, quedando éstos sujetos sólo por el alma del cabezal.

Si no es posible cambiar este tipo de detalle, debe solicitarse un asiento de montaje para trabajar con más seguridad, para apoyar las vigas en él mientras se sacan los tornillos para conectar las piezas al alma del cabezal; este asiento puede ser sólo un ángulo pequeño, pero debe tener suficientes conectores o soldadura de taller para soportar el peso del miembro y de la persona que está haciendo la conexión.

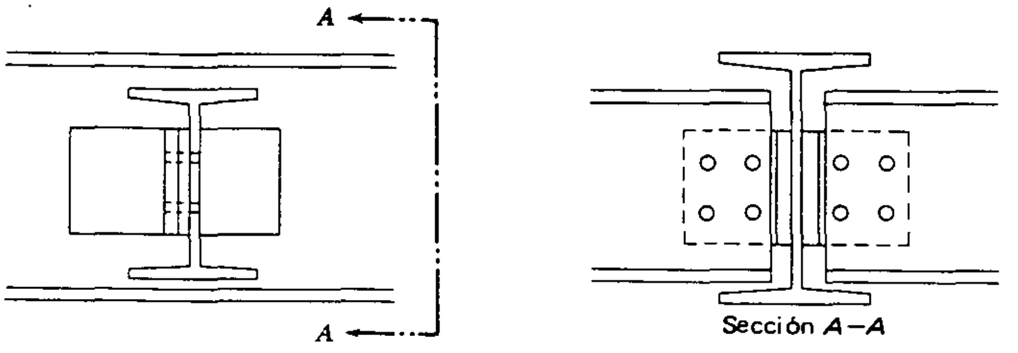


Figura 5.11.1 Conexiones con ángulos dobles.

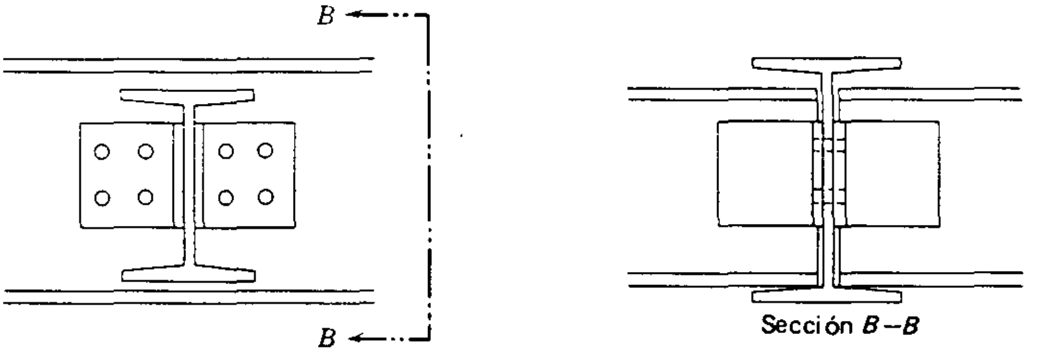


Figura 5.11.2 Angulos de conexión en vigas secundarias.

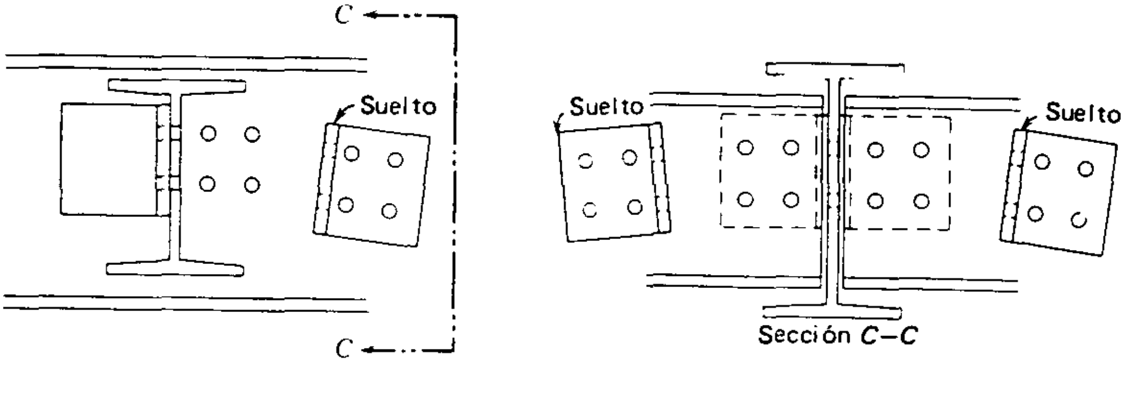


Figura 5.11.3 Conexiones con un ángulo suelto.

Cuando no se suministran conexiones en la parte inferior de una columna, por debajo de las vigas que se conectan a ella, deben suministrarse también asientos para montaje, para la seguridad de las personas que hacen la conexión.

Se debe pintar uno de los lados de una conexión a base de ángulos espalda con espalda, o marcarse en el diagrama de montaje, para asegurar que la conexión se haga en forma correcta.

En ocasiones, uno de los ángulos se fija de modo permanente al cabezal en el taller (Fig. 5.11.3) y el segundo ángulo se deja suelto, atornillándolo temporalmente para el embarque. Con esta solución, el montador necesita agarrar el ángulo suelto mientras hace la conexión, lo cual implica un riesgo si se deja caer el ángulo suelto y lastima a alguna persona en los niveles inferiores, pero elimina el peligro que representa jalar los tornillos de montaje para hacer la conexión en el otro lado del cabezal y no se requiere un asiento. En este caso, se dejan en su sitio los tornillos de montaje que conectan la primera viga al ángulo fijo y no se necesitan tornillos temporales a través del alma del cabezal.

La mayor parte de los gabinetes de diseño preparan dibujos de "diseño general" de conexiones y condiciones complicadas; es conveniente que el montador obtenga estos dibujos antes de que se usen para hacer los detalles de fabricación. De esta manera, si hay alguna objeción debido a incrementos de costos o conexiones que no sea fácil hacer, puede cambiarse el detalle antes de que sea demasiado tarde.

5.12 Entregas

Se debe dar al fabricante la división de la estructura en las áreas de entrega, así como los sectores que se requieren tan pronto se haya tomado la decisión sobre el tipo y cantidad de empalmes que se desean en las columnas y la capacidad y tipo de equipo que se usará; esta información debe incluirse en las hojas de detalles con las que se fabrica la estructura. Las longitudes de las columnas dependen de la localización de los empalmes y el fabricante no puede ordenar el material hasta conocer este dato.

Las entregas se deben seleccionar de tal modo que se cubran algunas áreas de manera que desde el taller se haya hecho una clasificación automática. El peso del material que se entregue en dichas áreas debe mantenerse dentro de las cantidades razonables para que ninguna entrega sea menor que la carga mínima que puede acarrearse en un camión o en un vagón de ferrocarril.

En caso de usar una pluma atirantada, las áreas de entrega deben quedar aproximadamente en el espacio que queda entre dos o tres tirantes, para eliminar los movimientos innecesarios del mástil de un área situada entre dos tirantes a otra de las áreas entre dos tirantes diferentes. En ocasiones es razonable dividir el área que cubre una pluma en cuatro partes formadas por líneas perpendiculares que se cruzan al pie de la pluma.

En caso de usar una grúa móvil para el montaje no sólo es importante el peso de las entregas, sino también el área cubierta; sería ideal que con cada una de las entregas se cubriera sólo el área que puede alcanzar el mástil de la grúa desde una cierta posición. Cuando la estructura es larga y no muy alta y si se va a usar una grúa, toda el área debe dividirse para efectos de embarque y montaje de manera que las entregas puedan descargarse y montarse con las menores maniobras posibles de la grúa.

Como principio, se debe revisar un área de aproximadamente 100 x 100 pies (30.48 x 30.48 m) y si con esto se obtienen cargas mejores que las mínimas, o cuando menos lógicas, la solución será una ayuda para el montador y quizá no represente problemas para los encargados de los embarques. Los conectores, tales como remaches o tornillos, se dividen en entregas mayores, cubriendo áreas mayores de manera que puedan embarcarse en cargas completas por camión o ferrocarril, a menos que el fabricante esté dispuesto a embarcarlos junto con el material de la entrega en que se utilizarán.

El caso de un edificio industrial largo es una excepción para dividir el área según se ha sugerido, pues por lo común se montan con una grúa móvil. Las columnas y el contraventeo lateral se deben embarcar en entregas separadas, ya que pueden montarse antes de que llegue el resto del material; para conveniencia del fabricante en sus embarques, el resto del material puede dividirse en entregas que le permitan embarcar las armaduras o trabes transversales separadas de los miembros secundarios y el contraventeo.

5.13 Hileras o pisos

A menos de que en los dibujos de diseño se indique específicamente que habrá más de dos pisos en una sola de las hileras que se montarán, en la mayor parte de los edificios que se montan con grúa de tirantes, grúa levadiza o grúa-torre fija, es conveniente hacerlo en hileras de dos pisos. Algunos diseñadores utilizan una misma sección (tamaño y peso por unidad de longitud) de columnas para tres pisos, en cuyo caso es lógico empalmar las columnas en tramos de tres pisos; con esto se incrementa la cantidad de piezas que se deben descargar, seleccionar y distribuir sobre el piso de trabajo, en caso de usar una pluma atirantada, una grúa levadiza o una grúa-torre fija o sobre el piso, en caso de usar una grúa de orugas, una grúa montada sobre camión, o una grúa-torre montada sobre camión. Con frecuencia, esto resulta en accidentes, ya que hay mucho menos espacio para moverse. Si se monta en hileras de dos pisos, en el área de trabajo habrá sólo una cantidad de estructura igual a dos tercios de la que habría en el caso anterior. En caso de usar una grúa de tirantes para el montaje, un cambio de nivel de tres pisos, con la altura promedio de 10 pies (3.05 m) entre piso y piso, a menudo es más difícil y costoso que un cambio de nivel de tres pisos; si los entrepisos son más altos que el promedio, puede ser necesario hacer el cambio de nivel en dos etapas, lo cual es más costoso y peligroso que un cambio de nivel en una sola etapa.

Si el montador insiste en usar hileras de dos pisos de altura cuando los dibujos de diseño indican tres, el fabricante estará forzado a incrementar la cantidad de empalmes en campo; lo cual implica material extra. También se debe hacer un empalme en taller en cada uno de los niveles correspondiente a los pisos en que cambia la sección de la columna; sin duda, esto no fue previsto por el fabricante cuando presentó su propuesta para el trabajo. Como una alternativa, puede elegirse eliminar los empalmes adicionales de taller y suministrar una sección más pesada en la longitud total del tramo de columna; esto también representa un costo adicional para el fabricante, no previsto en su propuesta, ya que muchos diseñadores rehusarán aprobar que el cliente o dueño pague el material extra.

Cuando debe montarse en hileras de tres pisos, puede ser necesario tener un empalme de campo adicional en las columnas contiguas a un aparejo de montaje que esté colocado en el interior de la estructura, uno o dos pisos por encima del piso de trabajo; esto es necesario para que el mástil pueda librar la estructura al montar elementos situados más allá de dichas columnas, sobre todo si el aparejo se ha localizado muy cerca de ellas o del cabezal que las une.

Las objeciones para usar tramos de columnas de tres pisos de longitud se eliminan cuando es conveniente tener columnas largas para montarlas con grúa desde el nivel del terreno, si la estructura se puede colocar sobre éste y la grúa puede manejar las piezas largas y por consiguiente más pesadas.

Algunas veces se pide al fabricante empalmar en taller algunos tramos diseñados para empalmarse en campo. En el caso de un edificio con núcleos de columnas, mientras más largos se fabriquen los tramos de dichas columnas, más económico será el montaje, siempre y cuando la grúa o la pluma tengan la capacidad y el alcance suficientes y que la columna pueda manejarse sin que se pandee. (Un edificio con núcleo de columnas es aquel en el cual los mismos miembros estructurales de acero son las columnas, con algunos puntales ligeros entre ellas; el resto de la estructura es de concreto reforzado. Usando columnas de acero, se requiere ocupar un área de piso más pequeña que si se usaran columnas de concreto para las mismas cargas, generalmente pesadas).

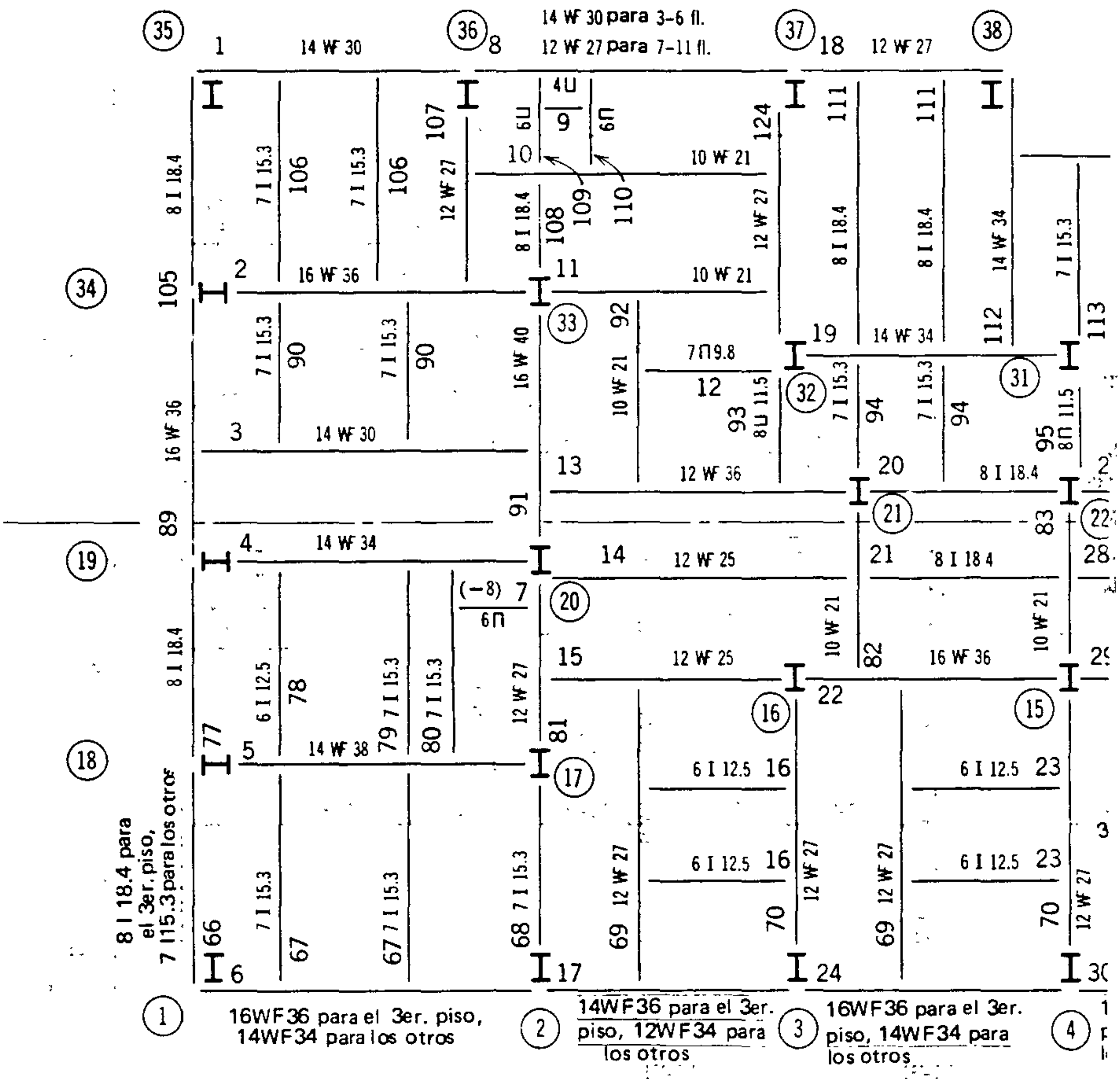
En este caso, la eliminación de los empalmes de campo reduce el trabajo de campo, eliminando el costo y el tiempo que representa hacer las conexiones permanentes en los puntos donde se diseñó un empalme de campo. Las limitaciones en las maniobras de carga, embarque, descarga y entrega, puede afectar la longitud que puede usarse de las columnas empalmadas en taller.

5.14 Diagrama de montaje

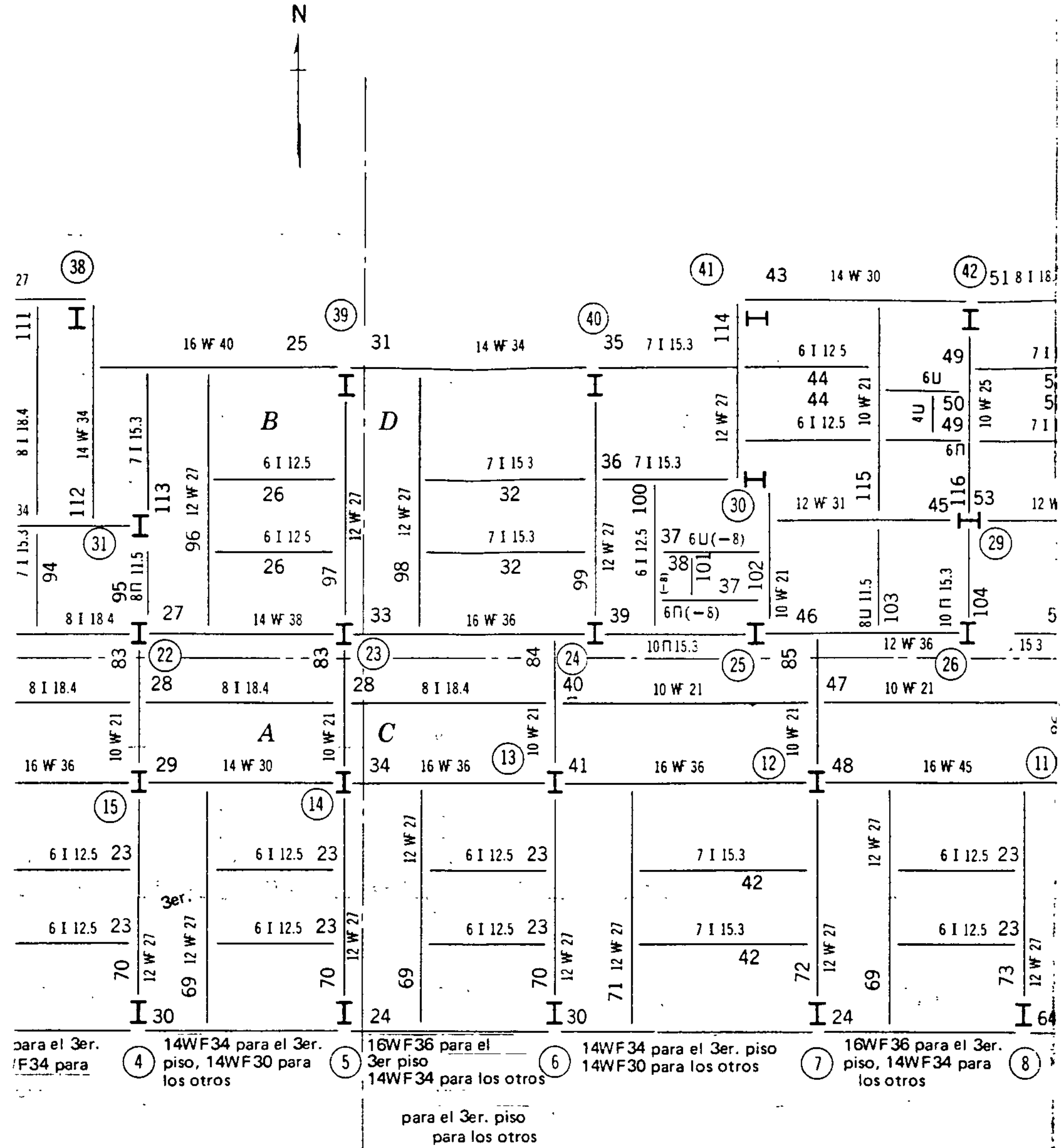
Debe haber coordinación con el departamento de proyectos, para asegurarse de que los diagramas de montaje se hagan de manera que sean útiles para el montador; un diagrama de montaje (Fig. 5.14.1) consiste en un plano lineal de cada piso de una estructura y una elevación o vista lateral cuando sea necesaria para localizar largueros, puntales y piezas similares. La vista lateral siempre es necesaria cuando en la estructura existe estructuración lateral.

En el diagrama de montaje se muestran las dimensiones entre columnas, y miembros intermedios, la localización de antepechos, dinteles suspendidos, etc., así como las elevaciones de los miembros de piso; en general estas elevaciones se indican como una dimensión por debajo del nivel de piso terminado, a menos de que la mayor parte de los miembros esté a la misma elevación, en cuyo caso esto se indica en una nota, con las excepciones que se anoten en el diagrama.

Se debe dar la sección de las piezas, por ejemplo, un 14 WF 228. [Esta descripción se basa en la designación aceptada para una viga, conocida como una viga de patines anchos de 14 plg de peralte nominal y que pesa 228 libras por pie. Para el tamaño real de este miembro es necesario referirse a manuales como los que publican la mayoría de los fabricantes de acero, o al "Manual of Steel Construction" publicado por el Instituto Americano de la Construcción en Acero (AISC); por ejemplo, una viga 14 WF 228, en realidad tiene 16 plg de peralte y patines con ancho de $15\frac{7}{8}$ plg]. Esta infor-



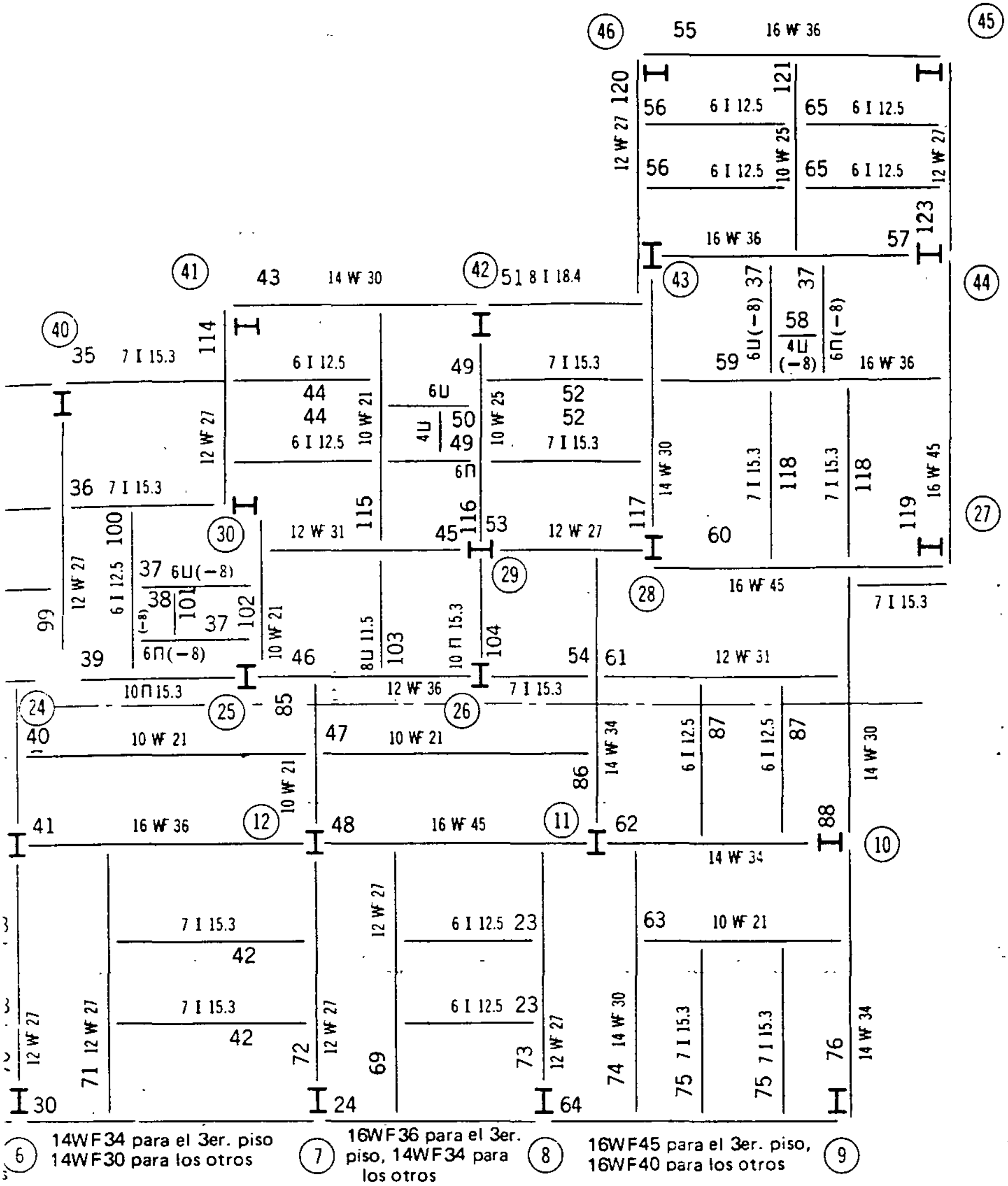
14 WF 30 para 3-6 fl.
12 WF 27 para 7-11 fl.
14 WF 30
12 WF 27
8 I 18.4
7 I 15.3
12 WF 27
10 WF 21
16 WF 36
16 WF 40
10 WF 21
7⊓9.8
14 WF 34
14 WF 30
12 WF 36
8 I 18.4
14 WF 34
12 WF 25
10 WF 21
14 WF 38
6 I 12.5
12 WF 27
16 WF 36
8 I 18.4 para el 3er. piso, 7 I 15.3 para los otros
16WF36 para el 3er. piso, 14WF34 para los otros
14WF36 para el 3er. piso, 12WF34 para los otros
16WF36 para el 3er. piso, 14WF34 para los otros



Pisos del 3o. al 11o.

Notas de montaje

Las conexiones de viga a columna deben ha
Todas las conexiones a menos de 3' de las col
tornillos de A.R.
Todas las demás conexiones deben hacerse c
máquina, a menos de que se indique lo contr



Notas de montaje

Las conexiones de viga a columna deben hacerse con tornillos de A.R.
Todas las conexiones a menos de 3' de las columnas deben hacerse con tornillos de A.R.
Todas las demás conexiones deben hacerse con tornillos tipo máquina a menos de que se indique lo contrario

mación se da para ayudar al montador a que encuentre algunas piezas en particular.

El concepto más importante en el diagrama de montaje es el número asignado a cada pieza individual; el sistema de numeración debe seguir una secuencia lógica en el diagrama. Si varias piezas son idénticas y pueden intercambiarse en la estructura, por lo general se les da el mismo número a todas; este número no sólo aparece en el diagrama de montaje, sino también en el plano de detalle con el que se fabrican todas estas piezas y el fabricante lo pintará sobre ellas. Cuando la pieza debe colocarse en una dirección definida, en general la marca se pinta cerca de un extremo y el número se muestra en el diagrama de montaje en el extremo respectivo.

Cuando una pieza se puede colocar en cualquier posición, el número se pinta cerca del extremo, pero en el diagrama se muestra dicho número en el centro de la línea que representa a la pieza. Algunos proyectistas no indican dónde se marca la pieza, pero en ese caso agregan en el diagrama una pequeña "X" o una "X" encerrada en un círculo, en el extremo de la pieza donde debe montarse el extremo marcado. Esta marca es mucho menor que el resto de lo indicado en la hoja y, con el uso que se da a los planos en el campo, a menudo la "X" se borra tanto que el montador tiene dificultades para encontrarla. Con frecuencia se trabaja cuando la iluminación es pobre, temprano cuando el Sol todavía no está alto, o tarde cuando ya se ha ocultado. A menudo se dejan los planos a la intemperie donde la lluvia, la nieve y el viento pueden estropearlos; es mejor omitir esta marca extra y mostrar los números de parte con números más gruesos en el extremo correspondiente al extremo en el cual se pintará la marca a la pieza en el taller.

Los tirantes o contravientos, sobre todo cuando existen muchos similares entre sí, se marcan sólo con una "X", seguida de un número igual a su longitud (en pulgadas o centímetros) y con una nota por separado en la cual se indica su diámetro; cuando los tirantes de contraventeo son poco comunes, se marcan individualmente y se coloca al tirante en sí una etiqueta metálica en la cual se indica el número de parte, o menos de que el tirante sea lo bastante grande como para poder pintarle la marca con claridad directamente encima.

Algunos proyectistas usan el número de dibujo de la pieza como parte del número que se da a la pieza en el diagrama de montaje; con esto se evita la necesidad de un índice de dibujos en que se muestren los números de parte contra los números de los dibujos. El montador debe evitar esto en forma definitiva, ya que no sólo hará confuso el diagrama de montaje, sino que hará más difícil encontrar las piezas para seleccionarlas, distribuirlas e izarlas, ya que el número incluirá una parte con el número del dibujo y otra con el número de la pieza; en algunos casos, esto representará seis o siete números y letras que la persona que esté localizando una cierta pieza debe recordar.

Además de toda la información ya mencionada, las conexiones complicadas deben indicarse por separado y en detalle; por ejemplo, si tres o cuatro piezas deben montarse en una secuencia especial, debe indicarse así. Cuando sea posible colocar un ángulo o un canal en forma poco usual, por ejemplo, un ángulo con uno de los patines hacia arriba o hacia abajo, o un canal con sus patines hacia un lado o hacia el otro, se debe dar una indicación por medio de una nota o un pequeño croquis de la sección cerca de la pieza y dentro del dibujo. Por lo general las secciones transversales se muestran cuando son necesarias para montar en forma correcta la estructura, o cuando la conexión es muy complicada como para mostrarla directamente en el diagrama lineal. Debe indicarse si es necesario mantener una contraflecha, por lo general con un croquis por separado, para prevenir al montador acerca de cómo mantenerla.

Para evitar un plano extra, algunas veces los proyectistas incluyen en el diagrama de montaje la información para colocar varillas o armaduras de refuerzo, o pisos, para ser usada por algunos otros contratistas; esto es aceptable siempre y cuando esta información se muestre en forma sencilla y los números de parte destaquen con claridad.

Mientras más limpio sea el dibujo, más fácil será para el montador encontrar en el campo el número de parte para localizar una pieza en el piso de trabajo o en el terreno; y viceversa, si tiene una pieza con el número marcado con claridad, le será más fácil saber dónde colocarla, al localizar pronto el número en su diagrama.

A menudo, para ahorrar dinero y esfuerzo, el fabricante no dibuja un nuevo plano, sino que hace copias de los dibujos de diseño y agrega en ellas las marcas para montaje; esto confunde el diagrama de manera innecesaria y la reproducción es tan mala, que el personal de campo no puede usarlos de modo satisfactorio. Si a pesar de esto, el superintendente trata de usarlos, perderá mucho tiempo tratando de descifrar las marcas entre todos los detalles que son necesarios sólo para el proyectista; de este modo, el tiempo de dibujo que ahorra el fabricante se pierde muchas veces más en el montaje. El montador tiene todo el derecho a que se le entregue un diagrama que sea fácil de usar y no atestado de detalles o información innecesarios, que le ayude a montar la estructura en forma correcta y rápida. Un diagrama confuso obstruye el montaje, un diagrama claro y bien dibujado lo expedita; un diagrama de montaje satisfactorio es tan necesario como un buen dibujo del plan de montaje. Los números de parte de las piezas deben sobresalir claramente, en comparación con el resto de la información, tal como las dimensiones y tamaños de las vigas.

Los números de parte se usan para ordenar el embarque de las piezas. se puede solicitar todo un grupo de piezas para algunas áreas, mostrando éstas de alguna manera en el diagrama; por ejemplo dividiéndolo por medio de líneas interrumpidas o punteadas.

En el diagrama debe agregarse una letra grande para identificar cada área (Fig. 5.14.1).

Cuando la estructura se selecciona y distribuye en el lugar de la obra, con frecuencia el número de parte se pinta con crayón en el patín superior de cada pieza, porque es difícil ver dicho número si está marcado en el alma y se tienen varias vigas una junto a la otra, con el espacio suficiente sólo para colocar un estrobo de montaje; si se puede convencer al fabricante de que agregue el número de parte en el patín superior de las vigas, ahorrará trabajo al montador, pero éste es un gasto extra y puede poner objeciones, aunque también le ayudaría si almacena las vigas muy cerca una de la otra antes de cargarlas y embarcarlas.

Además del número de parte pintado en el alma, por lo general el fabricante agrega también el número de su contrato, el número del dibujo con el que se fabricó la pieza y una marca de inspección; si las piezas de la estructura se han dividido por áreas por conveniencia del montador, esta marca debe mostrarse en el dibujo de montaje, en las hojas de detalle y en la pieza misma.

Cuando hay varias áreas similares, pero diferentes, como tiros de elevador, cúpulas o domos, algunos proyectistas tratan de hacer un solo croquis y agregan diferentes marcas para indicar el área a donde pertenecen; esto puede confundir tanto al montador que tiene todo el derecho a exigir un croquis por separado para cada área. Lo anterior no es aplicable cuando el plano de un piso es similar para varios niveles y el número de la pieza se indica en el piso al que pertenece.

La marca pintada en la pieza debe indicar no sólo su número de parte, sino también el piso o nivel en el que debe montarse. En el plano de montaje puede aparecer una marca que diga 42 (el número de la pieza) 3er. piso, e inmediatamente por debajo de ella: 42, del 4o. al 10o. pisos. En este caso, la marca de la pieza diría "42" con un pequeño "3" encerrado en un círculo, y todas las otras piezas se marcarán con el "42" y un pequeño "4-10" encerrado también en un círculo.

En este caso, cualquiera de las vigas marcadas "4-10" puede montarse en cualquiera de esos pisos. Si se entrega un dibujo y la misma secuencia de números, después de montar varios pisos, una cuadrilla de montaje bien capacitada sabrá dónde montar algunas piezas sin hacer referencia al dibujo de montaje.

Debe solicitarse el tamaño requerido de los dibujos de montaje, su cantidad y el tipo (papel o tela); también se debe indicar la cantidad de juegos de dibujos de detalle y hojas de croquis, por lo general un juego para la oficina y un juego para el campo.

5.15 Emparrillados y losas

Es necesario elegir el método de montaje para la colocación de los emparrilados y/o losas. Para losas de peso razonable pueden usarse lainas, usando tres o cuatro montones de lainas de placa de acero de más o menos 4 x 4 plg (10 x 10 cm); una de 1/2 plg (1.3 cm), una de 1/4 plg (0.6 cm) y dos de 1/9 plg (0.3 cm) de espesor en cada montón, para la primera capa de relleno de 1 plg (2.5 cm) de espesor y lainas adicionales de 1/2 plg (1.3 cm) de espesor, en caso de tener un relleno mayor de 1 plg (2.5 cm).

Usando lainas con espesor mínimo de 1/8 plg podrían colocarse las losas con una tolerancia de más o menos 1/16 plg sobre la elevación requerida. Si con el montón de lainas la losa está de 1/16 plg a 1/8 plg bajo el nivel requerido, al agregar una de 1/8 plg se colocará a menos de 1/16 plg sobre este nivel; de manera similar, si está de 1/16 plg a 1/8 plg por encima del nivel, quitando una laina de 1/8 plg quedará menos de 1/16 plg abajo.

Para nivelar la parte superior de una losa y colocarla a su elevación final, es más fácil hacerlo con tres montones de lainas que con cuatro, debido a la dificultad que se tiene para dar la misma elevación a cuatro montones; esto se debe a las irregularidades comunes de la superficie de concreto sobre la cual se colocan los montones de calzas. Las lainas pueden colocarse antes de tiempo, con rellenos de arena, cemento y agua a su alrededor para fijarlas con seguridad. La losa puede colocarse sobre lainas sueltas y por lo general el contratista de la cimentación o el contratista general construyen cajas para relleno alrededor de las losas, después de que la losa queda colocada en su posición final; el relleno de cemento se cuela por debajo de las losas y dentro de las cajas.

Algunos montadores prefieren usar pequeños montículos de mortero de cemento de consistencia espesa, colocando después una laina de 1/2 plg (1.3 cm) de espesor sobre la parte superior y golpeando ligeramente se sitúa a la elevación correcta de la parte inferior de la losa; después se permite que fragüe el mortero hasta que esté lo bastante firme como para colocar después la losa sobre las lainas. Esto puede hacerse sólo si se han maquinado las superficies inferior y superior de la losa, lo cual es muy aconsejable si en el taller la losa se ha ensamblado a una columna.

El exceso o la escasez en el espesor de una losa que se monta sobre lainas previamente colocadas puede implicar trabajo y gastos extras; si el espesor es menor, se requiere levantar el ensamble de la losa o del emparrillado, agregar lainas y reemplazarlas; si el espesor es mayor, no sólo se requiere quitar la pieza, sino demoler las lainas ya colocadas y colocar de nuevo la pieza en su lugar.

En losas pesadas es mejor que el fabricante suelde dos tuercas en dos de los lados opuestos de la pieza, colocando tornillos de punta cónica en estas tuercas, para que cada uno de ellos se apoye en dos lainas gruesas (de 1/2 ó 1 plg de espesor); es conveniente tener dos lainas, ya que la más baja de ellas quedará colocada sobre la base de concreto, mientras que la superior puede deslizarse sobre ésta y los tornillos se apoyarán sobre ella.

Es conveniente usar ángulos guía para los emparrillados, colocándolos como una "V" invertida. Dos ángulos, uno para cada extremo del lecho bajo del emparrillado, de $1\frac{1}{2}$ x $1\frac{1}{2}$ x 3/16 plg (3.8 x 3.8 x 0.5 cm) y 6 plg (15.2 cm) más largos que la dimensión exterior son satisfactorios para ensambles ligeros; para ensambles pesados, los ángulos pueden ser de $2\frac{1}{2}$ x $2\frac{1}{2}$ x 3/8 plg (6.4 x 6.4 x 1.0 cm). Se necesitan lainas de más o menos 3 x 5 plg (7.6 x 12.7 cm) para soportar los extremos de los ángulos guía y colocarlos a la elevación correcta; deben suministrarse lainas adicionales de 1/2 plg de espesor, en cantidad que dependerá del espesor del relleno de mortero especificado entre el nivel superior de la base de concreto y la parte inferior del emparrillado. Se debe tomar en cuenta la altura de los ángulos, con sus patines apoyados en las lainas.

Las losas o placas delgadas de nivelación pueden embarcarse con anticipación, para que sean otros quien las alineen y coloquen. Si las bases también se han nivelado antes de tiempo, no se necesitarán lainas ni ángulos guía. Esta es una condición ideal para el montador, pero el fabricante necesita hacer la altura total del emparrillado y/o de la losa a la dimensión exacta y el montador necesita revisar la colocación de las placas de nivelación antes de comenzar su trabajo.

Si se requieren pernos de anclaje, se debe acordar con el fabricante si él o el montador confirmarán con el cliente la fecha de entrega. Es conveniente contar con rondanas, para usarlas bajo las tuercas de los pernos de anclaje, tuercas que se embarcan con la estructura del primer nivel para asegurarse de que no se pierdan si se embarcan junto con los pernos de anclaje. Si se van a usar dichos pernos de anclaje, debe informarse al departamento de dibujo la tolerancia que desea darse al diámetro de los agujeros de las losas, para absorber las discrepancias que se presenten al colocar los pernos de anclaje en el campo; por lo general es suficiente usar agujeros 5/16 plg (0.8 cm) mayores en diámetro que los pernos de anclaje de diámetro grande, pero en caso de que esto difiera de lo indicado en los dibujos del contrato o de diseño debe obtenerse la aprobación del ingeniero de proyecto. De manera similar, si se requieren "camisas" en las columnas para conectar los pernos de anclaje, se debe informar al fabricante a qué medida debe hacer el diámetro de los agujeros; por lo general, para montaje, es aconsejable usar un diámetro 9/16 plg mayor, así como para tomar en cuenta las faltas de precisión del taller y del campo. Una tuerca piloto acartelada ayudará a atornillar los tornillos a través de los agujeros de las "camisas". Debe hacerse un esfuerzo para localizar los agujeros de los pernos de anclaje tan lejos del centro de la losa como sea posible, para obtener la máxima resistencia contra el volteo de la columna.

Cuando en el taller se sueldan los ángulos o placas de conexión para conectar las columnas a las losas, debe prevenirse al fabricante para que diseñe las soldaduras de modo que puedan tomar el peso total de la losa y del emparrillado, en caso de que estén conectados a dicha columna y también soportar la carga lateral del viento en el primer tramo de columna; de otra manera, debe indicarse al montador que contraventee cada columna al montarla, dejando el contraventeo en su sitio hasta que se haya montado y ajustado un panel completo de estructura, para que sea seguro dejar las columnas sin contraventeo. La falla de la soldadura al izar una losa o un emparrillado, o cuando se deja la columna sin contraventeo, puede ocasionar un accidente grave. Debe solicitarse al fabricante que indique las líneas de los ejes en las piezas, por medio de marcas de golpe o líneas trazadas cerca de los cuatro bordes de cada losa, para ayudar en la colocación final y correcta de la pieza en el campo, de acuerdo a los ejes teóricos de columnas.

5.16 *Tornillos, remaches, soldaduras*

Si no se ha establecido previamente una regla entre el fabricante y el montador, es oportuno que el departamento de proyectos registre el porcentaje adicional de tornillos, remaches o soldadura que se debe incluir en la relación de los materiales que se requieren; esto es para tomar en cuenta las pérdidas, los remaches quemados, los cabos no utilizados de electrodos, las cuerdas dañadas en tornillos y un pequeño porcentaje que puede usarse para propósitos de montaje, ya que en la mayoría de los contratos para la fabricación de estructuras se incluye un porcentaje adicional que el cliente debe pagar. Este porcentaje puede variar de 2 a 5 % , dependiendo de los tamaños, diámetros y tipo, pero debe mantenerse por debajo del porcentaje que el cliente pagará al fabricante. Las cabezas de los tornillos y las tuercas para el mismo diámetro deben ser del mismo tamaño, para que pueda usarse la misma herramienta para ambas.

Cuando en las conexiones se emplearán tornillos o remaches, esto se debe especificar en unas pequeñas hojas llamadas listas de tornillos y remaches. En las listas se debe indicar la localización de cada conexión, selañando el número de la viga y el número de la columna que se conecten, o los números respectivos de dos vigas que se conectan en el caso de que una viga secundaria se apoye en su cabezal; también deben indicar el tamaño de los tornillos o remaches de la conexión, junto con su longitud, diámetro y tipo, según sea A.R. (alta resistencia), A-325, A-345 ó A-490 tornillos máquinas, tornillos torneados; remaches A.R., remaches de sección variable, etc.

En el caso de la soldadura, se suministran listas de soldadura en las que se indica de modo similar la localización de las conexiones, junto con los detalles de soldadura, el tamaño y tipo del electrodo, según sea E-6010, E-7010, E-6011 y el tipo de soldadura, ya sea de filete, a tope. Si las conexiones son complicadas o es importante soldar un cordón de un lado y después un cordón del lado opuesto o en algún otro miembro que llegue a la conexión, antes de completar un segundo o tercer cordón en el primer punto que se soldó, deben suministrarse al montador croquis de cada uno de tales procedimientos y conexiones.

5.17 Piezas cortas o largas

Deben tomarse en cuenta las diferencias en la longitud de las piezas, debidas a la falta de precisión en la fabricación o en las tolerancias, en los casos en que se tiene una línea larga de vigas, trabes o armaduras; de otra manera, se tendría un edificio muy largo o muy corto, muy ancho o muy angosto, según sea el caso. Si las piezas quedan cortas, pueden usarse calzas o placas de relleno si las conexiones son sin asiento; si las conexiones no son de este tipo, pueden rimarse los agujeros, ya sea que la pieza quede corta o larga, usando conectores de mayor diámetro. Si las conexiones son soldadas, pueden usarse tornillos de menor diámetro en los agujeros suministrados para propósitos de montaje, y la soldadura mantendrá los miembros en su posición ajustada. Si existe alguna posibilidad de tener miembros largos cuando se usan conexiones sin asiento, el fabricante y el montador deben llegar a un acuerdo para acortar a propósito las piezas que pueden causar dificultades y suministrar lainas o placas de relleno de un espesor igual al corte, para usarlas en caso necesario.

5.18 Rampas y puentes

Si se necesita una rampa para llevar equipo dentro de la excavación (el "agujero"), el montador debe diseñarla, a menos que el contratista general suministre alguna, lo cual puede hacerse, ya que el contratista de la cimentación y algunos otros también pueden necesitarlo; a menos que se use una rampa maciza de terracería, se debe revisar el diseño de alguna rampa suministrada por otros, para asegurarse de que pueda usarse con seguridad. Se deben hacer indicaciones en el dibujo del plan de montaje para que el superintendente pueda confirmar si se construyó del modo que fue diseñada.

Cuando el montador de la estructura construya la rampa, deben de darse al superintendente los detalles completos en dibujos separados; en este caso, el material requerido lo suministrará el almacén o se comprará para ser entregado cuando se necesite. Si se va a usar una pluma para el montaje, el material puede descargarse desde la calle hasta el "agujero" mediante una grúa montada sobre camión, o sobre orugas, eliminado así la necesidad de una rampa; si se va a usar una grúa para el montaje, en general se necesita una rampa, a menos de que la grúa pueda montar desde el terreno que circunda a la excavación.

En el caso de un edificio que tenga muchos pisos por debajo del nivel de la calle puede ser necesario usar equipo auxiliar para la descarga, en caso de que no se pueda construir una rampa sin bloquear muchas de las zapatas o cimentaciones de las columnas. Si la excavación es muy profunda, el contratista general o el de la cimentación pueden construir un puente provisional desde la calle hasta la excavación, para poder trabajar desde ahí; es importante que este puente sea lo bastante fuerte para soportar una grúa para el montaje de la estructura, o para colocar una pluma en el fondo de la excavación y entregarle elementos estructurales por medio del puente, hasta que haya montado suficiente estructura para cambiarse de nivel, al nivel de la calle. Cuando las facilidades de descarga en las calles adyacentes son restringidas por las condiciones del tráfico, un puente de este tipo puede ser el único medio para entregar materiales para la estructura, no sólo elementos estructurales, sino también todos los demás materiales.

5.19 Casas de máquinas

Al preparar el plan o esquema de montaje para una casa de máquinas y el cuarto de calderas adyacente, es económico y práctico comenzar la obra, que se montará con una grúa de tirantes, usando primero una grúa montada sobre camión o sobre orugas de capacidad suficiente; con esta grúa se colocarán todos los emparrillados y losas y la primera o segunda hilera de estructura del cuarto de calderas. A continuación se colocarán las plumas sobre la estructura montada con las grúas. Con una programación adecuada, puede darse un uso óptimo a la grúa en la parte baja de la estructura del cuarto de calderas, hasta que las plumas se desmonten y se embarquen fuera de la obra. Es mejor utilizar dos plumas ligeras para montar por separado la mayor parte de la estructura del cuarto de calderas y usar las dos en conjunto para descargar y montar las trabes pesadas; si no pueden colocarse las dos plumas juntas para descargar, la grúa se puede utilizar para descargar e izar las trabes pesadas hasta el piso de trabajo, siempre y cuando tenga la suficiente capacidad y si el mástil es bastante largo. A continuación se colocan las trabes, de manera que las dos plumas puedan izarlas y montarlas y la grúa continúa de nuevo montando su parte de la casa de máquinas. Todas estas actividades deben indicarse en los dibujos del plan de montaje, para asegurarse de que el superintendente siga el procedimiento planeado y seguro; de otra manera puede sobrecargar las plumas o la grúa, o la estructura permanente, lo cual puede ocasionar un accidente.

5.20 Edificios con núcleos de columnas

Al seleccionar el método de montaje para un edificio con núcleos de columnas, se analizan las columnas de acero para que se pueda informar con oportunidad al departamento de proyectos acerca de las longitudes en que se deben fabricar; debe tomarse en cuenta el nivel del empalme que se diseñó, el peso que puede manejarse con el equipo disponible, la longitud que se puede embarcar y entregar dentro de las calles de la ciudad y, por último, la estabilidad de la sección o columna cuando se gira desde la posición horizontal en que se entrega hasta su posición vertical. Si en el contrato no están incluidos puntales permanentes, se deben diseñar puntales provisionales de madera o de acero, o algún contraventeo entre las columnas; los puntales deben conectarse a las columnas a una distancia razonable por encima del piso terminado, de manera que el personal pueda quitarlos después con seguridad, apoyándose en los pisos de concreto que se hayan colado ya. De otro modo, se necesitará algún tipo de andamio, lo cual ocasionará gastos y riesgos innecesarios en el trabajo del montador; las especificaciones de conexiones para estos puntales o contravientos se deben entregar a la oficina de proyecto del fabricante para que se tengan listas a tiempo.

Si el fabricante suministrara los puntales, debe determinarse por anticipado el costo que esto representa para el montador; puede ser que éste pudiera ahorrar dinero comprando el material y fabricando él mismo dichos puntales.

En el caso de columnas muy altas con puntales muy ligeros entre ellas, se usan tirantes diagonales, tanto horizontales como verticales; los tirantes horizontales, como en el caso de los puntales, deben quedar por encima de los pisos terminados, en el plano de dichos puntales. Los tirantes verticales quedarán hundidos en concreto y se arruinarán, a menos de que el contratista de obra civil los coloque dentro de cajas en los puntos donde atraviesan los pisos; esto se debe prever.

Si los tramos completos de columna pueden manejarse con una grúa que se tenga disponible, ésta puede ser una solución mejor que utilizar una pluma con tirantes; pero si el edificio es alto o, como a veces sucede, las columnas del núcleo llegan sólo a una cierta altura y después la estructura continúa a base de vigas y columnas convencionales, es mejor utilizar una grúa con tirantes. Deben hacerse los arreglos necesarios para dejar una abertura en los pisos ya colados, lo bastante grande como para que la pluma pueda cambiarse de nivel.

Si se van a utilizar plumas, el esquema de montaje debe planearse de manera que éstas monten la primera hilera de estructura según lo haya indicado el montador, lo cual puede incluir cuatro pisos o más; después se fijan las plumas hasta que se hayan colado estos pisos y el concreto fragüe lo suficiente para que pueda trabajarse sobre ellos. Después, el montador reanuda sus actividades, cambia de nivel las plumas y coloca otro tramo de columnas, o bien comienza con la parte de vigas y columnas de la sección superior. Las columnas del núcleo deben contar con "orejas" de izaje, para asegurarse de que el superintendente las toma por puntos de izaje seguros, evitando así el flambeo o la falla.

Como en el caso de un edificio común y corriente, se debe seleccionar la localización de tirantes en ciertas columnas y deben hacerse los arreglos necesarios para fijar eslingas de anclaje a estas columnas, quizá utilizando amarres embebidos en los pisos de concreto que se van colando, quemando después dichos amarres por encima y por debajo de los pisos.

5.21 Construcción a base de paneles abiertos

En la construcción a base de paneles abiertos, debe estudiarse también el material de los pisos de trabajo. Se da el nombre de construcción a base de paneles abiertos a una estructura que tiene grandes claros entre columnas, las cuales se conectan entre sí mediante miembros que llegan directamente a ellas, sin vigas intermedias que se conecten a estos miembros. Después se cuela una losa de concreto en todo el panel, el cual puede ser hasta de 20 ó 25 pies (6.10 a 7.62 m) por 25 a 30 pies (7.62 a 9.14 m).

Los tablones de piso comunes, de 2 x 12 plg (5.1 x 30.8 cm), serán inseguros si se descargan sobre ellos miembros estructurales, o si el personal camina sobre ellos, pues el claro es muy grande y no existe estructura de apoyo intermedio, como en el caso de las construcciones normales a base de vigas y columnas.

Hay varias maneras de resolver esta dificultad; la mejor es utilizar vigas simples provisionales, por ejemplo, vigas de patines anchos de 8 ó 10 plg (20.3 ó 25.4 cm) de peralte nominal, como apoyos intermedios. Estas vigas pueden venderse después casi al mismo precio de compra, pues su valor de rescate es muy alto si no tienen conexiones o agujeros; estas vigas, que cubren el claro del panel, se deben seleccionar de manera que soporten las cargas de los tablones del piso y cualquier elemento o elementos estructurales que se descarguen en el piso.

Las columnas que se vayan montando se deben izar por separado desde el piso y colocarlas directamente en su sitio; si esto no es posible y deben colocarse por un tiempo sobre el piso, esto debe hacerse en diagonal y sobre patines, para llevar la carga a las dos vigas que se conectan perpendicularmente a la columna que se montó antes en el punto en que se montará ésta.

El dibujo del plan de montaje debe mostrar la distribución de la estructura que se descargue sobre los pisos, la separación de las vigas de soporte temporales y toda la información necesaria para que el superintendente de montaje siga el plan ideado por los ingenieros del montador. Cuando los tablones de piso se cambien de nivel, éstos se deben apilar fuera de sus soportes, los cuales también se apilan en un montón de vigas para cualquiera de los paneles, se izan con la grúa o la pluma al nuevo piso de trabajo y se distribuyen de nuevo, después, los tablones se cambian al nuevo nivel y se tienden sobre la estructura de soporte.

Como un sustituto para este sistema, algunos montadores utilizan como soporte cables pesados de alambre, con templadores, enganchados sobre las vigas en los extremos opuestos de la nave, colocando dos o tres líneas de cables en cada nave. Los ganchos se fijan sobre los patines superiores de las vigas de antepecho, en cada uno de los bordes del edificio, si es que los patines son lo bastante fuertes; de preferencia, los cables y templadores individuales se conectan a estrobos separados que forman una "V", fijando los extremos de los estrobos a las dos columnas situadas a los lados de la nave, en cada extremo del edificio. Los templadores se aprietan hasta que sólo haya una pequeña flecha en los cables y después los tablones se colocan sobre ellos. Es muy difícil calcular el esfuerzo en los cables, lo cual es una razón para no aconsejar el uso de este método; además, el piso se moverá demasiado y el personal se moverá con mucha lentitud.

Los patines se deben usar para llevar la mayor parte de la carga del piso directamente a la estructura definitiva en cada panel; al voltear las columnas que depositan sobre el piso antes de montarlas, se debe tener cuidado para no sobrecargar los tablones de piso o los cables que los soportan. Debe usarse un madero largo o una pieza de acero que cubra el claro completo del panel, para soportar el extremo inferior de la columna al voltearla para el montaje; es necesario indicar este madero o viga en el dibujo del plan de montaje.

Cualquiera que sea el método que se use para soportar los tablones del piso, puede ser conveniente usar tamaños mayores, de las dimensiones usuales de 2 x 12 plg (5.1 x 30.5 cm) a 3 x 12 plg (7.6 x 30.5 cm) y su longitud por encima de los 22 ó 24 pies (6.71 a 7.32 m), que también son usuales; esto los hará demasiado pesados como para que los puedan cargar sin riesgo dos hombres y será necesario emplear más personas para ello. Todos estos detalles y cualquier otra precaución necesaria, se deben indicar en el dibujo, así como en las instrucciones escritas que se envíen al campo.

5.22 *Puentes entre edificios*

A veces deben construirse puentes entre edificios, para salvar las calles que tienen demasiado tráfico. En caso de que la altura y peso de los miembros queden dentro de las posibilidades de una grúa, ésta puede ser la solución ideal; pero si las restricciones del tráfico limitan su uso la solución adecuada sería usar una pluma de tirantes.

En el esquema más simple se usa un par de vigas o maderos estructurales como soportes, en voladizo sobre el borde de la estructura al nivel del puente y con sus extremos interiores fijados a la estructura ya montada; sobre éstos se coloca una pluma de tirantes, ya sea contraventeándola a las dos estructuras que se van a unir con el puente, o usando un soporte para el pasador de émbolo de la base, similar al que se describió en el caso de las plataformas de montaje.

5.23 Cobertizos para muelles

Cuando una estructura,.tal como un cobertizo para muelle, colinda con una masa de agua, y la entrega de materiales se puede hacer por vías acuáticas, el equipo indicado para el montaje puede ser una pluma montada sobre un bote; ésta también puede usarse para entregar los materiales al área, en caso de que los aparejos de montaje puedan funcionar de manera muy eficiente. En algunas ocasiones, el fabricante no tendrá la posibilidad de entregar materiales con tanta rapidez como lo requiere un programa eficiente de montaje; en estos casos, puede usarse una pluma montada sobre un bote, para entregar el material tan pronto como se reciba, descargándolo en las posiciones aproximadas en que se necesitará. Este tipo de esquema o plan puede funcionar en forma conveniente para el montador, el fabricante y el propietario, una vez que se ha entregado material suficiente para que el aparejo de montaje pueda empezar a funcionar sin demoras debidas a la espera del resto de dichos materiales.

Al preparar el plan de montaje, debe decidirse si se va a rentar una pluma con el mástil suficientemente largo y con capacidad adecuada para descargar los elementos estructurales y colocarlos lo bastante lejos del bote, o si sería más económico y práctico rentar una barcaza para colocar en ella una pluma y un malacate, o aún más, mover una grúa a la cubierta del bote. Si se va a montar una pluma, para utilizar mejor su capacidad puede ser conveniente amarrar dos barcazas, instalando una pluma de patas rígidas encima de esta torre. Debe revisarse la posibilidad de volteo de la barcaza o barcazas, bajo la condición en que el aparejo está manejando la pieza más pesada al alcance requerido, o bien una pieza ligera al máximo alcance. El factor limitante puede ser el calado permisible.

Los dibujos del plan de montaje deben de determinar con exactitud la distancia a la que puede operar el mástil con diferentes cargas, ya que mientras más lejos se manejen, más se inclinarán la barcaza o barcazas;

pueden inclinarse tanto que el giro del aparejo puede llegar a ser crítico.

Si el muelle es muy ancho, puede ser necesario descargar la estructura y pasarla a un área interior por medio de una grúa auxiliar a algún otro equipo; la mayor parte de los cobertizos para muelles tiene un ancho de tres naves y se presta para usar tres aparejos, uno en cada nave, con aparejos adicionales, seleccionando, pasando y distribuyendo materiales por delante de los tres principales, así como ensamblando armaduras desarmadas, de manera que se puedan montar en una sola pieza. Con este arreglo, debe vigilarse el área de trabajo para mantener libres los pasillos de manera que los aparejos de montaje se puedan mover con libertad; estas áreas de circulación se deben delinear en los dibujos del plan de montaje.

Los elementos estructurales se pueden entregar por medio de carros de ferrocarril, embarcarlos en una barcaza directamente desde el taller del fabricante, o descargarlos de los carros de ferrocarril a una barcaza en un patio del ferrocarril; la pluma o la grúa descargan después la barcaza que entrega los materiales y los coloca en el lugar de montaje. Se deben mostrar en un dibujo, la secuencia y las precauciones que limiten las cargas que pueden izarse, la distancia a que puede colocarse el mástil y la localización del bote de la pluma en cada entrega, el programa de embarques debe coordinarse con cuidado con los requisitos del programa de montaje.

Es necesario tomar las medidas necesarias para facilitar el movimiento del bote hacia los diferentes puntos, ya sea usando remolcadores o mediante líneas de cable conectadas a un malacate auxiliar o tomando del malacate principal que controla el mástil y las líneas de carga.

En el caso de una pluma montada sobre una torre, el mástil puede ser lo bastante alto para librar la estructura ya terminada, y así se podrá montar toda la estructura sin necesidad de usar otros aparejos en la misma base-grúas, plumas o plumas viajeras.

5.24 Acero de soporte para muros

Cuando se utiliza mucha estructura de soporte para muros, se tiene que decidir si se apuntala con soportes de acero o de madera, o si se dejan sin montar los miembros de la estructura de soporte hasta que se hayan construido los muros; esta última solución requiere viajes extra después de que se ha montado la estructura principal. En estos viajes de regreso puede colocarse el material disponible si es lo bastante ligero, o si en el lugar se cuenta con una grúa, que pueda usarse durante periodos cortos a un costo razonable, ésta se podrá utilizar si el mástil tiene la longitud suficiente para alcanzar el área (ver también la sección 6.17).

5.25 Dibujos del plan de montaje

Después de que se ha analizado la obra y se ha seleccionado el tipo de equipo de montaje, el plan de montaje debe estar ya bien definido; entonces se dibuja el proyecto que se seguirá en el campo.

Al trabajar en el plan de montaje pueden encontrarse dificultades que impidan usar algunos ensambles que ya se solicitaron, o quizá se requieran conexiones más pesadas para soportar el equipo de montaje ya seleccionado, o cualquier otro cambio en los detalles convenidos con el fabricante. Es vital que esta información se transmita con rapidez con el fin de evitar cargos extras en caso de que el fabricante tenga que hacer nuevos dibujos o cambiar su programa de fabricación.

Es conveniente que el departamento de diseño esté enterado de la magnitud de las cargas que soportará la estructura en caso de que algún equipo de montaje vaya a funcionar encima de la estructura permanente; esto es necesario para asegurar que las conexiones sean lo bastante fuertes para soportar la grúa, pluma o pluma viajera. No siempre es posible hacer estas conexiones a base de conectores permanentes, como tornillos de alta resistencia, remaches o soldadura, antes de que el equipo se mueva o se cambie de nivel; al revisar la resistencia de las conexiones, debe considerarse una resistencia mínima suponiendo que se usen tornillos de ajuste, de preferencia tratados térmicamente, así como algunos pasadores. Esto se debe indicar en los dibujos que se preparan para el campo, para mostrar la localización de la pluma o de la grúa levadiza en los diferentes pisos, o en las hojas que muestran el procedimiento para la operación de la grúa o la pluma viajera sobre la estructura.

Las copias de un croquis básico (Fig. 5.25.1) en que sólo se muestren las localizaciones de las columnas, pueden ser muy útiles ya que se pueden usar para mostrar detalles del método de montaje que no se encuentran en los dibujos del plan de montaje, y también serán útiles para la oficina y el campo, para seguir el avance de montaje. Pueden ser útiles para informar al departamento de proyecto acerca de las áreas de entrega y de los detalles de localización de las conexiones especiales que se requieren; los ingenieros de campo pueden usarlos para llevar registros de la diferencia en la posición real de las columnas, con respecto a su posición correcta. Cuando el contratista

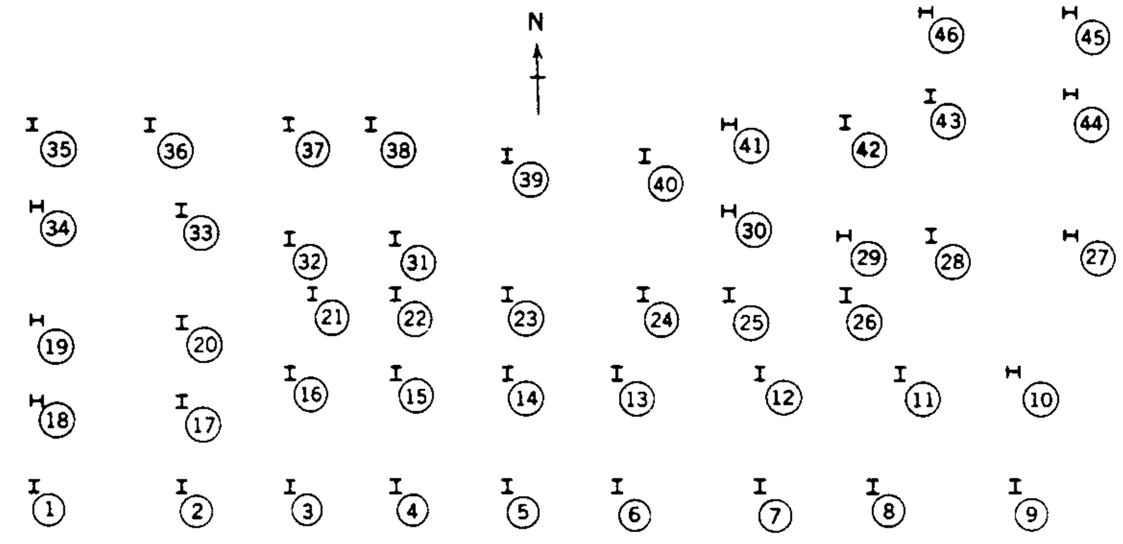


Figura 5.25.1 Croquis típico para uso general.

general o el dueño hayan asignado un área específica para cobertizos, oficinas, almacenamiento de materiales, etc., esto puede mostrarse en este tipo de croquis o en algunas instrucciones, en vez de indicarlo en el dibujo del plan de montaje.

Los planos del plan de montaje deben mostrar en detalle no sólo el plan que se decidió seguir, sino también cualquier condición no usual que pueda presentarse en el campo. Cuando el montaje es normal y no se tendrá ningún caso poco usual, los dibujos del plan de montaje se pueden reemplazar por un simple juego de instrucciones escritas en que se describa el procedimiento a seguir, dando todos los detalles necesarios para que el superintendente del campo siga el esquema planeado por los ingenieros del montador.

Se deben proveer tablas de capacidad para el equipo que se usará, mostrando el alcance máximo permisible para colocar las piezas pesadas y la carga máxima que se puede levantar al máximo alcance (con el mástil horizontal); para la preparación del plan de montaje debe hacerse un análisis del peso de las cargas críticas y, al mismo tiempo, de los pesos de la estructura que hay en cada nivel, la cantidad de piezas, los pesos por cada área, el número y tipo de los conectores en cada conexión y cualquier detalle relacionado con la obra.

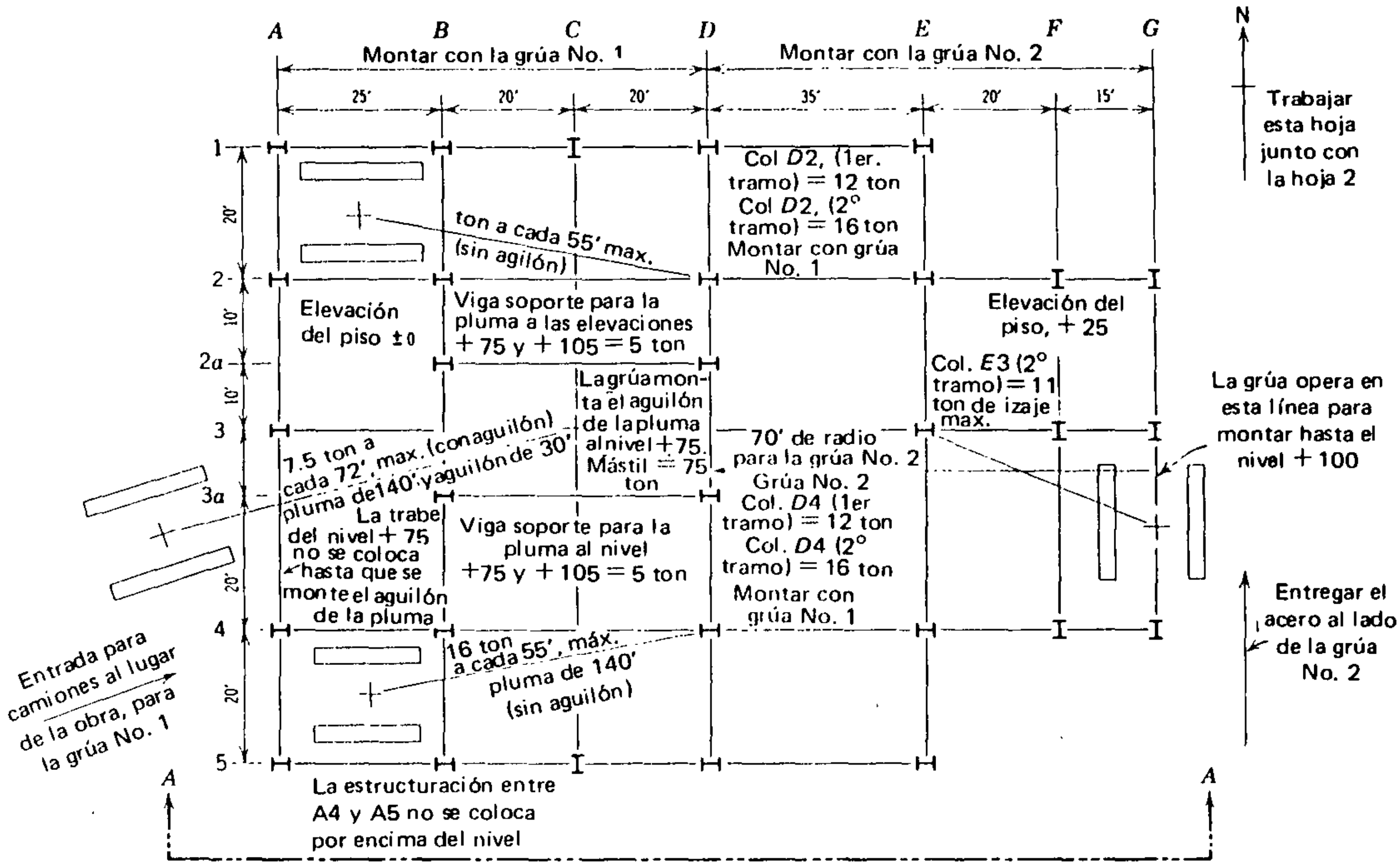
La longitud de mástil que se necesite para un aparejo de montaje se determina calculando la diferencia que hay desde el pie del mástil hasta la parte superior de la columna más lejana, con una tolerancia por encima de este punto, que permita pasar a las poleas de carga cuando están en su posición más alta, así como a la bola de contrapeso, el gancho y la eslinga o estrobo colocado entre el gancho y la parte superior de la pieza.

Si es necesario ensamblar algunas armaduras o trabes en el lugar de la obra, antes de izarlas como una sola pieza, las áreas de ensamble deben marcarse para que ningunas otras piezas se descarguen ahí y entorpezcan los trabajos. Debe darse la cantidad necesaria de soportes para ensamblar las piezas individuales, si se ensamblaran en posición plana, para permitir que el personal trabaje con seguridad instalando los tornillos o remaches permanentes, o soldando los miembros, mientras las piezas se encuentran todavía encima de los soportes. Si una armadura o una trabe se ensamblará en lo alto, debe diseñarse la obra falsa necesaria para soportar los miembros que se están ensamblando y debe revisarse su estabilidad una vez colocada en su sitio; el procedimiento se debe especificar en detalle.

Si algunos de los miembros se van a izar con dos aparejos adyacentes, o si existe un impedimento para levantar alguna pieza en particular en una cierta posición, esto debe indicarse en el dibujo del plan de montaje, o agregar un dibujo por separado para explicar esta situación; en el caso del montaje con grúas móviles, es necesario considerar y prever el movimiento y localización de las grúas al levantar cargas críticas, la secuencia de montaje y los conceptos similares, mostrándolos en detalle en los dibujos (Figs. 5.25.2 y 5.25.3).

Por lo general, las escuelas, iglesias y teatros tienen áreas para auditorios; éstas se deben revisar con cuidado para confirmar si las armaduras de techo, las trabes u otros miembros situados directamente sobre el área del auditorio se pueden presentar como una sola pieza, o si se necesitará obra falsa. Los balcones o voladizos requieren algún apuntalamiento u obra falsa, con algunas medidas para ajustarlos a su elevación correcta por medio de gatos, antes de fijarlos en definitiva. Cuando la estructura se sujetará permanentemente por medio de tirantes, contraflambeos, o varillas con templadores, estos miembros se pueden presentar primero fijando a ellos los elementos estructurales, con lo cual se elimina la obra falsa; cuando sea necesario, se diseña la obra falsa adecuada y se detalla para fabricarla en el almacén o en el campo, o tal vez en el taller del fabricante de la estructura. Esto debe indicarse en los dibujos.

En los hangares con armaduras en voladizo que se proyectan por fuera de soportes interiores, por lo general se requieren apoyos temporales debajo de cada armadura hasta que se han montado y ajustado todas. Las guías para puertas que se cuelgan de los extremos de éstas armaduras requieren de una precisión mayor que la normal para colocarlas a plomo y a nivel, por lo que las armaduras que las



A B C D E F G
Montar con la grúa No. 1
Montar con la grúa No. 2
25' 20' 20' 35' 20' 15'
N
Trabajar esta hoja junto con la hoja 2
1 2 2a 3 3a 4 5
20' 10' 10' 20' 20'
ton a cada 55' max. (sin agilón)
Col D2, (1er. tramo) = 12 ton
Col D2, (2° tramo) = 16 ton
Montar con grúa No. 1
Elevación del piso ±0
Viga soporte para la pluma a las elevaciones +75 y +105 = 5 ton
Elevación del piso, +25
Col. E3 (2° tramo) = 11 ton de izaje max.
La grúa opera en esta línea para montar hasta el nivel +100
La grúa monta el aguilón de la pluma al nivel +75. Mástil = 75 ton
7.5 ton a cada 72', max. (con aguilón) pluma de 140' y aguilón de 30'
70' de radio para la grúa No. 2
Grúa No. 2
Col. D4 (1er tramo) = 12 ton
Col. D4 (2° tramo) = 16 ton
Montar con grúa No. 1
La trabe del nivel +75 no se coloca hasta que se monte el aguilón de la pluma
Viga soporte para la pluma al nivel +75 y +105 = 5 ton
Entregar el acero al lado de la grúa No. 2
16 ton a cada 55', máx. pluma de 140' (sin aguilón)
Entrada para camiones al lugar de la obra, para la grúa No. 1
A A
La estructuración entre A4 y A5 no se coloca por encima del nivel

Planta

Notas de Montaje

1 Aparejar la grúa No. 1 con pluma de 140', y aguilón de 30'

2 Aparejar la grúa No. 2 con pluma de 110', sin aguilón

3 Colocar la grúa No. 2 en la línea *G* según se muestra. Usar madera y camas como camino, entre *F* y *G*

4 Los camiones deben entregar el acero a las grúas según se muestra

5 Ensamblar el aguilón de la pluma en el piso. Colocarlo sobre el soporte temporal de acero al nivel + 75

6 Ensamblar el mástil de la pluma en el piso, con la grúa. Colocarlo en su sitio con el aguilón de la pluma al nivel + 75

7 Elevar la pluma al soporte temporal de acero al nivel + 105 de la manera usual

8 Usar las columnas *B*1, *D*1, *E*2, *E*4, *D*5, *B*5, *A*4, *A*2 como tirantes para la pluma

9 Montar al acero hasta el nivel + 100, con las grúas. Montar el acero por encima de este nivel con pluma.

Notas Generales

Las grúas deben estar niveladas, el mástil de la pluma debe estar a plomo y las líneas de carga verticales. Revisar el terreno y usar camas para mantener las presiones de apoyo dentro de los límites de seguridad.

No debe hacerse ningún cambio en la secuencia de montaje según se muestra, sin confirmar con la oficina principal.

Figura 5.25.2 Dibujo del plan de montaje – Hoja 1. Trabájese esta hoja con la hoja 2.

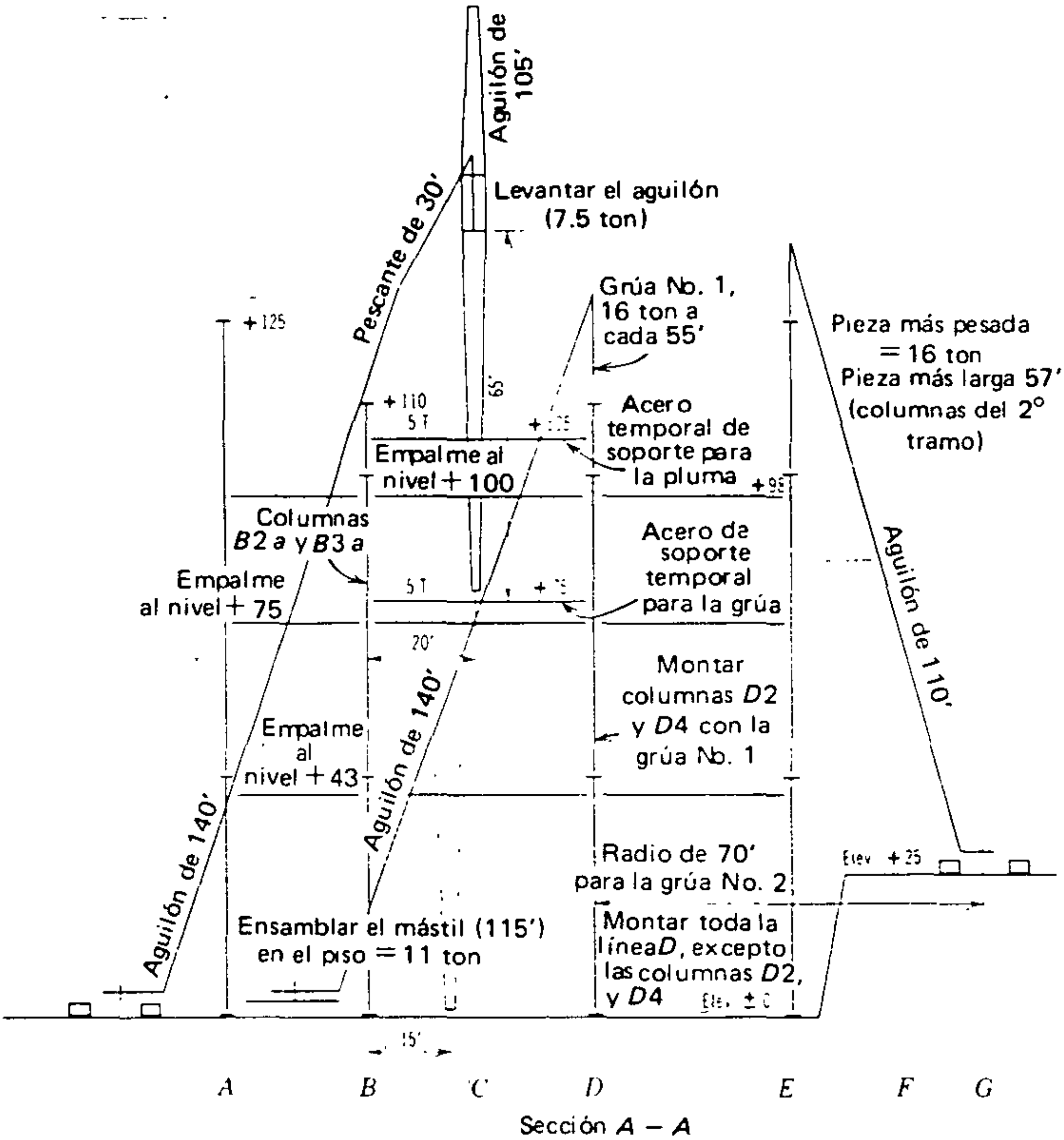


Figura 5.25.3 Dibujo del plan de montaje – Hoja 2. Trabájese esta hoja con la hoja 1.

soportan deben colocarse con precisión; debido a las tolerancias de fabricación, puede ser necesario que algunas de las armaduras se nivelen con gatos, bajando algunas de ellas, o usando lainas hasta lograr una línea recta y niveladas entre sus extremos exteriores. En las instrucciones debe incluirse un aviso para revisar las elevaciones de los extremos de este tipo de armaduras en voladizo, antes de fijarlas en su sitio. Si no puede colocarse obra falsa puede ser necesario apuntalar temporalmente la cuerda superior de cada armadura, para soportar su extremo exterior. Todo esto debe mostrarse en los dibujos.

Si se usan puntales en todo el terreno para soportar los taludes de

Capacidades en toneladas
Grúa No. 2

Radio (pies)	20	30	40	50	60	70	80	90	100	110
Mástil de 100 pies	30	15	10	7	5	4	3	2	1	
aguilón de 30 pies		8	8	6	4	3	2	1	0.5	
Mástil de 110 pies	25	15	10	7	5	4	3	2	1	
aguilón de 30 pies		8	8	6	4	3	2	1	0.5	
Mástil de 120 pies		15	10	7	5	4	3	2	1	0.5
aguilón de 30 pies			8	6	4	3	2	1	0.5	0

Redúzcase la capacidad del mástil en 2 ton.
cuando está colocado el aguilón de 30 pies

Grúa No. 1

Radio (pies)	20	30	40	50	60	70	80	90	100	110	120	130	140
Mástil de 100 pies	50	35	25	20	15	10	8	6	5				
Aguilón de 30 pies		12	12	12	11	10	8	6	5	4	3		
Mástil de 120 pies		35	25	20	15	10	8	6	5	4	3		
Aguilón de 30 pies			12	12	11	9	7	6	5	4	3	2	1
Mástil de 140 pies		35	25	20	15	10	8	6	5	4	3	2	1
Aguilón de 30 pies			12	11	10	8	7	5	4	3	2	1	1

Redúzcase la capacidad de mástil en 2 ton.
cuando está colocado el aguilón de 30 pies.

la excavación, se debe decidir si estos puntales interfieren con la operación de las grúas o la colocación de las plumas en los puntos adecuados; si existe interferencia, las grúas no podrán trabajar bien. Si se selecciona una pluma para el montaje, puede ser necesario diseñar una torre de obra falsa en donde pueda colocarse la primera vez; si es posible usar esta torre, debe construirse con algunos de los miembros permanentes de uno de los niveles superiores; con esto se elimina el costo de embarcar una obra falsa temporal y manejarla en el almacén. Al diseñar la torre debe tomarse en cuenta que el encargado de manejar la pluma pueda operar con seguridad encima de la torre de obra falsa, a menos que se use un mecanismo para mover la pluma desde otro sitio. Será necesario confirmar si por causa de interferencias algunos miembros de la estructura definitiva no se montarán temporalmente hasta eliminar el contraventeo. Toda esta información se debe incluir en las instrucciones para el campo y presentarse con claridad en los dibujos del plan de montaje.

Si el fabricante no tiene suficiente espacio para almacenar la es-

Ejemplos de levantamiento de un larguero

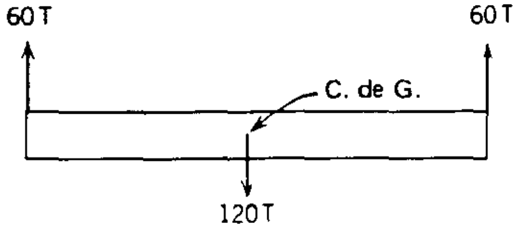


Figura 5.25.4 Dos grúas de igual capacidad; la trabe es lateralmente estable cuando se iza de sus extremos.

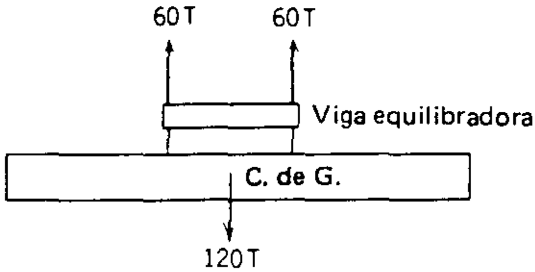


Figura 5.25.5 Dos grúas de igual capacidad; la trabe es lateralmente estable cuando se iza por su centro de gravedad, usando una viga equilibradora.

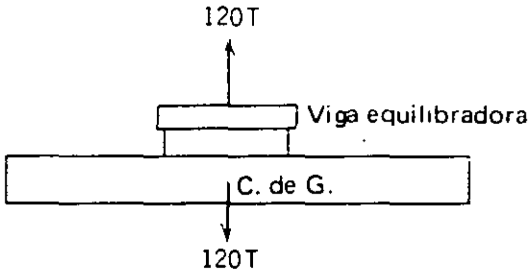


Figura 5.25.6 Una sola grúa izando una trabe con una viga equilibradora, levantándola por dos puntos equidistantes del centro de gravedad de la trabe; la trabe es lateralmente estable cuando se iza por esos puntos.

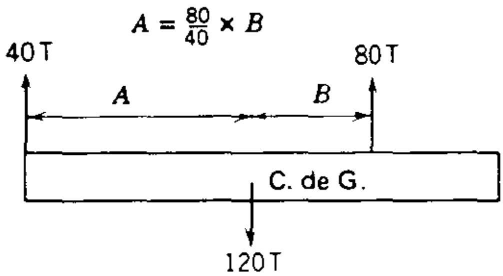


Figura 5.25.7 Dos grúas de capacidades desiguales, dividiéndose la carga en proporción directa a sus capacidades; la trabe es lateralmente estable cuando se iza como se muestra.

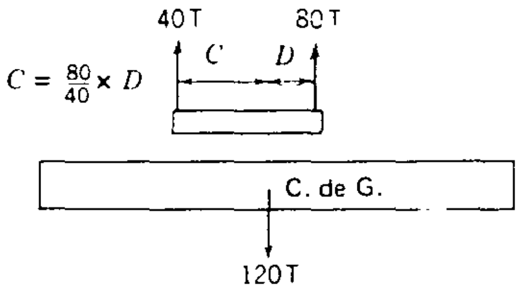


Figura 5.25.8 Dos grúas de capacidades desiguales, dividiéndose la carga en proporción directa a sus capacidades, por medio de una viga equilibradora; la viga es lateralmente estable cuando se iza por su centro de gravedad.

tructura antes de embarcarla en la secuencia y el momento adecuados para un programa de montaje eficiente, podrá pedirle al montador que la descargue antes de tiempo; en el dibujo debe mostrarse el área, el método de almacenamiento, los separadores que se deben colocar entre las piezas, la altura de las pilas, etc., (en general, el fabricante pagará el costo de este servicio extra).

Se debe indicar con claridad cualquier obstrucción o cualquier otro riesgo que haya que vigilar, mostrando en el dibujo las precauciones que deben tomarse, incluyéndolas en las instrucciones escritas para el superintendente. Deben mostrarse con claridad la secuencia y dirección del montaje, así como las áreas de entrega.

Cuando una pieza es demasiado pesada para un solo aparejo, pueden ser necesarios usar dos; cuando ambos son de igual capacidad, pueden tomar el miembro de puntos equidistantes de su centro de gravedad, si es que puede izarse con seguridad por estos puntos (Fig. 5.24.4) y si su estabilidad lateral es satisfactoria. Si la pieza puede izarse por el centro, puede usarse una viga equilibradora (Fig. 5.25.5), enganchando cada uno de los aparejos en uno de los extremos de la viga y levantando después la pieza por medio de una "oreja" colocada en el centro de la viga; esto es más seguro que izar la pieza por puntos separados, ya que permanecerá nivelada aun si uno de los aparejos levanta su extremo de la viga antes que el otro. Si sólo se usa un aparejo, la viga equilibradora permitirá usar varios puntos de izaje si la pieza no es estable al izarla por un solo punto (Fig. 5.25.6). Si los dos aparejos no son de igual capacidad, deben calcularse los puntos que se usarán para el izaje, mostrándolos después en un dibujo (Fig. 5.25.7). Estos puntos se determinarán dividiendo el peso total entre los pesos que levantará cada aparejo, en proporción directa a sus capacidades. Si el miembro puede izarse por su centro de gravedad, pero los dos aparejos son de diferente capacidad, puede usarse una viga equilibradora; el gancho de izaje estará entonces en un punto de la viga tal que la carga se divide entre las dos, de acuerdo a sus capacidades relativas (Fig. 5.25.8).

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

REQUERIMIENTOS DE INSPECCIÓN PARA ESTRUCTURAS

FABRICACIÓN

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

QUALIFICATION REQUIREMENTS
FOR STRUCTURAL STEEL SHOP INSPECTORS

1. SCOPE

1.1 This program has been prepared to establish criteria for the training and qualification of structural steel shop inspectors and of specialists in one or more specific areas of inspection.

1.2 These guidelines have been developed by the American Institute of Steel Construction to aid employers in recognizing the essential factors to be considered in qualifying employees engaged in structural steel shop inspection.

2. DEFINITIONS

2.1 Qualification - Demonstrated skill, training, knowledge and experience required for personnel to properly perform duties of a structural steel shop inspector .

2.2 Inspection - Examination of material and the fabricating process to verify that the completed product complies with the contract requirements.

2.3 Recommended Practice - A set of guidelines to assist the employer in developing uniform procedures for the training and qualification of structural steel shop inspectors to satisfy specific requirements.

2.4 Training - The program developed to impart the knowledge and skills necessary for qualification.

2.5 Structural Steel Shop Inspector - An individual who has been trained and qualified in accordance with the requirements of this program.

2.6 Structural Steel Shop Inspector Specialist - An individual who has been trained and qualified in accordance with the requirements of one or more specific areas of this program.

3. LEVELS OF QUALIFICATION

3.1 Structural Steel Shop Inspector - Proficient in all areas.

3.2 Structural Steel Shop Inspector Specialist - Proficient in one or more areas.

4. WRITTEN PLAN

4.1 The employer should establish a written plan for the control and administration of personnel training.

4.2 The written plan should include a list of subjects in which training is required and a detailed outline of the training to be given in each subject.

5. EDUCATION AND EXPERIENCE

To be considered for qualification as a Structural Steel Inspector, app. cants must meet the following minimum requirements:

5.1 Shall have a minimum of 8th grade education or equivalent.

5.2 Shall have a minimum of two (2) years in the fabricating shop engaged in actual fabrication operations (layout, cutting, fitting, welding, bolting, surface preparation and painting or have equivalent educational experience acceptable to the employer, which includes at least six (6) months in shop fabrication operations prior to examination as a structural steel inspector.

5.3 The applicant must demonstrate integrity, ability, and sound judgment in order to be considered for certification.

6. PHYSICAL REQUIREMENTS

6.1 Applicants must be in adequate physical condition, including visual acuity, to perform the duties of the job.

7. TRAINING

7.1 Personnel being considered for qualification should complete sufficient organized training to become thoroughly familiar with the codes, standards, specifications and practices to be used and with the products to be inspected. The training program should inclu sufficient examinations to assure that the necessary informatio has been comprehended.

2 7.2 Candidates for qualification as Structural Steel Shop Inspector should receive not less than five hours of organized training in each of the following eight areas of shop inspection, for a total of forty hours minimum:

I. Specifications, Codes and Standards
II. Material Preparation
III. Fitting and Fastening
IV. Welding
V. Shop Assembly
VI. Non-destructive Examination
VII. Surface Preparation
VIII. Shop Painting

Candidates for qualification as Structural Steel Shop Inspector Specialist should receive not less than five hours of organized training in each of the areas of shop inspection in which he or she is to be qualified.

Recommended training guides for each of the eight areas of shop inspection listed in paragraph 7.2 are provided on the following pages. Supplementary useful information on the subjects of Tolerances and Loading for Shipment is also provided as Appendices to the trainir guides.

STRUCTURAL STEEL SHOP INSPECTOR TRAINING GUIDE

SECTION I. SPECIFICATIONS, CODES AND STANDARDS

A. Introduction

Every step of structural steel design, fabrication and erection is controlled, to some degree, by specifications, codes and standards. To function intelligently and effectively, a structural steel shop inspector must have a working knowledge of these documents and their application in the fabrication process.

This section of the training program does not deal with details of specific provisions of codes and specifications. Rather, it identifies and classifies the most important documents commonly applicable to shop fabrication and inspection.

B. Objectives

1. For the inspector to understand how to use the various codes and where they apply in the fabrication process.

2. For the inspector to develop a working knowledge of the more frequently used codes and specifications in the fabricating industry.

3. For the inspector to develop good judgment and a sense of reasonableness within the parameters established by company management and accepted industry practices.

C. Specification and Code Publications

1. Buildings

In building construction, the following two specifications will normally be used by the shop inspector:

a. AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (commonly identified as the "AISC Specification")[1]

The shop inspector is normally concerned only with the following sections of the AISC Specification:

Section 1.4 - Material
Section 1.21 - Column Bases
Section 1.23 - Fabrication
Section 1.24 - Shop Painting
Section 1.26 - Quality Control

The inspector must also be familiar with the Commentary covering these sections of the Specification.

b. AWS Structural Welding Code - Steel D1.1[2]

AWS Code D1.1 consists of 11 Sections, Appendices and a Commentary. A structural inspector must be familiar with the workmanship requirements as defined in Sections 3, 8 and 10, covering general requirements, buildings and tubular structures. The inspector should also be familiar with the following:

Section 1 - General Provisions
Section 2, Part C - Details of Welded Joints
Section 5, Parts C & E - Welder & Tacker Qualifications
Appendix G - Flatness of Girder Webs - Buildings

The parts of the Commentary which relate to the above sections should also be familiar to the structural steel inspector.

c. RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts[3]

This specification, approved by the Research Council on Structural Connections (formerly called the Research Council on Riveted and Bolted Structural Joints) contains criteria for the acceptability, installation and inspection of high-strength bolted connections. The shop inspector should be familiar with both the provisions of the specification and its commentary on those provisions.

2. Bridges

a. Highway Bridges

The fabrication of highway bridges is normally governed by the Standard Specification of the Department of Transportation of the state which has jurisdiction over the bridge being constructed. The requirements for fabrication vary from state to state but, in general, follow the standards set forth in Standard Specifications for Highway Bridges[4] as adopted by the American Assocation of State Highway and Transportation Officials (AASHTO).

These requirements are stipulated in Division 2 of the AASHTO specifications, under Section 10, Steel Structures Fabrication. The various articles in this section cover materials, holes, punching, reaming, edges, bearing surfaces, webs, stiffeners, assembly, welding, painting and inspection.

The welding of highway bridges is governed by the AASHTO Standard Specifications for Welding Of Structural Steel Highway Bridges[5] and subsequent AASHTO Interim Specifications Bridges.

b. Railroad Bridges

The fabrication of railroad bridges is normally governed by the requirements of the Manual for Railway Engineering,[6] Chapter 15, Steel Structures, "Specification For Steel Railway Bridges of the American Railway Engineering Association (AREA). Part 3, Fabrication, covers the requirements for materials, holes, punching, reaming, edges, bearing surfaces, webs, stiffeners, assembly, welding, painting and inspection. This specification states that the requirements for welding in the fabrication of railroad bridges is to be done in accordance with AWS Code D1.1.

c. Fracture Critical Bridges

In a plant which fabricates bridges containing fracture critical elements, the inspector should also be knowledgeable in the requirements contained in AASHTO publication Guide Specifications for Fracture Critical Nonredundant Steel Bridge Members[7] and revisions contained in Interim Specifications Bridges.

3. Other References

The inspector must also be familiar with the following publications which clarify and explain certain specification requirements or serve as an expert guide for special subjects:

a. AISC Code of Standard Practice[8]

This code defines the commonly accepted standard practices of the structural steel fabricating industry. The shop inspector should be familiar with Section 5 — Materials, Section 6 — Fabrication and Delivery, and the commentary discussion of these sections.

b. AISC Quality Criteria and Inspection Standards[9]

This publication discusses some common problems and conflicts of interpretation of standards involving fabricating tolerances and procedures and provides AISC recommendations for their clarification and resolution. It should be used with existing specifications and codes.

c. AISC A Guide to the Shop Painting of Structural Steel[10]

This publication provides a checklist of the factors which fabricators should consider in the painting of structural steel.

d. SSPC Steel Structures Painting Manuals[11]
Vol. 1, Good Painting Practice
Vol. 2, Systems and Specifications
These manuals, published by the Steel Structures Painting Council, are intended to serve as a practical guide to the painting methods, equipment and systems commonly used for

structural steel. They also provide detailed specifications for surface preparation, paint application and coatings.

e. ASNT Recommended Practice No. SNT-TC-1A[12]

This publication of the American Society for Nondestructive Testing was written to provide a guideline for establishing a written practice for the qualification and certification of NDT personnel. It is not intended to be used as a strict specification.

f. Annual ASTM Standards[13]

These books contains all the formally approved American Society for Testing and Materials (ASTM) standard specifications, test methods, classifications, definitions, and practices related to the material commonly used in the fabrication industry. Section 1, Vol. 01.04, contains the specifications for the steel normally used in buildings and bridges.

g. AISC Manual of Steel Construction[14]

The "AISC Manual", as this publication is commonly known, contains a wealth of information and data required for the design and fabrication of structural steel. The inspector should have a general knowledge of the contents of the Manual. Part 1 contains standard dimensions and properties of shapes and a section on standard mill tolerances. Part 4 contains useful data on fasteners, welds and welded connections, and fabricating practice. Part 5 contains the AISC Specification[1], the AISC Code of Standard Practice[7], and the RSCS (RCRBSJ) Specification.[3]

D. Specification Reference Chart

The following chart has been prepared to provide the inspector with a ready reference as to which sections of the above listed specifications may apply to the various facets of fabrication. It is not intended to provide comprehensive coverage of all applicable specification provisons; rather it is meant to be a convenient tool that will be useful in many cases.

	SPECIFICATION				
SUBJECT	AISC	AWS	AASHTO	AREA	AISC QC (2nd Edition)
I. WORKMANSHIP (GENERAL)		Chap.3	2.10.2	3.1.1	
II. PREPARATION OF MATERIALS		3.2			Chap.1
a) Materials Approved for Use		8.2.9.2.10.2			
Mill Material	1.4.1.1 1.4.3.1.4.4		2.10.3		
Fasteners			2.10.3(A)(7)(8) (9) & 2.10.20		
Weld Filler Metal & Flux	1.4.5	4.1	1.17.21*		
Studs	1.4.6	Chap.7 Appendix K	2.10.3(a)(11)		
Mill Material Tolerances	Refer to ASTM A6				Chap. 1, 11
b) Storage of Materials			2.10.4	3.1.4	
c) Identification	1.26.5		2.10.42(B)		Chap.1, III, C
d) Preassembly		3.3, 10.14			
Burning & Flamecutting	1.23.2			3.1.6	Chap. I, II, A
Edges	1.23.3	3.2.2,3.2.3	2.10.22	3.1.6(d)&(e) 3.1.8	Chap.1, II, C
Surface Condition		3.2.1			Chap.1, II, B Chap.7, IV
Straightening	1.23.1	3.7.3	2.10.5	3.1.5	Chap.1, III, B
e) Holes	1.23.4		2.10.7	3.2.5,3.2.7	
Holes, Punched	1.23.4.2		2.10.8	3.2.6,3.2.7	
Holes, drilled			2.10.9	3.2.6,3.2.7	
Edge Distance	1.16.4,1.16.6		1.7.22(E)	1.9.4	
f) Finishing & Milling	1.21.3		2.10.6	3.1.17 3.1.7.2(a)	
g) Curving & Bending	1.23.1		2.10.5(B)		Chap.1, III, B
h) Camber	1.23.1	3.2.6,3.2.7	2.10.5(B),(6)		Chap.I, II, A
III. FITTING		3.3.7,8.14, 9.24	2.10.28		Chap.2, II
a) Accuracy of Holes			2.10.12	3.2.12	
b) Contact Surfaces,Condition	1.23.5	3.3.1,3.6.3, 10.14.1	2.10.13	3.2.13	
c) Bearing Surfaces	1.21.2,1.23.7		2.10.24	3.1.16,3.1.17	Chap.7, VI Chap.7, IX
d) Abutting Joints			2.10.25	3.1.7.2(a), 3.1.13	
e) Connection Angles			2.10.26	3.1.12	Chap.2, II, C
f) Web Plates		8.13.2	2.10.29	3.1.11	
g) Stiffeners		3.5.1.9,3.5.10, 3.5.1.11,9.21.3 3.5.1.12	2.10.31	3.1.10	
h) Pins & Pinholes			2.10.35,2.10.36, 2.10.37	3.1.14,3.1.15	
i) Diaphragms				3.1.7.2(c)	
IV. WELDING	1.23.6	--	1.17.21,Refer to AASHTO Weld Spec 1981	Refer to AWS D1.1	Chap.4
V. BOLTING	1.23.5		2.10.20	3.2	Chap.2, III
VI. ASSEMBLY					Chap.2, II, B
a) Numerically Controlled Drilling			2.10.10(B) 2.10.10(F)	3.2.7(e) 3.2.7(f)	
b) Shop Assembly Methods			2.10.14	3.2.7,3.2.10	
c) Use of Templets				3.2.8,3.2.9	
VII. PAINTING	1.24		2.10.46,2.14	3.4	Chap.5
VIII. SHIPPING			2.10.45	3.5	
IX. INSPECTION	1.25;Also Code of Std.Prac- tice	Chap.5	2.10.40,2.10.41	3.5	Chap.7, VII

E. Recommended Training

1. Study of Codes

Every inspector should be required to become familiar with and understand, through individual study, those sections of the above listed codes (and/or others) that management determines to be essential to satisfactory inspection performance. A logical sequence of the information to be studied should be designated by management.

2. Classroom Training

To supplement individual study and to develop in the inspector a sense of relative values, and to pass on management's guidelines relative to the various specifications and codes, classroom training is essential. The trainer must be prepared to discuss the specifications and codes, determine responsibility levels, and discuss the need for judgement and reasonableness.

3. On the Job Training

Management must establish on the job training periods based upon the entry skills of the inspector. These training periods should include checkpoints and performance appraisal techniques to insure that the skills and habits develop properly.

4. Audits and Updates

Management has the responsibility to audit the inspector's performance. This can be accomplished through review of field reports, inspecting the inspector's work and/or performance reviews.

Management must further keep inspectors current, through updated training sessions, distribution of new literature, and individual counselling.

F. Instructional Aids

1. Library of specifications, codes and manuals (see Appendix C, References)
2. This Training Guide
3. Examples of workmanship
4. Gauges and tools
5. Results of poor techniques or inspection

STRUCTURAL STEEL SHOP INSPECTOR TRAINING GUIDE

SECTION II. MATERIAL PREPARATION

A. Introduction

Material preparation consists of those fabrication operations normally undertaken prior to fit-up or assembly. It includes cutting by sawing, shearing, or thermal means; edge preparation; holemaking; rolling or bending; cambering, and straightening.

ASTM Specification A6[13] stipulates permissible tolerances for structural steel shapes produced by rolling mills. Inspection to ASTM A6 requirements is also covered in this section.

B. Objectives

The objectives of inspection of material preparation are:

1. To assure that material of proper shape and grade has been prepared.

2. To assure proper dimensional fit-up at later stages of fabrication.

3. To assure that edge preparation requirements for structural welding have been satisfied.

4. To assure that the provisions of ASTM A6 have been met by the mill that produced the material.

C. Inspection Checks to be Made

Checks to be made include the following, as applicable:

1. Shape and Grade

   The inspector should verify that the proper shape has been used for the piece, and that material of the proper grade has been utilized. Grade can be verified by checking the color code on the material, or by a check of identification using the fabricator's internal system.

2. Dimensions of the Piece

   Using a steel tape and a framing square, the inspector should check the length of the piece; if the member is a plate, he should also check the thickness and width. (Where reference is made to use of a framing square, it should be understood that very large pieces may require use of other devices to provide the necessary precision.) For members with both ends finished for contact bearing, a variation of 1/32" is permissible in the overall length. Members without ends finished for contact bearing, but which frame between other steel parts of the structure, may have a variation from the detailed length of:

a. Not more than 1/16" for members 30 feet or less in length

b. Not more than 1/8" for members over 30 feet in length.

In the case of plates, no such general rules exist. The inspector must often investigate the end use of the material if a question exists as to whether it is out-of-tolerance. For instance, a simple anchor plate embedded in concrete could have more relaxed tolerances than a column. On large plates, the inspector may need to tape diagonals of the plate to verify that the plate is adequately square.

3. Location of Connection Holes.

Using a steel tape and a framing square, the inspector should check the location of connection holes.

No generally recognized code covers the location of holes. A sometimes-used guideline is that connection holes should be located within a tolerance of:

a. 1/8"± for the location of hole patterns in beams or columns. This applies in both directions. However, on members having open holes at each end, such as bracing, the total tolerance for the end hole-to-end hole dimension should be limited to 1/8"±.

b. 1/16"± for the location of holes within a connection ho[illegible] [illegible]. This applies in both directions. Inspectors sometim[illegible] also check the diagonal [illegible]ance between the extreme holes in a pattern. A tolerance of 1/8" is allowed if holes are through thin material; a tolerance of 1/16" is used [illegible] [illegible]es are through thicker material.

c. 1/2"± (or more) for holes to be matched by other trades after the steel is in place.

When problems arise, an inspector may determine -- through experience in the shop bolting of misaligned connection patterns or throug[illegible] investigation with the drafting room of the actual condition[illegible] the specific piece in question -- that more li[illegible] [illegible]ances permitted. On the other hand, the inspecto[illegible] [illegible]ould also be [illegible] [illegible] where the use of the piece will [illegible]e more res[illegible] [illegible]erances [illegible] usual. Prior to starting [illegible]p inspecti[illegible] inspectors review erectio[illegible] [illegible] and details to determine [illegible] [illegible] condition.

4. Rolling and Bending

The inspect[illegible] should use a steel tape and template to che[illegible] [illegible]mensions [illegible] rolled or bent items.

[illegible] [illegible]ed items, the inside radius sho[illegible] be checked against th[illegible] [illegible]tailed dimension. This [illegible] most conven[illegible]ntly done by inserting template of the proper radius into the rolled item. On items rolled through an angle of 180° or more, the diameter can be measured and divided by two.

The length of the rolled segment can be measured directly by taping along the outside surface of the item, if a length along the outside surface is available. It can be checked indirectly by taping the tangent-to-tangent chord, if this has been shown on the shop drawing or has been calculated by the inspector from information given on the shop drawing.

On bent plates, the proper angle must be checked by use of a sliding bevel square or, on large plates, by use of a calculated diagonal dimension. The location of the bend line with respect to the edges of the plate and with respect to any connection holes in the plate must be measured and verified. If specified on the shop drawing, the bend radius must be checked. See AISC Manual of Steel Construction,[14] Part 4, for minimum radii for bends and recommendations on edge treatment at bends.

5. Cut Edges.

All cut edges, whether saw cut, shear cut, or thermal cut, should be visually examined for discontinuities (cracks, laminations, voids, inclusions, etc.)

The acceptability of discontinuities depends on the condition at the edge in question:

a. Edges to be welded.

Section 3.2 of AWS D1.1[2] provides criteria for discontinuities in edges which will become part of a welded joint. Depending on the extent and the depth, a discontinuity may:

- not require exploration, removal, or rewelding
- require exploration, but not removal or rewelding
- require exploration and removal, but not rewelding
- require exploration, removal, and rewelding

Exploration can sometimes be done with probes, or may require NDE (non-destructive examination). Alternatively, the area may be explored by grinding.

b. Edges not welded, to be dynamically loaded

Quality Criteria and Inspection Standards,[9] Section II-C, recommends that the criteria of AWS D1.1 for welded edges be used, but with specific relaxations.

c. Edges not welded, to be statically loaded

In general, more relaxed criteria are acceptable here, but no specific guidelines have been published. Questions as to acceptability may need to be referred to the design engineer.

If an edge containing discontinuities will be subjected to thru-thickness tension, the discontinuities must be carefu[illegible] explored, and repair procedures should be approved by the engineer.

Cut edges should also be visually inspected for notches and gouges. A notch is defined as a V-shaped indentation; a gouge is a cavity having a curved shape.

Notches are not permitted under any circumstances, but occasional gouges not more than 3/16" deep are permitted. All notches, and gouges deeper than 3/16", must be repaired by grinding and fairing in, or by welding, as described in Section III-A of Quality Criteria and Inspection Standards.[9]

Thermal cut edges must also be visually inspected for "roughness". The inspector customarily does this by comparing the cut edge to a surface roughness comparator such as AWS C4.1 Surface Roughness Guide for Oxygen Cutting,[15] and selecting the surface texture on the comparator which most nearly matches the cut edge. The ANSI roughness value, in microinches, can then be read on the comparator.

Section III-A of QCIS[9] provides rules for roughness of non-welded edges. A range from 1000 microinches to 1/16" is permissible, depending on the stress condition.

For welded edges, Section 3.2 of AWS D1.1[2] defines the maxin permissible roughness, which ranges from 1000 microinches to 2000 microinches.

In addition, cut edges or surfaces to be welded must be visually inspected and found to be smooth, uniform, and free of fins or tears which would adversely affect the weld.

The inspector should also visually inspect any reentrant cuts to verify that they provide a smooth transition. A radius of about 1/2" is preferred, but an even radius is not required. In addition, the inspector should visually inspect for evidence that the torch cut may have accidentally been extended past the tangent point and notched into the filleted corner. Such notches must be ground out or must be repaired by welding and then ground smooth.

6. Limitations on Punched Holes

The AISC Specification stipulates that holes through material whose thickness is greater than the normal diameter of the bolt plus 1/8" shall not be punched, but can be drilled or sub-punched and reamed. Thermal-cut holes should be made at least 1/4" smaller than the final hole size, and then reamed. Plasma cut holes may be made to final size, provided a satisfactory hole size is produced. Th- inspector should verify by visual inspection that holes have be properly made.

7. Cracks

Bent plates, and items rolled to tight radii, should be visually inspected to verify that tensile cracks have not developed on the outer surface during the bending or rolling process. Close attention should be paid to cut edges at the outer surface of bent plates, as these contain stress-raisers. (It is good practice to grind off stress-raisers before bending.)

The extent of cracks should be investigated using NDE methods, and cracks should be repaired by grinding and/or welding.

8. Cambering and Straightening

When cambering or straightening of members is in progress, the inspector should make periodic checks to verify that the member is substantially free of stress or external forces, and the temperature of heated areas does not exceed 1200°F (1100°F for quenched and tempered steels). See QCIS,[9] Section III-B.

9. The tolerances in ASTM A6 should be checked. These include length, sweep, camber, tilt of flanges, and overrun or underrun in depth. The mill material must be checked to see if it meets these criteria.

D. Special Problems

1. Material fit-up at contact joints is often a cause of controversy for inspectors. The following guidelines apply:

   a. Surfaces noted as "finished" on the drawings are defined as having a maximum ANSI roughness value of 500. Any fabricating technique, such as friction sawing, cold sawing, milling, etc. that produces such a finish may be used (AISC Code of Standard Practice[8] Section 6.2.2).

   b. Regardless of the type of column splice used (riveted, bolted, partial-penetration welded) lack of contact bearing not exceeding a gap of 1/16" is acceptable. If the gap exceeds 1/16", but is less than 1/4", and if an engineering investigation shows that sufficient contact area does not exist, it is permissible to pack the gap out with non-tapered steel shims, rather than to refinish the surfaces (AISC Specification Section 1.25.4)

   Where a question exists as to the gap which will exist, the two column sections to be spliced can be set end-to-end in the shop, and the gap measured.

2. Bridge specifications commonly impose more restrictive requirements on many operations. AASHTO and AREA and state and municipal codes should be consulted for specific limitations on:

   - Holes and holemaking
   - Plate cut edges and repair

- Thermal cutting
- Facing of bearing surfaces
- Bending of plates
- Eyebars
- Pins and rollers
- Specifications[4,5,6]

Chapter 9 of AWS D1.1[2] also contains edge requirements for plates used in bridges.

E. Recommended Training

Recommended training for material preparation inspection includes:

1. Classroom training and/or guided self-study sufficient to enable the inspector to be knowledgeable about the topics related to material preparation covered in each of the applicable codes, specifications and manuals listed in Appendix C, References, as well as applicable local codes. He or she should become familiar with all important applicable provisions.

2. Training in reading shop drawings and, if feasible, erection drawings.

3. Training under an experienced inspector in the use of tapes and framing squares or other fitting aids is recommended. The inspector should be taught how to use a 3-4-5 triangle to establish a right angle. Training should also develop the inspector's judgement to the acceptability of dimensional variations and the acceptability of visually inspected surfaces.

4. Instruction on when to ask for help in making trigonometric calculations of dimensions needed for inspection, and which persons to contact for this help.

5. Indoctrination in the capabilities and the limitations of various types of NDE. Training should be directed toward enabling the inspector to call on an NDE specialist appropriate for the situation to be investigated.

F. Instructional Aids

Instructional aids include:

1. Library of specifications, codes and manuals (see Appendix C, References)
2. This Training Guide
3. Articles or pamphlets comparing NDE methods, such as Lincoln Electric pamphlet G410, Assuring Weld Quality, by J. E. Hinkel.[16]

SECTION III. FITTING AND FASTENING

A. Introduction

Fitting is the operation in which the parts of a component are placed together in proper position and temporarily held together by clamps, tack welding, or temporary bolts. Fastening is the operation of permanently joining the fitted parts by welding or bolting.

Accurate fitting is essential to minimize material wastage and to provide a finished product of proper dimensions. Proper fastening is essential to the safety of the finished product.

No specific code or standard provides rules for fitting, but tolerances for the fitted-up assembly are provided by the AWS Structural Welding Code — Steel D1.1,[2] the AISC Specification[1] and Quality Criteria and Inspection Standards.[9]

Fastening by high-strength bolts is controlled by the RCSC specifications.[3] Welding is controlled by AWS D1.1[2] and is treated in Section IV of this Training Guide.

B. Objectives

The objectives of the fitting operation are (1) to provide an assemblage with parts correctly located and (2) to provide an assemblage which is ready for fastening. The objective of the fastening operation is an assembly which provides the required strength and safety.

C. Inspection Checks to be Made

1. Accuracy of Location of Fitted Parts

   a. Using a tape and square, the inspector checks location and orientation of parts shown on the shop drawing.

   b. Tolerance on locations of parts can be found in AWS D1.1[2] and AISC QCIS.[9] Sometimes more restrictive tolerances will be specified on the shop drawings or other contract documents.

2. Fit and Condition of Edges to be Welded

   a. Tolerances on fit of edges to be welded are given in AWS D1.1. In particular, edges against with which fillet welds will be placed must be in contact or not more than 1/16" apart, otherwise weld size must be increased. The inspector should note the location where this occurs to ensure proper weld size is provided. The degree of surface smoothness required for welding is addressed in AWS D1.1 and is further discussed in Section II of this Training Guide.

3. Fastening with High-strength Bolts

a. Surfaces to be high-strength-bolted must be free of loose scale, dirt, objectionable burrs, and other defects that would prevent solid seating of the parts. High-strength bolted parts should fit solidly together when assembled. Contact surfaces must be free of oil, paint, lacquer, or other coatings except those listed in Section 1.23.5 of the AISC Specification.

b. Surfaces of parts in contact with the bolt head and nut must not have a slope of more than 1:20 with respect to a plane normal to the bolt axis. Where the slope is more than 1:20, a beveled washer must be used to compensate for the lack of parallelism.

c. All high-strength bolts which require shop torquing must be tightened to a bolt tension as specified in the RCSC specification.[3]

d. A490 bolts tightened by the turn-of-the-nut method require a hardened washer under the turned element. Also, with A490 bolts, a hardened washer must be used under the element not turned if it bears against material with specified minimum yield strength less than 40 ksi (such as A36).

e. When tightened, the point of the bolt must be flush with or outside the face of the nut. Where necessary to achieve th requirement, the use of a longer length bolt, packed out w. additional flat washers, is acceptable.

f. Only plain A325 bolts may be reused. A490 and galvanized A325 bolts must not be reused after full tightening as stipulated in the AISC and RCSC specifications.[1,3] Fit-up bolts and retightening of the bolts loosened during torquing of adjacent bolts are not considered a reuse.

D. Special Problems

1. The RCSC specification contains an arbitration inspection procedure with which the inspector should be familiar. This involves the use of an "inspection torque wrench" which is calibrated by means of a device (Skidmore Wilhelm device) capable of measuring the actual bolt tension.

2. In fitting up an assembly for welding, it may be necessary to anticipate and compensate for distortion, in order to achieve a finished product which satisfies specified tolerances.

3. In bolted assemblies, connection attachments need not be flat in the connection (faying surface) plane bef. .e assembly, if it can be determined that the bolts, when properly tensioned, will provide faying surface contact.

4. The inspector should be aware that tack welds used for fitting are subject to the requirements of AWS D1.1.[2] These requirements are stringent and basically require that tack welds satisfy all the requirements of fillet welds unless they are removed or remelted in submerged arc welds.

5. Faying surfaces in friction-type connections require special treatment with regard to cleanliness.

6. Tightening a group of bolts should start near the group center or most rigid point and progress towards the edges or more flexible areas. This should be followed both in snugging up and final tightening.

E. Recommended Training

1. Codes and Standards. The inspector should receive formal classroom training or guided self-study covering the fitting and fastening areas of AWS D1.1[2] and the AISC Specification,[1] and covering the whole of the RCSC specification.[3] He should also have some formal training in the AISC QCIS booklet[9] and the AISC Code of Standard Practice.[8]

2. The inspector should be given some formal training in the reading of shop drawings and erection drawings.

3. The inspector should be given on-the-job training in the use of tape, square, sliding bevel square, inspection torque wrench, and Skidmore Wilhelm device.

4. The training should develop in the inspector an awareness of what a properly fitted-up assembly should look like and a confidence in the appropriateness of the tolerances involved.

F. Instructional Aids

1. Library of specifications, codes and manuals (see Appendix C, References), particularly Refs. 1, 2, 3, 8 and 9.

2. This Training Guide.

SECTION IV. WELDING

A. Introduction

Of the many operations performed by a steel fabricator, welding is the most complex, technically, and welding inspection requires a considerable amount of knowledge, experience and judgment. The function and safety of the finished product are directly related to welds being of proper quality. While welding is itself a simple operation, the production of quality welds is not. Among the characteristics which can be controlled by inspection to achieve welds of proper quality are (1) preparation of base metal, (2) control of welding materials (wire, rod, and flux), (3) joint geometry and fit-up, (4) welding position, (5) preheat, (6) inter-pass cleaning, (7) weld surface appearance and profile, (8) postheat. The inspector should receive training in each of the above items. Also, (9) he should be trained to recognize weld discontinuities, to know when discontinuities become defects which must be repaired. Finally, (10) he should receive some training in the nondestructive methods (see Section VI of this Training Guide) of evaluating welds, with special emphasis on when each method can be used and the strengths and weaknesses of each method.

The code most commonly used for structural welding is the American Welding Society's Structural Welding Code -- Steel (AWS D1.1).[2] This is the code upon which this section of the Training Guide is based.

B. Objective

The objective of weld inspection is the achievement of welds of proper quality, to ensure the safety and serviceability of the product without undue emphasis on cosmetic and other considerations which increase cost, without improving safety and performance.

C. Inspection Checks to be Made

1. Preparation of Base Metal

Surfaces and edges against which weld metal will be placed must generally be free of foreign matter such as loose mill scale, rust, oil, grease, mud, and moisture. These substances are sources of hydrogen which is a major cause of weld cracking. However, tightly adhering mill scale or rust need not always be removed. AWS D1.1, paragraph 3.2, provides requirements for cleanliness and for permissible discontinuities on edges and surfaces. Section VII of this Training Guide also discusses this topic.

2. Control of Welding Materials

Electrodes and flux are little affected by atmospheric temperatures, but they are affected by atmospheric humidity. They may pick up moisture from the air and, if the moisture is excessive, the quality of the weld can be adversely affected. When excessive moisture is present, the water is broken down into its components, hydrogen and oxygen, in the intensely hot arc. The hydrogen is then in atomic form, and some it will be readily absorbed by the hot molten metal and adjacent base metal. As the metal cools, most of the hydrogen escapes. But some of it may be retained in parts of the lattice structure of the steel, which may become super-saturated with hydrogen, particularly when the cooling rate is relatively high. It is believed that, on further cooling, this hydrogen reverts to molecular hydrogen under extremely high pressure. Internal stresses resulting from transformation, superimposed on weld shrinkage stresses, combine to stress the metal further during the latter stages of cooling. Consequently, underbead cracking may occur in the adjacent base metal or in the weld metal. Fractured surfaces of tension-test specimens exhibit small defects called fisheyes at points where tiny hydrogen-filled voids existed.

To prevent cracking, which might cause initiation of fracture, AWS D1.1 provides special requirements for storage of electrodes and flux for exposure time, for drying electrodes with low-hydrogen coatings if they have been exposed to humid air, and for rejecting electrodes that have been wet. The inspector must be familiar with these AWS requirements and check to assure that they are met.

3. <u>Joint Geometry and Fit-up</u>

There are many types of joint geometry. Part 2 of AWS D1.1 contains a wide variety of joint geometries which are termed "prequalified." This means that these geometries have been used for many years in the industry and have a history of successful use. Shown in AWS D1.1, Part 2, are many types of both complete and partial penetration groove welds, as well as fillet, slot, and plug welds. Groove welds may be butt, corner, or T-joint welds, and may be of single or double bevel or single or double V configuration. There are also joints with J or U shaped grooves.

The inspector should be familiar with the various kinds of joint geometries and with the symbols which represent these geometries on the shop drawings. These symbols, which are referred to as weld symbols, can completely define the geometry of a joint without the joint actually being detailed on shop or field drawings. They have been developed over many years to eliminate any ambiguity in their use. AWS <u>Symbols for Welding and Non-destructive Testing</u> A2.4[17] is a guide to the welding symbols and is recommended reading for all personnel involved in welding. These symbols also can be found in the AISC Manual, 8th Edition[14] on page 4-148.

Joint geometry and fit-up are intimately related. A joint must be properly fitted-up in order to achieve the intended joint geometry and the intended groove or fillet weld quality and size. AWS D1.1 provides tolerances on fit-up in Part 3 and tolerances on joint geometry in Part 5. These joint geometry and fit-up tolerances are brought together in Part 2 and listed for each prequalified joint geometry. These joint geometry and fit-up tolerances should be part of the weld inspector's general knowledge. In particular, because fillet welds are so common in structural welding, the inspector must thoroughly understand what must be done regarding fillet weld s when the fillet weld fit-up tolerance of 1/16" is not achieved. Also, he should know why AWS D1.1 paragraph 2.7.1.2 says that fillet weld size should preferably be 1/16" less than the plate thickness when placed against the edge of a plate over 1/4" thick. This is to insure foolproof determination of the true fillet weld size.

4. Welding Position

These are four basic welding positions (flat, horizontal, vertical and overhead), and weld can be satisfactorily placed in any of them. However, not all joint geometries can be used in all positions in all welding processes. This can be seen from Part 2 of AWS D1.1. Also, not all electrodes within a process are recommended for welding in all positions. The shop drawings or weld procedure should show any position limitations imposed by joint geometry, process, or electrode, and the inspector's function is to see that these limitations are adhered to. The inspector should also be aware that welders must be qualified for the various positions of welding. These requirements are detailed in AWS D1.1, Part 5C.

5. Preheat

Preheat is sometimes required by the welding procedure. Its function is to slow down the cooling of the deposited weld and thereby improve the quality of the weld and reduce shrinkage stress Certain welding processes and base metal thicknesses can only welded when proper preheat is applied. General requirements for preheat will be found in AWS D1.1, Part 4, Table 4.2. Generally, preheat is required for joints made in thick plates and parts, and when high-strength steel greater than 36 ksi is used. When properly preheated, the base metal is hot throughout its thickness and for a specified distance on each side of the area to be welded. Preheat can be measured by surface temperature gages or temperature sensitive crayons. One way to ensure that the parts are properly preheated is to measure the temperature on the side of the part opposite to that heated.

6. Interpass Cleaning

As each pass of a multipass weld is placed, the flux which provides the proper welding environment solidifies into slag. This slag must be removed before the next pass is placed or the weld will contain possibly detrimental slag inclusions and porosity, which may lead to rejection of the weld and its replacement with new weld. This expensive repair can be easily avoided by removing the slag.

7. Weld Surface Appearance and Profile

Much can be told about the quality of a weld from its appearance. While good appearance can be obtained for a weld with internal flaws, a weld with poor appearance is very likely to have ot hidden defects. The appearance of a weld can attest to the appropriateness of the current and voltage settings, travel speed, electrode types used, electrode condition, interpass cleaning, etc. AWS D1.1 provides criteria for weld appearance in paragraphs 8.15.1 and 9.25.1.

The profile of a finished weld may have considerable effect on weld performance under load. Moreover, the profile of a single pass or layer of a multipass weld may have a considerable effect on the tendency to produce such defects as incomplete fusion or slag inclusions when subsequent layers are deposited. Some aspects of profiles of fillet welds which are considered to be defects are listed in AWS D1.1, paragraph 3.6 and Figure 3.6. Among these are undersize, excessive convexity, overlap, and undercut.

Undersize fillet welds can be detected using a weld gauge. The usual problem here is unequal length of fillet weld legs due to welding in the horizontal position. The solution is another weld pass, but the inspector should be aware that AWS D1.1 actually allows undersize fillets in most cases, provided the underrun does not exceed 1/16" nor 10% of the length of the weld. See AWS D1.1, paragraphs 8.15.1.7 and 9.25.1.7.

Excessive convexity refers to a fillet weld or groove weld in which the weld metal bulges out from the specified size. It can cause a poor path for transfer of stress and result in weld failure under load. AWS D1.1 provides some limits for acceptable convexity in paragraph 3.6.1. A possible solution for excessive convexity is the placing of additional weld passes, but care must be taken not to produce too large a weld (overrun). AWS D1.1 is silent on oversize welds (overrun) which can be (and has been) interpreted to mean that none is allowed. The AISC QCIS booklet[9] provides some guidance in this area by recommending that either or both legs of a fillet weld be allowed by oversize of 1/8" without correction. If an occasional fillet weld is inadvertently made in excess of this oversize tolerance, and if there is no excess distortion, removal or correction should not be required. Such welds have no detrimental effect, but removal and rewelding may induce serious shrinking stresses and distortion.

Overlap is usually associated with fillet welding, but also can occur at the edge of groove welds. The term describes a protrusion of weld metal beyond the bond line at the toe of a weld. This condition tends to produce notches that are obviously harmful, because of the resultant concentrations of stress under load, and also reduces the effective size of the fillet. Overlap is usually caused by incorrect welding techniques or welding currents, and should be eliminated by gouging, grinding, etc.

Undercut is the term used to describe a groove melted into the base metal adjacent to the toe of a weld and left unfilled by the weld metal. It also describes the melting away of the sidewall of a welding groove at the edge of a layer or bead. This melting away of the groove forms a sharp recess in the sidewall in the area in which the next layer or bead must fuse. Slag may be "keyed" into this undercut which, if not removed prior to subsequent passes, may become trapped in the weld. An undercut, therefore, is a groove that m vary in depth, width, and sharpness at its root.

AWS D1.1 limits the degree and extent of undercut as a function of structure end use and type of stress in the base metal. In addition to these, the inspector should be aware that while every effort to minimize undercut should be made during welding, not all undercut is equally damaging.

For example, undercut which occurs in the edge of a cover plate which is fillet welded to a beam flange has no effect on weld or cover plate performance. Also, stiffeners welded to the web of a beam are usually welded with the beam web flat, which forces any undercut to occur in the stiffener and not the web. This undercut will not interfere with the performance of the stiffener.

8. Postheat

Postheat is also referred to as stress relief treatment, which is descriptive of the function of postheat. Postheat may be performed in ovens or with portable induction heaters or gas torches. When welded items are postheated, inspection of welding may be required both before and after such treatment. It would be prudent to perform inspection before postheating to avoid postheating a defective item, which when subsequently found to be defective would need to be repaired and postheated a second time. Also, postheating may give rise to weld defects such as cracks. Thus, inspection should al be performed after postheating.

9. Discontinuities and Defects

The inspector should be aware that a discontinuity is not necessarily a defect. AWS D1.1 contains definitions for these terms in Appendix I. A discontinuity is defined as "an interruption of the typical structure of a weldment such as a lack of homogeneity in the mechanical or metallurgical or physical characteristics of the material or weldment. A discontinuity is not necessarily a defect." A defect is defined as "a discontinuity or discontinuities which by nature or accumulated effect render a part or product unable to meet minimum applicable standards or specification. This term designates rejectability."

In order to determine if a discontinuity is a defect, a code or standard or specification which provides "acceptance criteria" is required. The acceptance criteria will be dependent upon the inspection method used and the end use of the product or part. For instance, AWS D1.1 contains acceptance criteria for visual, magnetic particle, radiographic, and ultrasonic inspection methods. Also, different criteria are specified for buildings and bridges.

The following paragraphs discuss the various types of discontinuities which will occasionally occur during welding.

a. Porosity

Porosity is the term for the gas pockets or voids, free of any solid material, that are frequently found in welds. Porosity can come from gases released by the cooling weld metal because of reduced solubility as the temperature drops, and from gases formed by chemical reactions in the weld. Porosity may be scattered uniformly throughout the entire weld, isolated in small areas, or concentrated at the root. Pores are usually spherical in shape, although they may also occur as non-spherical pockets along grain boundaries or as elongated tubular voids called piping porosity or "wormholes." Most welds contain some amount of porosity which may be macro or micro in size.

It is an established fact, and one worthy of inspection recognition, that scattered porosity well in excess of that permitted by most codes does not detract from the static strength of a welded joint.

b. Slag Inclusions

This term is used to describe the oxides and other nonmetallic solids that are entrapped in weld metal or between weld metal and base metal. Slag inclusions may be caused by contamination of the weld metal by the atmosphere. They are generally derived from electrode covering materials or fluxes employed in arc welding operations. In multilayer welding operations, failure to remove the slag between layers will result in slag inclusions in these zones.

The majority of slag inclusions may be prevented by proper preparation of the groove before each bead is deposited, using care to correct contours that will be difficult to penetrate fully with the arc.

Figure 1 shows slag inclusions to be discontinuities embedded in the weld. Therefore, visual inspection will not detect this type of discontinuity. The inspector should insure that proper cleaning methods are used in multipass welds to minimize the occurrence of slag inclusions.

c. Incomplete Fusion

Incomplete fusion, frequently termed "lack of fusion", describes the failure of adjacent weld metal and base metal to fuse together completely. (See Figs. 2 and 3) This failure to obtain fusion may occur at any point in the weld.

Incomplete fusion may be caused by (1) failure to raise the temperature of the base metal (or previously deposited weld metal) to the melting point or (2) failure to remove slag, mill scale, oxides, or other material alien to the base metal, which may be present on the surfaces with which the deposited metal must fuse. Incomplete fusion is usually elongated in th direction of welding and may have either rounded or sh edges, depending on how it is formed.

d. Inadequate Joint Penetration

This term is used to describe the condition where joint penetration is less than that specified. Hence, partial joint penetration may or may not be a defect, depending on what is specified for that particular joint.

The root area shown in Fig. 4 shows a case of inadequate penetration if penetration to the full depth of the bevel groove was specified. However, if partial penetration was specified (see weld symbol), the joint may be satisfactory. The AISC QCIS booklet[9] points out in Chapter 4 that the evaluation of partial penetration joints should be limited to the effective throat, not the depth of penetration. If complete penetration is required, as it usually is in highly stressed joints, then inadequate penetration or partial penetration is a defect, because it leaves a very sharp notch at the root of the weld.

The most frequent cause of this type of defect is a groove design not suitable for the welding process used or for the conditions of actual construction. When a groove is welded from one side only, complete penetration is not likely to be obtained consistently with any arc welding process if (1) u of the wrong size of filler metal has made the root face d mension too great, even though the root opening is adequate, (2) if the root opening is too small, or (3) if the included angle of a V-type groove is too small.

If the design is known to be satisfactory, inadequate joint penetration may result from the use of too large an electrode (leading to improper heat distribution), an abnormally high rate of travel, or the use of insufficient welding current.

e. Cracks

Cracks are linear ruptures of metal under stress. Although sometimes wide, they are often very narrow separations in the weld or adjacent base metal. Usually little deformation is apparent. Three major classes of cracks are generally recognized: hot cracks, cold cracks, and microfissures. All types can occur in the weld or base metal.

Figure 5 illustrates a variety of cracks, including underbead cracks, toe cracks, crater cracks, longitudinal cracks, and transverse cracks. The underbead crack is a base metal crack usually associated with hydrogen. Toe cracks can be of similar origin. Crater cracks are shrinkage cracks which result from stopping the arc suddenly.

f. Arc Strikes

Arc strikes represent unintentional melting or heating outside the intended weld deposit area. They usually are caused by the welding arc, but can be produced beneath an improperly secured ground connection or magnetic particle inspection electrical connections (prods). The result is a small re-melted area that can produce undercut, hardening, or localized cracking, depending upon the metal composition. Such an area can be a stress raiser, which is detrimental to the serviceability of structures subject to fatigue conditions.

The AISC QCIS[9] recommends that the engineer identify areas of members subject to critical fatigue conditions. In these areas arc strikes should be removed by grinding. AWS Code D1.1 says in Section 3.10 that cracks or blemishes caused by arc strikes must be ground to a smooth contour and checked to insure soundness. This applies to all situations, not just critical fatigue areas. The inspector should be aware that this is an area which requires judgment on his part.

10. Inspection Methods

a. Visual

Visual inspection is the primary weld inspection method. It is performed before, during, and after fabrication. It is performed by everyone connected with the job, not just the steel inspector, and can therefore be considered a production function as well as an inspection function.

Visual inspection prior to welding is used to determine proper joint fit-up (Fig. 6), alignment (Fig. 7), and surface preparation. In this way expensive weld repairs to correct weld discontinuities and mismatch are avoided. Prior to the arc strike, the weld inspector should examine the material to be joined to be certain that it:

- Meets specification requirements for quality
- Is correct type and size
- Is free of grease, paint, oil, oxide film and heavy mill scale
- Is within tolerances for straightness, flatness and dimensions

And should also check the welding process to determine if

- The process is correct
- A procedure has been established
- Electrode type and size are correct
- Electrode condition is satisfactory
- Equipment settings for voltage and amperage are correct

Visual inspection during welding insures proper cleaning, which will prevent discontinuities such as lack of fusion, porosity, slag inclusions, lack of penetration, and cracks. The root pass in a multipass weld is the most critical one from the standpoint of weld soundness. It is especially susceptib to cracking and, because it tends to solidify quickly, is pro. to trap gas and slag. Subsequent passes are subject to a variety of weld defect-creating conditions that result from the shape of the weld bead or change in the configuration of the joint. These can be visually detected by the inspector, and repair cost can be minimized if the problem is corrected before welding progresses. During welding, checks should be made for the following defects (see Fig. 8):

- Cracks
- Surface slag inclusions
- Surface porosity
- Undercut

Visual inspection after welding is useful even if other methods, such as magnetic particle or ultrasonic inspection, are to be used. This visual inspection will reveal cracks, porosity, crater, undercut, improper profile, laps, and undersize and oversize welds. These problems may need correction; if so, they should be fixed before the weld is submitted to more sophisticated inspection methods.

b. Other Methods of Inspection

Other commonly used inspection methods are: magnetic particle inspection, liquid penetrant inspection, ultrasonic inspectio and radiographic inspection. These methods are described in Recommended Practice SNT-TC-1A[12] of the American Society for Nondestructive Testing (ASNT), and are discussed in Section V of this Training Guide.

D. Special Problems

Welds deposited through heavy mill scale may have undercut, overlap, lack of fusion, or fusion line cracks which are obscured by the mill scale. If this steel is blast cleaned after fabrication, these discontinuities may become visible and lead to rejection at a later stage of fabrication. which repair is more expensive, or, worse still, they may necessitate expensive field welding repairs. On the other hand, the weld inspection should be done before blasting, because the peening effect of the blasting medium can close up small surface cracks in the weld and make them invisible.

Some weldments, especially those involving heavy plates and sections, are subject to a form of cracking called delayed cracking. This may not appear for days or weeks after welding is completed. Thus, the inspection performed immediately after welding may sometimes need to be followed by a delayed inspection.

If all the various inspection methods were applied to a particular weld, there would often be disagreement as to whether or not the weld is satis factory. This is why AWS D1.1 clearly states that the method of in spection must be stipulated in the contract requirements. Also, not all methods can easily be applied to all welds. For instance, AWS D1.1 does not cover radiographic and ultrasonic inspection of fillet welds. Also, ultrasonic inspection is not intended by AWS D1.1 to be used for groove welds in plates less than 5/16" thick.

When weldments are postheated, the inspector needs to know if weld inspection is to be done before or after such treatment. AWS D1.1 is silent on this. The inspection, could of course, be done both before and after, but this is perhaps an unnecessary expense.

E. Recommended Training

1. Codes and Standards

The inspector should receive formal classroom training or guided self study covering the Prequalified Procedures, Workmanship, and Technique areas of AWS D1.1.[2] He should also receive instruction on the use of training in the AISC booklet Quality Criteria and Inspection Standards (QCIS)[9] and the AISC Code of Standard Practice.[8]

2. Welding Inspection Literature

Training should include the introduction of the inspector to publications dealing with welding and welding inspection, such as:

- AWS Certification Manual for Welding Inspectors[18]
- AWS Welding Inspection[19]
- Lincoln Electric Company Procedure Handbook of Arc Welding Design and Practice[20]
- AWS Welding Handbook, 7th Edition[21]

3. Welding Symbols and Shop Drawings

Formal classroom training or guided self study should be provided on AWS A2.4[17], which covers symbols for welding and nondestructive examination. Also, inspectors should be trained formally or "on the job" in the reading of shop drawings and, if possible, erection drawings.

4. Weld Inspection Tools and Methods

Formal and on the job training should be provided in the use of weld size gages and nondestructive examination (NDE) methods. The training in NDE methods is to create an awareness in the inspector regarding the capabilities and limitations of NDE, not to make him competent in the use of these methods.

5. The training programs should develop in the inspector an instinctive awareness of what constitutes satisfactory welds and what constitutes unsatisfactory welds.

F. Instructional Aids

1. Library of specifications, codes and manuals (see Appendix C, References), particularly Refs. 2, 9, 12, 14, 17, 18, 19, 20, 21, 22.

2. This Training Guide

3. AWS Package Courses

   a. Current Welding Processes
   b. Introductory Welding Metallurgy
   c. Fundamentals of Welding Inspection

4. American Society of Metals (ASM) and American Society for Nondestructive Testing (ASNT) Course #5 - Fundamentals of Nondestructive Testing

5. Equipment:

   a. Weld size gauge
   b. Magnifying glass
   c. Pocket rule
   d. Straight edge
   e. Workmanship standards
   f. Flashlight

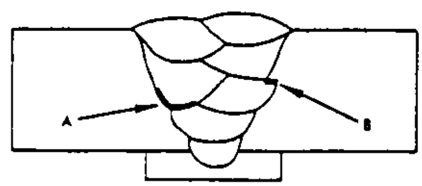


Fig. 1 - Slag inclusions, between passes at A and B
(from AWS Welding Handbook 6th Edition)

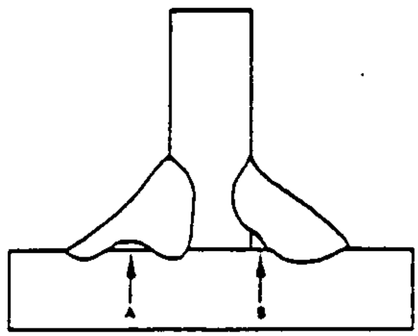


Fig. 2 - Incomplete fusion in fillet welds. B is often termed "bridging."
(from AWS Welding Handbook, 7th Edition)

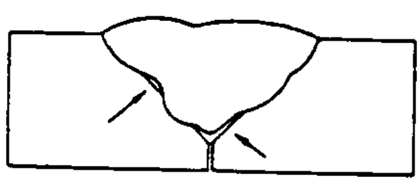


Fig. 3 - Incomplete fusion in a groove weld
(from AWS Welding Handbook, 7th Edition)

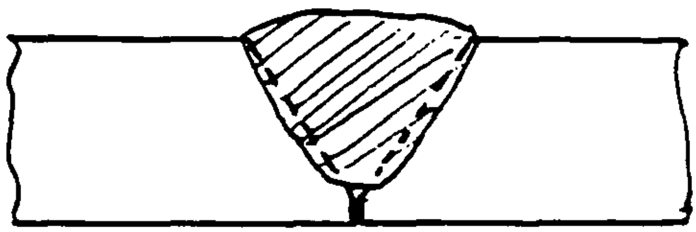


Fig. 4 - Inadequate penetration

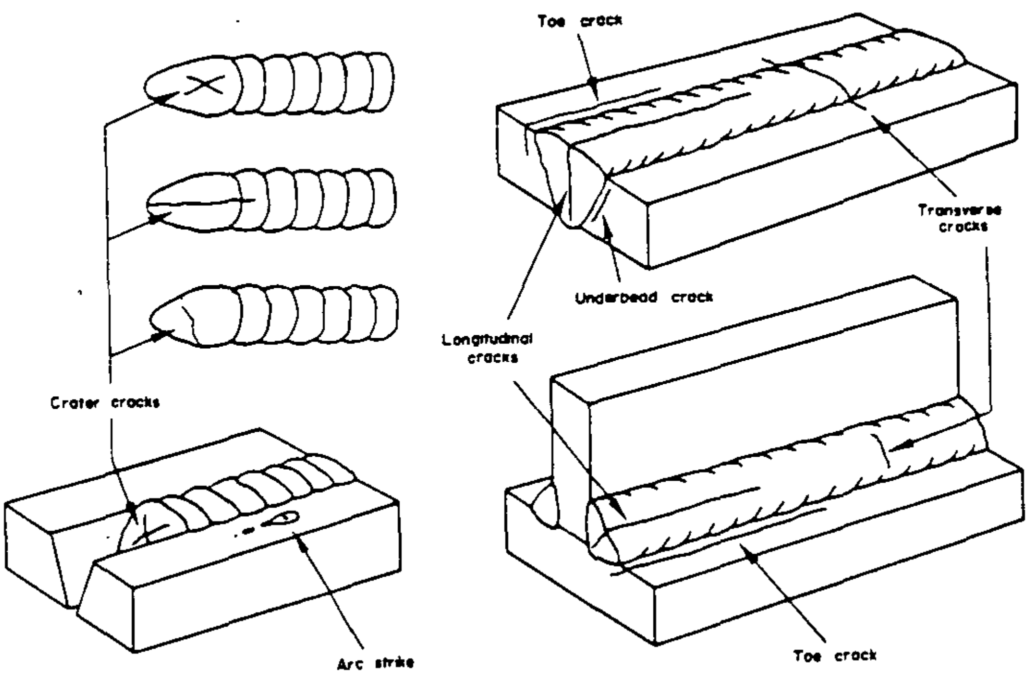


Fig. 5 - Types of cracks in welded joints
(from AWS Welding Handbook, 7th Edition)

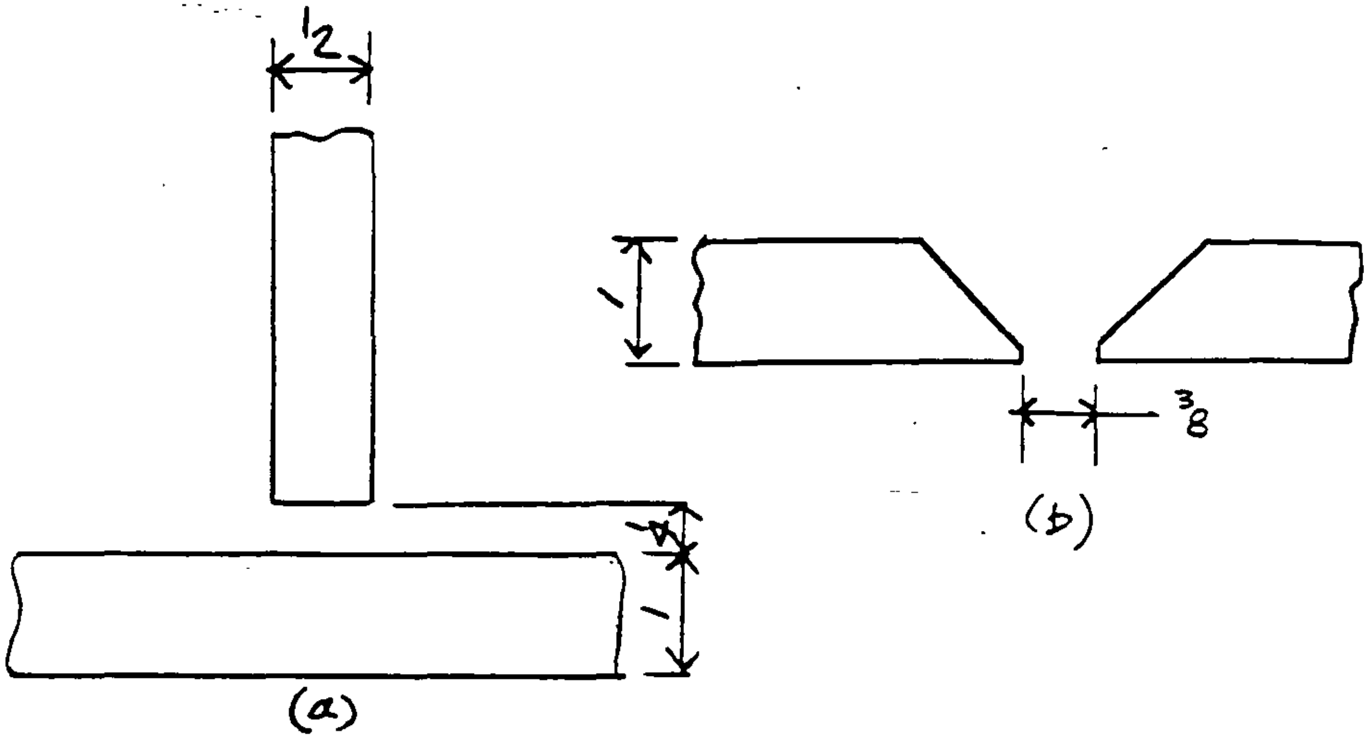


Fig. 6 - Excessive root opening: (a) fillet weld, (b) groove weld

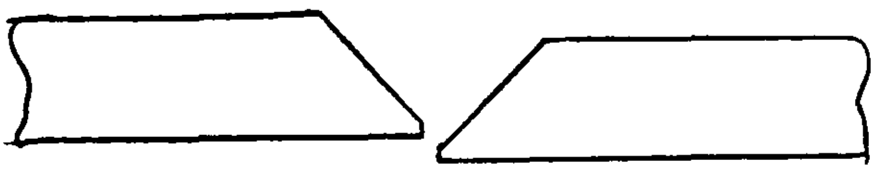


Fig. 7 - Misaligned groove joint

Cracking - can be detected visually unless the cracks are internal or are very fine. It is usually associated with excessively high current, insufficient preheat, insufficient bevel angle and/or highly restrained joints.

Surface Slag Inclusions - are usually clues to faulty technique. Improper electrode manipulation, improper electrode size or too steep a downhill angle are typical possibilities.

Surface Porosity - is often caused by excessive speed, rusty or dirty plate, wet electrode or flux, insufficient flux coverage, or critical arc blow conditions.

Undercut - results from poor procedural selections - - may be due to too large an electrode, excessive current, voltage or arc speed.

Figure 8

SECTION V. SHOP ASSEMBLY

A. Introduction

The term shop assembly as used in this section refers to the assembly in the shop of several individual shipping pieces. This operation will usually involve assembly of the members to perform one of the following operations:

- Drill or ream field splice connections
- Verify the accuracy of previously drilled or punched field connections
- Prepare and check the ends of members that are to be field welded

Shop assembly is generally used in the fabrication of bridge structures. Certain other structures with complicated geometry or complex connections may require partial assembly to verify the accuracy of fabrication.

B. Objectives

1. Documents relative to bridge structures will generally specify the degree of shop assembly required. Other type structures may or may not specify shop assembly. Although specifications may r require shop assembly, individual shops may assemble certain comp connections to verify their accuracy. The inspector should be awa of these conditions and advise the operating personnel accordingly.

2. Aside from the usual tolerances for dimensions such as bearing location, camber, and splice gap, other less obvious tolerances may be critical. Tolerances for edge distance, girder depth, flange tilt, and fit of splice material are also important.

C. Inspection Checks to be Made

Checks to be made include the following, as applicable:

1. Material

   Many times members to be shop assembled will be very nearly identical. The inspector should verify that the proper members, including appropriate splice material, are used in the assembly. He should also check to be certain that the piece mark end is in its proper location per the assembly drawing.

2. Dimensions

   Bearing location, camber or blocking dimensions, and other check points should be verified by the inspector. Tapes used in checki dimensions should be verified for accuracy.

3. Splices

The inspector should check each splice to verify the following:

a. The correct splice material has been applied.

b. The edge distance for splice material and main members is satisfactory. A careful check on the position of the inside splice plates is often necessary to avoid insufficient edge distance or interference with the flange-to-web weld.

c. The depth of matching ends of main members is within the appropriate tolerance. Fill plates may be required even though the depths are within the appropriate tolerances. The use of fill plates may be restricted in certain state specifications.

d. The tilt of adjacent flanges is within the appropriate tolerance.

e. Splice material and main material is securely clamped or fit-up bolted prior to drilling or reaming.

f. Splice material and main material is properly match-marked prior to disassembly.

g. The gap at the field splice is within the appropriate tolerance.

h. The preparation of joints to be field welded is in accordance with the appropriate specifications.

D. Special Problems

1. Haunched Girders

There are several special problems that require attention of the inspector.

a. Bearing-to-bearing dimensions require care when checking. The required tape tension will vary with the unsupported length of the tape.

b. Review the drawing carefully to be certain that the dimensioning shown is in agreement with dimensions measured.

c. The depth of adjacent girders may vary due to sloping bottom flange. The depth should be checked prior to cutting the girder to length. Special care is necessary when fitting the bottom flange to the web to maintain the proper depth at field splices.

2. Splice Alignment

The inspector should verify that the longitudinal alignment is correct. When members are drilled or reamed with the web in a horizontal position, the supporting skids should be positioned so that the girder deflection does not cause a kink in the field splice.

For girders which are curved prior to shop assembly, the inspector should measure a mid-ordinate for a short chord length (4 feet) at the splice.

3. Curved Girders

Assembly requirements for curved girders vary from state to state. When curving is done in the shop after assembly, an adjustment must be made to the bearing location to allow for shrinkage due to the heat curving.

E. Recommended Training

Recommended training for shop assembly inspection includes:

1. Classroom training and/or guided self-study sufficient to enable the inspector to be knowledgeable about topics related to shop assembly.

2. Training in reading shop drawings and assembly diagrams.

3. Training in the use of a surveyor's level and the use of tapes.

4. In-shop training and experience to determine that the shop assembly has been properly performed.

F. Instructional Aids

1. Various state codes and specifications on shop assembly.
2. The proper use of calibrated tapes.
3. Use of a surveyors level.
4. This Training Guide.

SECTION VI. NON-DESTRUCTIVE EXAMINATION

A. Introduction

Non-destructive examination of welds can be accomplished by five methods:

1. Visual Inspection
2. Dye-Penetrant Inspection
3. Magnetic Particle Inspection
4. Ultrasonic Inspection
5. Radiographic Inspection

Non-destructive examination of welding that is to be performed is stipulated in the contract documents. The welds to be examined, the method of examination, the extent of examination and the standard of acceptance will be designated.

Acceptance standards for various methods of examination are given in the AWS Structural Welding Code -- Steel D1.1[2]. Individuals that are qualified to perform Magnetic Particle, Ultrasonic and Radiograph Inspection are specialists trained under the provisions of ASNT SNT-TC-IA[12]. Performance of these types of inspection is not part of the training required for Structural Steel Shop Inspector.

B. Objectives

The inspection objectives of Non-destructive Examination are:

1. To assure that all required welds are examined.
2. To assure that the proper method of examination is performed.
3. To assure that acceptance criteria for the examination have been met.

C. Inspection Checks to be Made

Checks to be made include the following:

1. Welds to be Examined. The inspector checks which welds are to be examined, the method of examining, and the extent of examination.

2. Visual Examination. The inspector should examine the weld visually for correct size, indication of undercut, profile of fillet weld, porosity, slag inclusion or incomplete fusion.

3. Dye-penetrant Examination. The inspector should examine welds where required by use of dye-penetrant to determine surface cracks.

4. Other Examination. The inspector should determine which welds require magnetic particle, ultrasonic or radiographic inspection, and verify that they are examined by a qualified person, and check to see that the results are correctly interpreted.

D. Special Problems

The acceptability of the welded product is dependent on the interrelations and variations of individual elements of workmanship.

1. Consideration must be given to the following major elements essential to achieve sound welds: joint preparation, fit-up, cleaning, preheat, techniques, processes and procedures.

2. Proper setting of volts and amps on the welding equipment is an additional element essential to achieve sound welds.

E. Recommended Training

Recommended training for Non-destructive Examination includes:

1. Study of applicable codes. The following codes should be available for reference, and the inspector should be given classroom training or guided self-study sufficient to enable him or her to be knowledgeable as to the topics covered in each and to be familiar with important provisions.

   AWS Structural Welding Code - Steel D1.1[2]
   AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings.[1]
   AISC Manual of Steel Construction[14]
   Quality Criteria and Inspection Standards[9]
   AISC Code of Standard Practice[8]
   AASHTO Standard Specification for Highway Bridges[4]
   AREA Manual for Railway Engineering[16]
   Other state bridge codes applicable to the fabricator

2. The inspector should be indoctrinated in the capabilities and the limitations of various types of NDE. Training should be directed toward enabling the inspector to call an NDE specialist appropriate for the situation to be investigated.

3. The inspector should be proficient in visual inspection and the use of dye-penetrant inspection.

F. Instructional Aids

Instructional aids include:

1. Library of specifications, codes and manuals (see Appendix C, References)
2. This Training Guide
3. Articles on pamphlets comparing NDE methods

SECTION VII. SURFACE PREPARATION

A. Introduction

1. Surface preparation is the cleaning of structural steel to receive a shop coat of paint, to provide an acceptable surface for unpainted weathering steel, or to provide acceptable surfaces for members or components in contact with concrete or fireproofing materials.

   Cleaning prior to painting is the most common use. Many of today's highly sophisticated paint systems are extremely sensitive to the quality of the surface preparation. Paint problems that do develop are often attributed (rightfully or wrongfully) to improper surface preparation.

   Precleaning of steel plates and shapes prior to fabrication is a common practice for some fabricators. This practice provides a clean surface for welding and other shop operations.

2. Surface preparation consists of cleaning the surface of the structural steel to remove mill scale, grease, oil, dirt, weld spatter, moisture, and other contaminants to a degree specified in the contract documents.

3. Methods commonly used to clean structural steel are blast cleaning, power tool cleaning, hand tool cleaning and solvent cleaning.

4. The most common specifications used are those published by the Steel Structures Painting Council (SSPC). The SSPC Specifications[11] describe in detail the methods of cleaning and degree of cleanliness required. The most common cleaning requirements are those listed below:

   SSPC-SP1 Solvent Cleaning
   SSPC-SP2 Hand Tool Cleaning
   SSPC-SP3 Power Tool Cleaning
   SSPC-SP5 White Metal Blast Cleaning
   SSPC-SP6 Commercial Blast Cleaning
   SSPC-SP7 Brush-off Blast Cleaning
   SSPC-SP10 Near-White Blast Cleaning

   Additional cleaning requirements supplementing the SSPC Specifications will usually be included in the paint manufacturer's recommendations. These recommendations are outlined in the "Data Sheet" for the paint supplied.

   Construction specifications, special provisions and shop standards may also impose requirements on the Surface Preparation operation. The AISC publications A Guide to the Shop Painting of Structural Steel[10] and Quality Criteria and Inspection Standards[9] provide information that is pertinent and applicable.

B. Inspection Checks to be Made

1. Blast mixture check. The SSPC specifies the maximum particle si. allowed. The maximum abrasive particle size is no larger than that passing through a #16 mesh screen. The maximum allowable grit size is S.A.E. No. G-25. The maximum allowable shot size is S.A.E. No. S-330.

   Certain specifications may require a mixture of shot and grit.

   The inspector should ascertain that the proper size abrasive has been used. If a shot-grit mixture is specified, the supplier should be able to aid in determining the mixture ratio that is in the machine.

2. Surface profile. The surface profile can be extremely important to the performance of the paint system. The SSPC specifications do not specify profile requirements, other than indicating that the maximum profile height should not be detrimental to the life of the paint film.

   Profile height may be specified in the contract documents. The paint manufacturer's recommendation should be followed. The profile shape can be of considerable importance for certain type paints. The use of S-330 shot (maximum size permissible in SSPC) will ordinarily provide the maximum profile height as well as a profile shape that is rounded at the peaks and valleys. Adding an appropriat grade of grit to a smaller shot (S-230 or S-270) will ordinari sharpen the profile shape and reduce the profile height to a leve that will improve the performance of paints that are sensitive to the rounded profile.

   Surface profile may be checked by any one of several means. Roughness gauges and visual comparators are the most common methods of judging the profile. These comparators are available for either shot, shot/grit, or sand blasted surfaces. The comparators are electroformed copies of a master, and as such may be open to subjective comparison with the as-blasted conditions.

   Blast samples, approved by the customer, using the proposed blasting materials and having documented profile readings, provide a direct means to compare profile and assure common acceptance criteria.

3. Degree of cleanliness. A number of visual aids are available to judge the surface profile and degree of cleanliness. Color Photographic Standards for Surface Preparation[22] (SSPC-Vis 1) is a

publication of photographic standards which may be used to help judge the cleanliness of steel surfaces. Judging the cleanliness of a surface tends to become very subjective. The SSPC specifications define the degree of cleanliness for each of the cleaning categories. The inspector should be familiar with the definitions, particularly as they apply to the amount of mill scale, shadows, discolorations residue, etc., that may remain on the cleaned surface.

4. Degree of Inspection. The degree of inspection required will normally be proportionate to the type of cleaning specified and the sophistication of the paint system to be applied. The Contract Documents may outline very detailed procedures for inspection requirements; the inspector should satisfy himself that the cleaning procedures are in accordance with SSPC requirements, paint manufacturers' requirements, and fabricator standards.

C. Special Problems

1. Oil, grease, soil and other contaminants. These may be removed by procedures outlined in SSPC-SP1 "Solvent Cleaning". Solvent cleaning may be the only method specified; however it is very often used in conjunction with other cleaning methods.

   Final wiping or brushing should be done with clean solvent and clean brushes, otherwise the contaminants are merely spread over the surface. There should be no detrimental residue left on the surface. Wiping of previously blast cleaned surfaces with rags should be avoided.

2. Rust formation. The SSPC specification reads, "If any rust forms after blast cleaning, the surface shall be reblast cleaned before painting." The inspector should be aware of the ambient conditions that cause rust formation and visually check blasted material for rust formation.

3. Weld Spatter. The removal of weld spatter occasionally causes disagreement among interested parties. In the absence of other specification requirements, the AISC publication Quality Criteria and Inspection Standards[9] (QCIS) should be considered. It states, "Moderate amounts of tight spattered weld metal which are not detrimental to the performance of the structure or the coating system need not be removed."

4. Loose abrasive. Loose abrasive should be removed from field faying surfaces; particular attention should be given to items loose bolted for shipment.

   Trapped abrasive at toes of angles, pockets and crevices should be removed.

5. Mill defects. After blast cleaning, surface defects in mill material may be visible. The inspector should refer to the appropriate specifications as well as the AISC QCIS[9], Section III G.

6. Inaccessible surfaces. Fabricated members may, on occasion, conta·· surfaces that are inacessible for a particular cleaning meth Whenever possible, this condition should be anticipated and consideration given to cleaning these surfaces prior to the fitting operation. This may not be practical or advisable for all such inaccessible surfaces. Solvent or hand tool cleaning may be an acceptable alternate.

D. Recommended Training. The inspector should receive both classroom and in-shop training on surface preparation requirements and inspection techniques.

1. Applicable Codes. The inspector should have a general knowledge of the pertinent provisions of the AWS, SSPC and AISC specifications.

2. Instruments. The inspector should receive training and experience in the use of the various surface comparators and photographic standards SSPC-Vis 1.

3. In-shop Training. The inspector should receive adequate in-shop training and experience to evaluate various cleaned surfaces as to their conformity to a given specification.

E. Instructional Aids. The following items should be available for inspector training and use.

1. Library of specifications, codes and manuals (see Appendix C, Refe ences)
2. Surface profile comparators
3. Typical blast samples of known profile
4. Blasting equipment manufacturer's publications

STRUCTURAL STEEL SHOP INSPECTOR TRAINING GUIDE

SECTION VIII. SHOP PAINTING

A. Introduction

1. With the ever-increasing complexity of paint systems and the environments to which they are subjected, the inspector's role becomes increasingly important. A familiarity with standard preparation and painting specifications is necessary.

   The shop coat of paint is an important part of the fabricating operation; aside from the obvious rust inhibiting function, the quality of the fabricated product will, more often than not, be first judged by the appearance of the shop painted members. A properly painted job will do much to demonstrate that the shop is producing a quality product.

2. There are many types of paint systems in current use. The degree of sophistication as well as the pertinent cleaning, storage and application parameters vary significantly from system to system. Painting specifications generally cannot cover these detail requirements. The contract documents will often contain detailed information and requirements that apply to the specified paint system. The paint manufacturer will normally supply appropriate data sheets tailored for the type of paint to be applied.

   The technical services department of many manufacturers are willing to provide assistance in all phases of the painting operation. Whenever an unfamiliar paint system is specified or problems or questions develop relative to the coating, the manufacturer should be consulted.

   The SSPC publication titled Measurement of Dry Paint Thickness with Magnetic Gages,[23] SSPC-PA2, provides commonly accepted requirement for the measurement of dry film thickness. The AISC publications A Guide to the Shop Painting of Structural Steel[10] and Quality Criteria and Inspection Standards[9] also provide excellent guidance and should be used as reference documents.

   Individual fabricators may have their own guidelines for the standard shop coat that is often specified. These requirements should be available to the inspector.

B. Inspection Checks to be Made

1. Prepaint

   a. Elasped time prior to painting. Many specifications have a maximum time allowance between the cleaning and painting operation. Cleaned surfaces should be checked for rust bloom prior to coating.

Occasionally a second coat will be applied in the shop. Most systems will specify a minimum time for curing prior to plying the second or successive coats.

b. Unpainted areas. Certain parts of most jobs are to be unpainted. The inspector should review the drawings to ascertain that these areas are not coated. These could include such areas as faying surfaces of field bolted connections, areas to be field welded, embedded or fireproofed, or machine finished surfaces.

c. Climatic conditions. The inspector should verify that the relative humidity, dew point, ambient temperature and steel surface temperature requirements are satisfactory. He may also need to consider the expected climatic conditions during an: after painting.

d. Striping. Specifications may occasionally require that certain areas be given a coat of paint prior to the general paint application. This operation is generally termed "striping" and could apply to such items as plate edges, welds, bolt heads or nuts.

e. Paint. Manufacturers' certification and/or customer testing are often required. The inspector should be certain that these are in order.

Shelf life and storage conditions are important with cert paints. The manufacturers' recommendations should be followeu.

The manufacturers' recommendations may also speciiy types of thinners and mixing instructions.

2. Application

a. Application method. Is there a recommended application method (brush, air spray, airless)? Are the recommended equipment and pressures used?

b. Technique. The manufacturers' recommendations relative to spray pattern, overlap, gun to surface distance, etc. should be observed.

c. Wet film thickness. Some specifications may require a minimum wet film thickness. Where they do not, it may be advantageous to make these measurements in order to assure adequate and consistent coating thickness.

d. Dry Film Inspection.
   1) Dry film thickness measurements to be made in accordance with specifications.

2) Dry film to be checked for runs, sags, wrinkling, overspray, holidays, dryspray, pinholes, mudcracking, etc. These conditions to some degree are unavoidable, and acceptance will likely be a subjective matter. The type of finish coat may be of considerable importance and the manufacturers' recommendations should be obtained.

e. Handling and Storage. Handling scars should be minimized and skids should be clean. Is the material properly stored above ground and adequately blocked? Has the material been protected from dust and dirt during curing? Has touch-up after handling and storage been completed prior to shipping?

C. Special Problems

1. Wet film thickness. Paint specifications will occasionally specify a minimum wet film thickness. Gages are available that accurately and quickly measure the wet film thickness. Although the wet film thickness may not be a specification requirement, its constant use by the painter provides a very good means for controlling the thickness of the paint film. Readings should be taken within a few seconds after application to minimize the effect of solvent evaporation. Wet film thickness should be correlated with actual dry film thickness measured on several areas or calculated from the volume of percent solids of the paint.

2. Dry film thickness. Dry film thickness is the more common method for specifying film thickness. The dry film thickness required on blast cleaned surfaces is the paint thickness above the peaks of the surface profile. In the absence of other testing requirements, the SSPC publication titled Measurement of Dry Paint Thickness with Magnetic Gages,[23] SSPC-PA2 provides a commonly accepted method for taking the dry film thickness measurements.

   There are many types of magnetic gages available to measure the dry film thickness. They are generally classified as either pull-off gages or fixed probe gages. Both type of gages are often used; however, the pull-off type is commonly preferred by steel fabricators. If the specification requires, or the customer uses, a gage different from that used by the fabricator, it would be prudent to compare measurements on other samples prior to starting the paint operation.

   Calibration and measurement procedures are described in detail in SSPC-PA2[11]. Note that SSPC-PA2 recommends five evenly spaced spot readings per 100 square feet. Each spot reading consists of the average of three closely spaced point readings.

3. Mud cracking. Mud cracking of zinc-rich paints can be difficult to detect. Fillets of rolled shapes, corners, or areas where overlapping spray patterns may occur should be carefully checked. Visible (without the aid of magnifiers) mud cracking should be removed and the areas repainted.

4. Inaccessible surfaces. Certain surfaces of fabricated members mav be inaccessible to the normal application of the shop coat. ' inspector should be constantly aware of this possibility so tı. unpainted surfaces are not inadvertently allowed to be shipped to the job site.

D. Recommended Training. The inspector should receive both classroom and in-shop training per this section.

1. Applicable Codes. The inspector should be knowledgeable in the pertinent provisions of the codes or documents listed in Appendix C, References, particularly Refs. 1, 4, 6, 8, 9, 10, 11, 14, 23.

2. Instruments. The inspector should receive training and experience in the use of instruments that measure dew point, relative humidity, steel temperature and ambient temperature. The inspector should be thoroughly trained in the calibration and use of the various types of wet or dry film thickness measurement gages.

3. In-shop Training. The inspector should receive adequate in-shop training and experience to evaluate commonly used paint systems as to their conformance to a given specification.

E. Instructional Aids

1. Library of specifications, codes and manuals (See Appendix C, Refe ences)
2. Typical paint manufacturers' data sheets
3. Typical job specifications and special provisions
4. Appropriate instruments to measure ambient eliminate conditions
5. Wet film thickness gages
6. Dry film thickness gages
7. Shims for calibrating thickness gages
8. Typical painted samples of known thicknesses
9. Seminars by paint manufacturers are available

APPENDIX A.

TOLERANCES

A. Introduction

1. Many of the tolerances discussed in this appendix have been covered in the various sections of this publication. They are included in the Appendix for easier use and for comparision of various tolerances specified by different codes or documents.

2. Each operation performed in the shop will have an attendant tolerance. These tolerances may be specifically enumerated, negotiated, or represent the state-of-the-art.

   The published tolerances reflect either the degree of workmanship attainable or a limitation required due to design considerations. Since structural integrity may be involved, the inspector should not make judgments other than whether or not an item is within the allowable tolerances.

3. Tolerances governing fabricated steel members are generally established in the codes, specifications, or manuals listed in Appendix C.

4. The tolerances discussed in this appendix are based on documents effective as of the date of this publication. The inspector must understand that the provisions of the various codes and specification are subject to change at any time. Up to date copies of these documents must be available at all times.

B. Objectives

1. The inspector should have a general knowledge of the specifications, guides and standards referenced in Appendix C. He should know which operations are covered by a written tolerance and those that are implied as state-of-the-art.

2. The inspector should be able to measure the appropriate dimensions or quantities as well as judge workmanship requirements. He should be familiar with the many tools and instruments used to determine whether or not an item is within tolerance.

C. Inspection Checks to be Made

1. Mill Material Tolerances. Dimensional tolerances of mill material should be in accordance with Section 1, Vol. 04.04 of ASTM A6. The specification includes tables listing permissible variations for Camber, Cross Section of Shapes and Bars, Diameters, Out-of-Square, Flatness, Length, Straightness, Sweep, Thickness, Waviness, Weight and Width. The commonly used tolerances are summarized in Part 1 of the AISC Manual of Steel Construction.[14]

Material as received from the mill may on occasion be out of tolerance. Properties such as camber, out-of-square, straightness, swe etc., may be corrected in the shop using commonly accepted sh practices.

2. Mill Material Quality

a. Quality requirements of mill material should be in accordance with Section 1, Vol. 04.04 of ASTM A6, Article 9. The specification states that "the material shall be free from injurious defects and shall have a workmanlike finish." Interpretation of this requirement tends to become very subjective and ill-defined.

The AISC position, as stated in Quality Criteria and Inspection Standards[9] (QCIS), Article II-B, reads, "The surface reconditioning limitations of ASTM A6 are intended to apply only to operations performed at the rolling mill; they are not intended to apply to surface reconditioning or repairs in the shop of the steel fabricator, where qualified welders and special equipment are available and where surface variations or defects exceeding those permitted in ASTM A6 can be satisfactorily repaired.

Guide Specifications for Fracture Critical Nonredundant Steel Bridges Members[7] prohibits repair of material defects by welding at the producing mill for fracture critical materia

b. Surface condition repair of mill material may be performed in the fabricating shop, QCIS states in Paragraph II-B4: "In general, surface imperfections discovered after blast cleaning, which are determined to be non-detrimental to the strength of the member, need not be repaired or removed for cosmetic reasons, unless specifically stated within the contract documents."

c. Edge discontinuity tolerances are covered in a number of governing documents. The AISC position is clearly stated in QCIS Article II-C, summarized as follows:

1) For cut edges prepared for welding, the criteria of AWS D1.1 Art. 3.2.3 should be followed.

2) For cut or rolled edges not prepared for welding on dynamically loaded members, AWS D1.1 Art. 3.2.3 is applicable except as modified as follows:

With the approval of the purchaser, discontinuities need not be explored to a depth greater than 1". When the discontinuity depth exceeds 1", the discontinuity shall be gouged to a depth of 1" and repair welded.

For discontinuities over 1" in length, with depth mc than 1/8" and less than 1", the discontinuity shall removed and repaired, but no single repair shall exceed 20% of the length of the edge being repaired.

3) For cut or rolled edges not prepared for welding on statically loaded members, the following should be considered:

With the approval of the owner, increased tolerances over AWS may be acceptable.

More restrictive tolerances may be in order where loading is perpendicular to the through thickness direction.

4) Internal discontinuities for certain severe service applications may be critical. Tolerance criteria for such conditions should be established prior to commencing fabricating operations.

d. Edge discontinuities for bridge members will generally be governed by Article 3.2.3 of the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges.[5] This article specifically enumerates the acceptable tolerances and the appropriate repair requirements.

e. Edge discontinuities in the fracture critical elements of bridge members are also governed by the AASHTO steel welding specifications[5], except that "there shall be no visible lamellar discontinuities in the boundaries of tension groove welds." Repair of edge defects in fracture critical members are generally governed by the AASHTO Guide Specifications for Fracture Nonredundant Steel Bridge Members.[7]

3. Fabricating Tolerances

a. Thermal-cut Edges

1) Lamellar Type Discontinuities - See the provisions of AWS D1.1[2] par. 3.2.3.

2) Notches and Gouges:

AISC (QCIS)[9]

Edge condition, tension element: ANSI 1000 as defined in Surface Texture[25] ANSI B46.1 - 1978

Edge condition, other than tension element machine guided -ANSI 2000 as defined in Surface Texture[25] ANSI B46.1 - 1978

Edge condition, other than tension element hand guided: Max. 1/16"

Occasional gouges equal to or less than 3/16" deep: no repair. Gouges greater than 3/16" and all notches to be repaired.

AASHTO[5]

Edge condition of material to 4" thick: ANSI 1000, except end of members not subject to calculated stress - ANSI 2000

Edge condition of material 4" to 8" thick: ANSI 2000

Occasional gouges may be ground, if net area after grinding is equal to or greater than 98% of the nominal area.

Welding of any gouges requires approval of the engineer.

Laminar type defects revealed by burning: Evaluate per Art. 3.2.3 of AASHTO[5].

b. Assembly prior to welding

1) Joints to be fillet welded (AISC-AWS - AASHTO) : (Refs. 2, 5 or 9)

Material less than 3" thick: 3/16" max. gap

Material 3" thick or greater: 5/16" max. gap, provided "melting-through" is prevented. Fillet size to be increased if gap exceeds 1/16".

For lap joints, the separation between faying surfaces shall not exceed 1/16".

2) Butt welded joints: (AISC-AWS-AASHTO) : (Refs. 2, 5 or 9)

For butt joints using backing bars, the separation of faying surfaces shall not exceed 1/16".

Other tolerances pertaining to root face, root opening, groove angle, etc. to be in accordance with Arts. 3.3.2, 3.3.3 and 3.3.4 of AWS D1.1.

c. Dimensional tolerances: These tolerances apply to structural members, whether of a single rolled shape or built-up.

1) Length (AISC): Ref. 9

Two ends finished for contact bearing: 1/32"

Ends not finished which frame to other steel parts of the structure: 1/16" for members 30 feet or less and 1/8" for members over 30 feet.

2) Straightness

AISC[14]

All members except as noted below: 1/8" x total length in feet ÷ 10.

Sweep in members with a flange width less than 6": 1/8" x total length in feet ÷ 5.

Sections used as columns that are 8" deep sections 31 #/ft and heavier, 10" deep sections 49 #/ft and heavier, 12" deep sections 65 #/ft and heavier, 14" deep sections 90 #/ft and heavier

Lengths less than 30':
1/8" x total length in feet ÷ 10

Lengths of 30' to 45':
3/8"

Lengths over 45':
3/8" + 1/8" x (total length in feet-45) ÷ 10

Compression members: 1/1000 x axial length between points which are to be laterally supported.

AWS[2]

Welded columns and primary truss members:

Lengths less than 30':
1/8" x total length in feet ÷ 10

Lengths of 30' to 45':
3/8"

Lengths over 45':
3/8" + 1/8" x (total length in feet -45) ÷ 10

Welded beams or girders where there is no specified camber or sweep: 1/8" x total length in feet ÷ 10.

Welded beams or girders with specified camber: The permissible variation from the specified camber is the greatest of (a), (b), or (c):

(a) -0, +1/4"
(b) -0, +1/4" x test length in feet ÷ 10 but not to exceed 3/4".
(c) -0, + 0/8 x no. of feet from nearest end ÷ 10
(d) Members with specified camber whose top flange is encased in concrete without a designed concrete haunch ± total length in feet ÷ 160, but not more than 1/4".

Horizontally curved welded beams or girders: The permissible sweep deviation is ± 1/8" x total lenş in feet ÷ 10.

3) Flatness of Webs -

AWS[2]

Static Loading: Tolerances of AWS D1.1 Art. 8.13 apply, except for web thickness less than 5/16" tolerance need not be less than 1/2".

Dynamic loading: Tolerances of AWS D1.1 Art. 8.13 are applicable.

Tolerances of AWS D1.1 Art. 9.23 are applicable to flatness of bridge girder webs.

4) Combined warpage and tilt of flanges of welded beams or girders (AWS[2] and AISC[9])-

1/100 of total flange width or 1/4", whichever is greater. This dimension is measured as the offset of the toe of the flange from a line normal to the plane of the web through the intersection of the center line of the web with the outside surface of the flange.

5) Other specified dimensional tolerances (AWS[2] and AISC[9]):

Tolerances of girder depth, contact area of bearing stiffeners, fit of intermediate stiffeners. straightness of intermediate stiffeners and straightness and location of bearing stiffeners are per AWS Articles 3.5.1.8 through 3.5.1.12 inclusive.

6) Twist of box girders or columns (AISC[9]):

Twist tolerances for box members must be clearly stipulated and mutually agreed upon prior to fabrication.

7) Unspecified dimensional tolerances:

Dimensional tolerances not covered by AWS or AISC shall be individually determined and mutually agreed upon by the contractor or fabricator and the owner with proper regard for erection requirements.

d. Weld Tolerances

Fillet weld profile to be in accordance with AWS[2] Article 3.6.1. Also note that the QCIS[9] Articles H and I modi the AWS requirements.

Groove weld profile to be in accordance with AWS[2] Articles 3.6.2 and 3.6.3. Also note that the AASHTO steel welding specifications[5] in Article 3.6, modify the AWS requirements.

Many other tolerances are included in the AWS welding code. These tolerances may occasionally be modified by the AASHTO steel welding specifications and QCIS[9].

N.D.T. tolerances are included in the AWS Welding Code. They may be modified by the AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges[5] and the AASHTO Guide Specifications for Fracture Critical Nonredundant Steel Bridge Members.[7]

e. Other Tolerances

It is not the intent of this Appendix to enumerate all tolerances governing the fabrication of structural steel. Additional tolerances will be found in the specifications, codes and manuals listed in Appendix C.

E. Special Problems

1. The inspector should recognize that the pertinent specifications do not accurately reflect a workable tolerance for every situation. A 100 foot long girder will have a sweep tolerance of 1 1/4". This value might be unduly restrictive for 3/4" x 12" flanges and unduly lenient for 3" x 28" flanges. Decisions as to the applicability of certain tolerances under extreme conditions should be made only by those persons who have that authority.

2. Conditions will arise where certain members may be out of tolerance. Correction of out of tolerance elements may be inordinately expensive or, in some cases, do more harm than good. The inspector should be alert for such situations and refer them to those people who have the authority to make the proper judgment as to a recommended solution.

F. Instructional Aids

1. Library of specifications, codes and manuals (see Appendix C, References)
2. This Training Guide

APPENDIX B

LOADING FOR SHIPPING

A. Introduction

Proper procedures for packaging and loading structural steel and accessories for shipment are essential to:

1. Avoid loss in transit.

2. Avoid damage and permanent deformation in transit and the consequent costs associated with delays in progress of erection as well as repairs made by shop or field forces.

3. Avoid shortages and sequencing errors which delay and add cost to the work.

Other than Sections 6(d) and 6(e) of the AISC Code of Standard Practice,[8] no specifications have been formulated to cover this function. However, guidelines have been prepared for loading railroad cars in the form of Rules Covering the Loading of Commodities on Open-Top Cars,[24] published by the American Association of Railroads.

B. Objective

By adopting certain basic rules of good shop practice and careful inspection it is hoped to prevent damage to and loss of materials, and to avoid costly errors in sequencing of these materials to the jobsite.

C. Inspection Checks to be Made

Some of the important areas for review are:

1. Blocking or supporting materials loaded on trucks, railroad cars or barges to permit convenient unloading by the erector, thereby minimizing or eliminating the potential for damage during unloading, including scuffing and abrasion of shop coatings.

2. Blocking or supporting materials as in paragraph 1, above, to prevent deformation of members during transit or inducing stresses which might cause welds to crack or fail.

3. Proper application of handling lugs and lifting aids for large members such as girders, trusses, vessels, plate work, etc., where the usual handling equipment such as chains or chokers are impractical.

4. Blocking of railroad cars to minimize shifting of loads resulting from rough car handling techniques such as "humping."

5. Computation of load center-of-gravity to insure load stability.

6. Checking of overall width and height of loads to be certain these are within restrictions imposed by routing clearances.

7. Checking of marks, tags, etc., to assure proper identification at the destination.

8. Checking shipping papers to assure conformance with customer requirements and necessary documentation to provide field unloading tally.

9. Assure that all bundles, boxes, crates, cartons, drums or other containers are so placed and secured that they will not be damaged or lost in transit.

10. Assure that the contents of containers as in paragraph 1, above, are properly identified and enumerated.

D. Special Problems

Due recognition should be given to special problems which may occur in loading for shipment. Examples of these are:

1. Use of special devices (such as lubricated swivel bolsters) for shipment of long assemblies requiring the employment of two or more railroad cars.

2. Placement of material on truck bodies to prevent imposing axle loads exceeding the appropriate highway limitations.

E. Training Recommendations

It is recommended that those to be given the responsibility of loading trucks, cars or barges for shipment undergo apprentice training for an appropriate period, using such instructional aids as may be prepared by the particular facility for use by its personnel.

F. Instructional Aids

1. Library of specifications, codes and manuals (see Appendix C, References)
2. This Training Guide

APPENDIX C

REFERENCES

1. AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, American Institute of Steel Construction, 1978.

2. AWS Structural Welding Code-Steel D1.1, American Welding Society, 1985.

3. RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections, 1980.

4. Standard Specification for Highway Bridges, American Association of State Highway and Transportation Officials, 1983.

5. Standard Specifications for Welding of Structural Steel Highway Bridges, American Association of State Highway and Transportation Officials, 1981.

6. Manual for Railway Engineering, American Railway Engineering Association.

7. Guide Specifications for Fracture Critical Nonredundant Steel Bridge Members, American Association of State Highway and Transportation Officials, 1978

8. AISC Code of Standard Practice, American Institute of Steel Construction. 1976.

9. AISC Quality Criteria and Inspection Standards, Second Edition, American Institute of Steel Construction, 1980.

10. AISC A Guide to the Shop Painting of Structural Steel, American Institute of Steel Construction, 1972.

11. SSPC Steel Structures Painting Manuals, Steel Structures Painting Council, Vol. 1, Good Painting Practice, 1982; Vol. 2 Systems & Specifications, 1985.

12. ASNT Recommended Practice No. SNT-TC-1A, American Society for Nondestructive Testing, 1980

13. Annual ASTM Standards, American Society for Testing and Materials, 1985.

14. Manual of Steel Construction, 8th Edition, American Institute of Steel Construction, 1980.

15. AWS C4.1 Surface Roughness Guide for Oxygen Cutting, American Welding Society.

16. Assuring Weld Quality, J. E. Hinkel, Lincoln Electric Company, Pamphlet G410.

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

ASPECTOS FUNDAMENTALES DEL EMPLEO DE GRÚAS PARA SU UTILIZACIÓN EN EL MONTAJE DE ESTRUCTURAS

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

1.1 BASIC TYPES AND CONFIGURATIONS

The evolution of the mobile crane has led to many types and designs to satisfy both the general as well as the specific needs of construction and industrial operations. This manual is concerned with mobile cranes used for construction purposes as well as industrial applications.

The basic operational characteristics of all mobile cranes are essentially the same. They include:

- Adjustable boom lengths
- Adjustable boom angles
- Ability to lift and lower loads
- Ability to swing loads
- Ability to travel about the jobsite under their own power.

Within the broad category of mobile cranes there have evolved the following basic types and configurations:

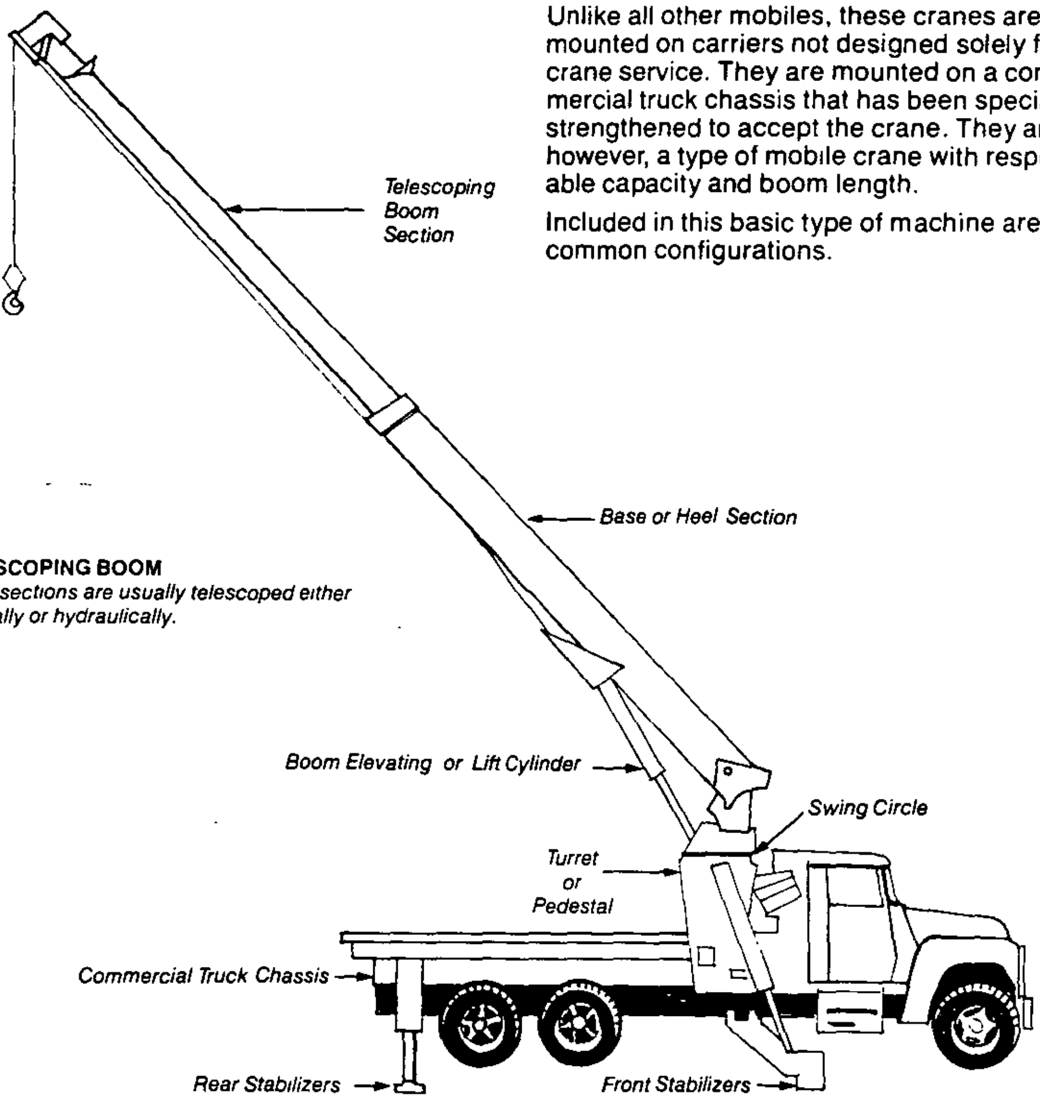
- Boom Trucks
- Industrial Cranes
- Carrier-Mounted Lattice Boom Cranes
- Crawler-Mounted Lattice Boom Cranes
- Carrier-Mounted Telescopic Boom Cranes
- Crawler-Mounted Telescopic Boom Cranes
- Rough Terrain Cranes
- Mobile Tower Cranes
- Heavy Lift Mobile Cranes.

(1)

1.2 BOOM TRUCKS

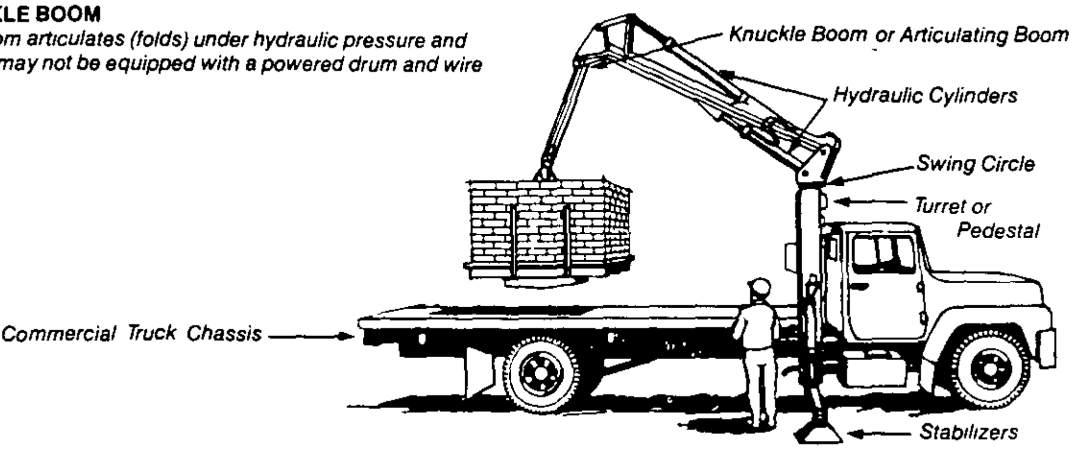
Unlike all other mobiles, these cranes are mounted on carriers not designed solely for crane service. They are mounted on a commercial truck chassis that has been specially strengthened to accept the crane. They are, however, a type of mobile crane with respectable capacity and boom length.

Included in this basic type of machine are two common configurations.



TELESCOPING BOOM
Boom sections are usually telescoped either manually or hydraulically.

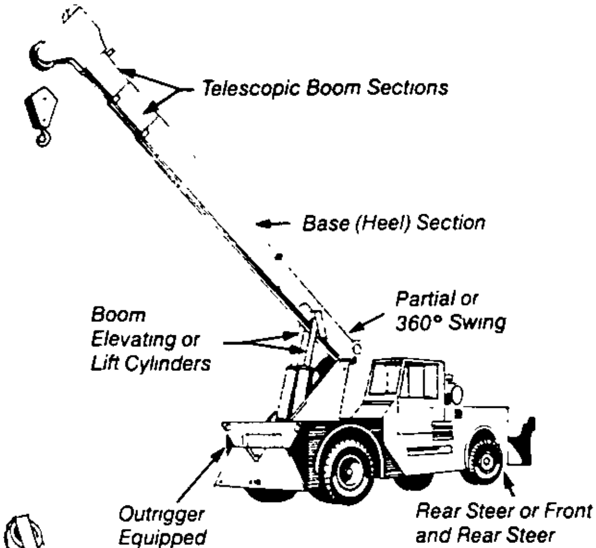
KNUCKLE BOOM
The boom articulates (folds) under hydraulic pressure and may or may not be equipped with a powered drum and wire rope.



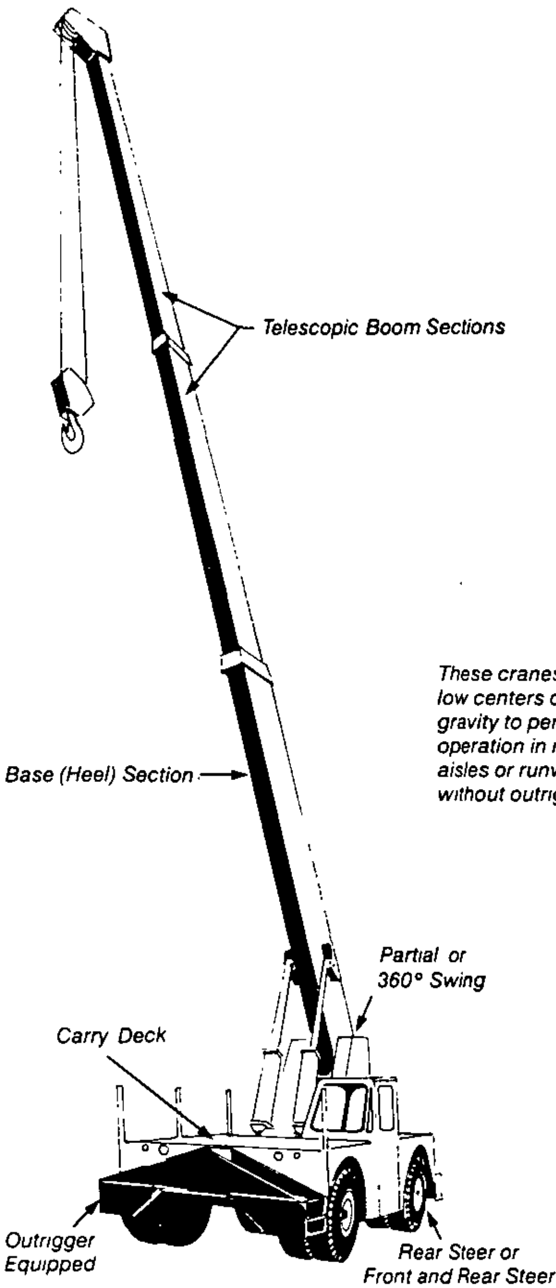
1.3 INDUSTRIAL CRANES

These cranes are primarily intended for operation in industrial locations where working surfaces are significantly better than those found on most construction sites.

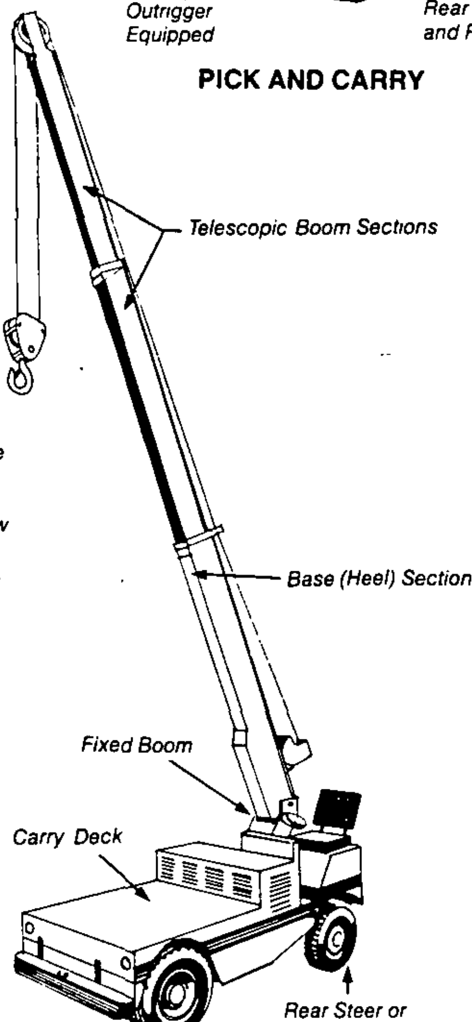
Although these cranes will not be analyzed specifically, their characteristics are basically identical to those of telescopic boom mobiles, which are covered in detail.



PICK AND CARRY



These cranes have low centers of gravity to permit operation in narrow aisles or runways without outriggers.



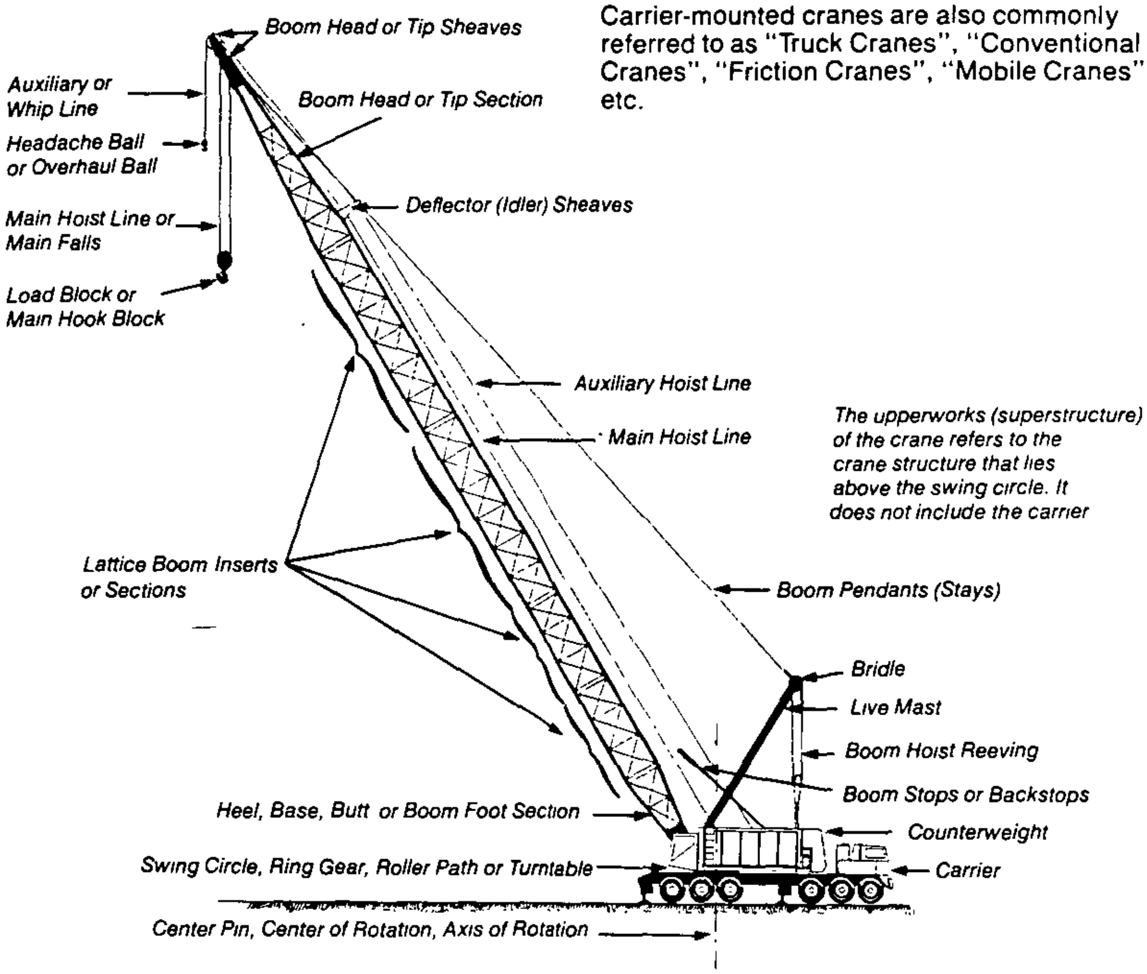
CARRY DECK — ROTATING BOOM

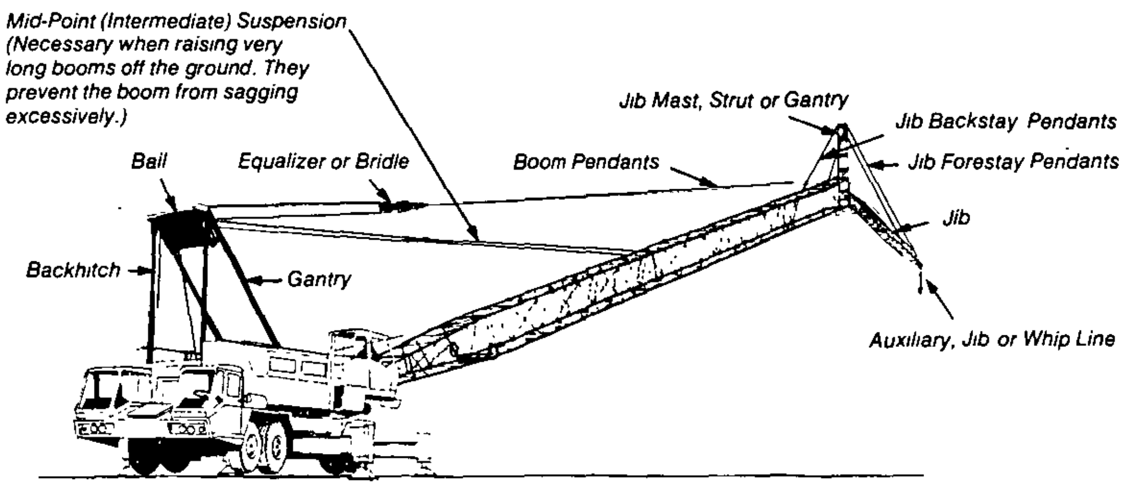
CARRY DECK — FIXED BOOM

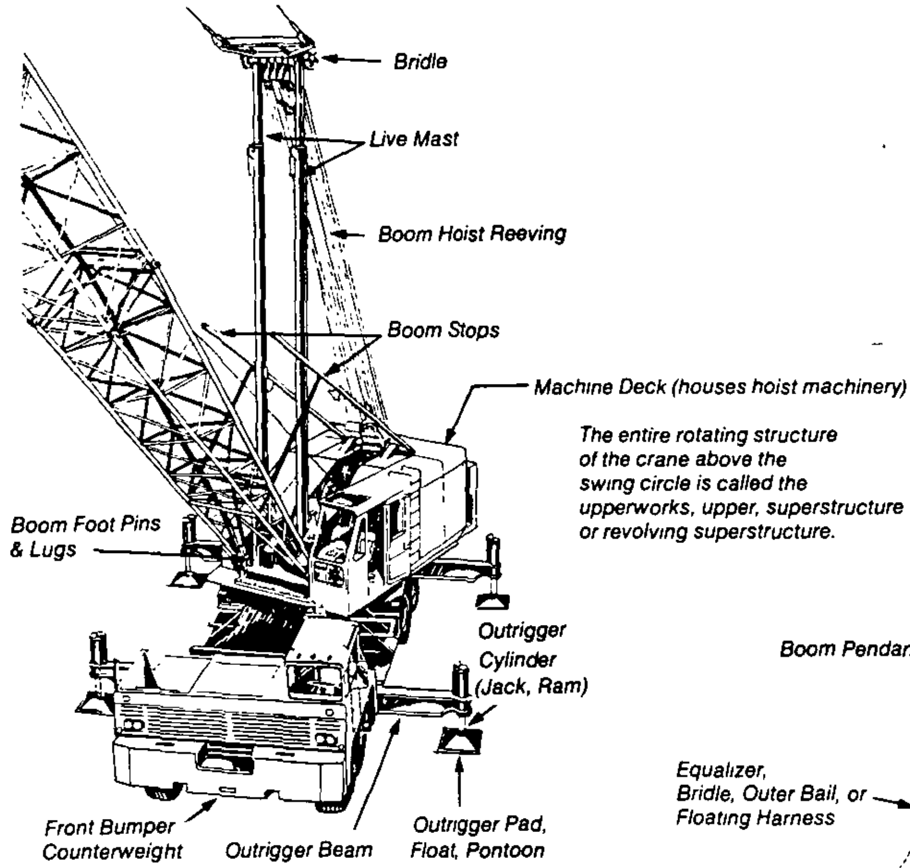
1.4 CARRIER-MOUNTED LATTICE BOOM CRANES

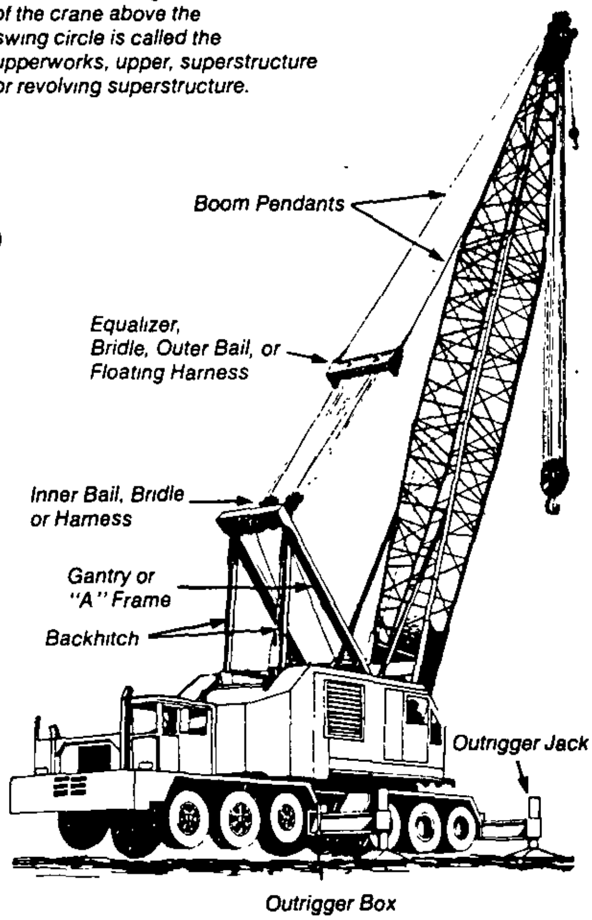
This "truck type" carrier must not be confused with the ordinary commercial truck chassis. It is specially designed for crane service and the heavy loads these cranes are required to withstand.

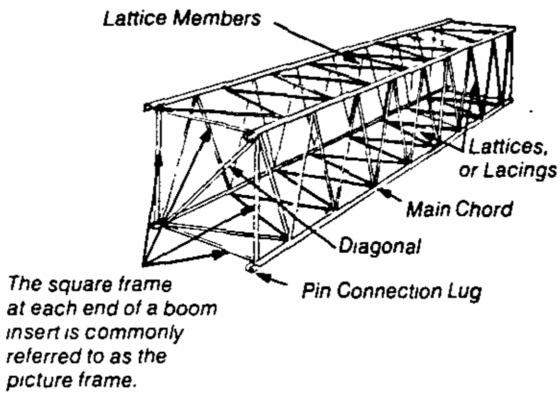
Carrier-mounted cranes are also commonly referred to as "Truck Cranes", "Conventional Cranes", "Friction Cranes", "Mobile Cranes", etc.





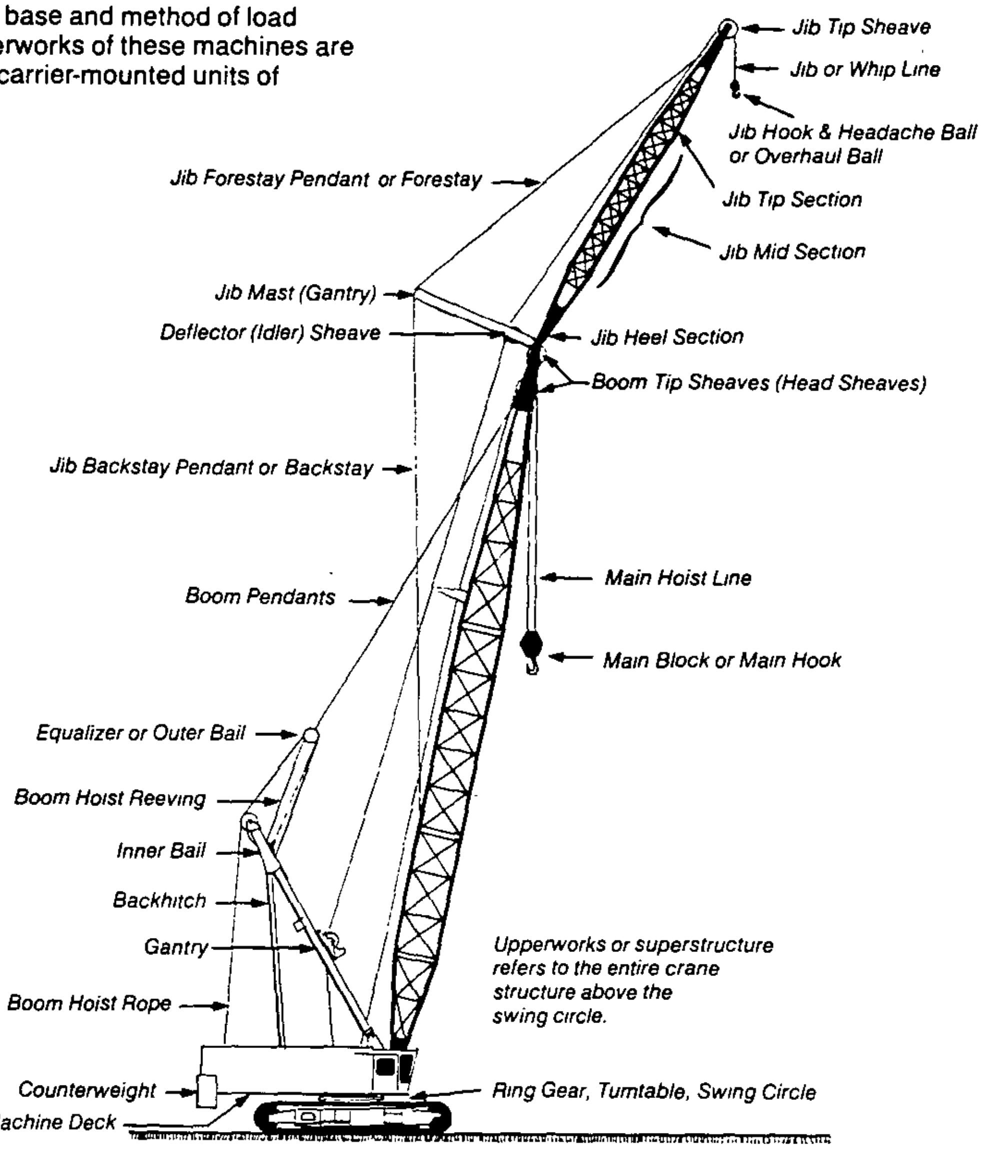

Bridle
Live Mast
Boom Hoist Reeving
Boom Stops
Machine Deck (houses hoist machinery)
The entire rotating structure of the crane above the swing circle is called the upperworks, upper, superstructure or revolving superstructure.
Boom Foot Pins & Lugs
Outrigger Cylinder (Jack, Ram)
Front Bumper Counterweight
Outrigger Beam
Outrigger Pad, Float, Pontoon


Boom Pendants
Equalizer, Bridle, Outer Bail, or Floating Harness
Inner Bail, Bridle or Harness
Gantry or "A" Frame
Backhitch
Outrigger Jack
Outrigger Box


Lattice Members
Lattices, or Lacings
Main Chord
Diagonal
Pin Connection Lug
The square frame at each end of a boom insert is commonly referred to as the picture frame.

1.5 CRAWLER-MOUNTED LATTICE BOOM CRANES

Except for their base and method of load rating, the upperworks of these machines are identical to the carrier-mounted units of Section 1.4.



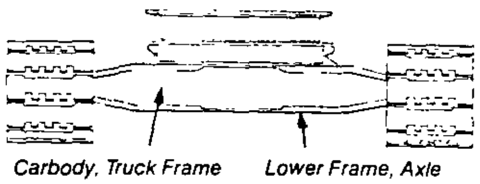
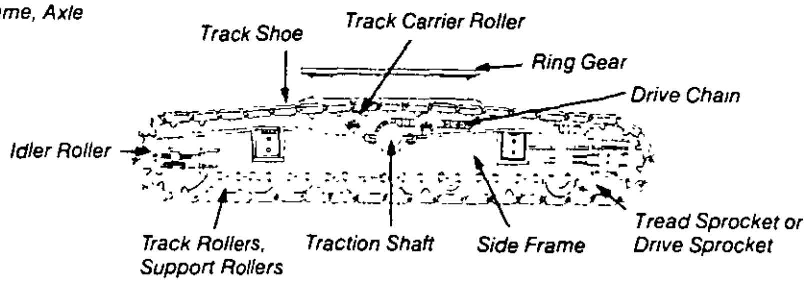
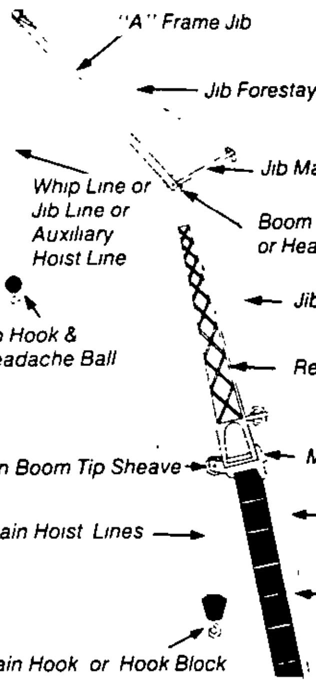


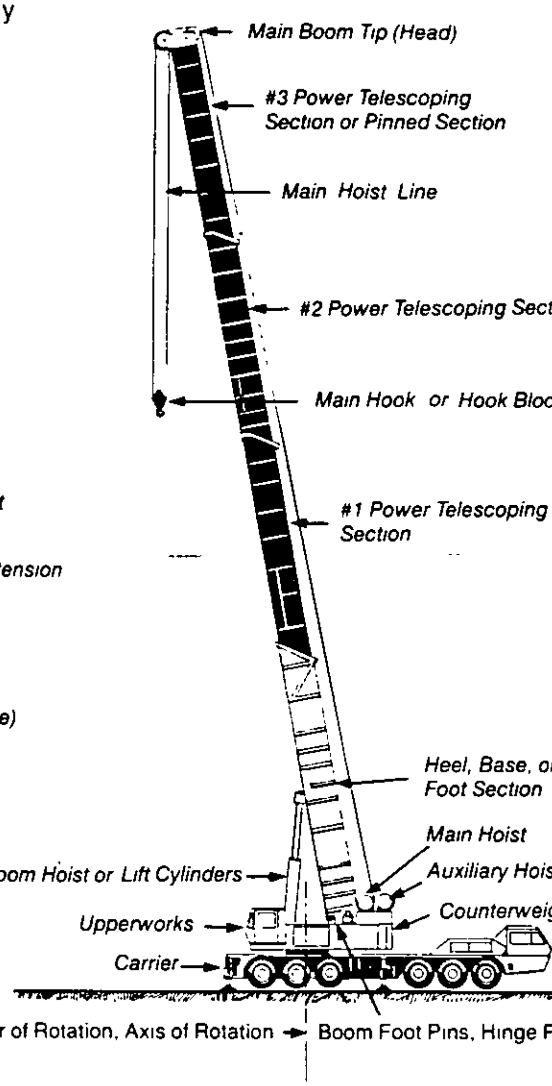
Illustration shows a unit with traction shaft and chain drive but hydrostatic track drive systems are also available.

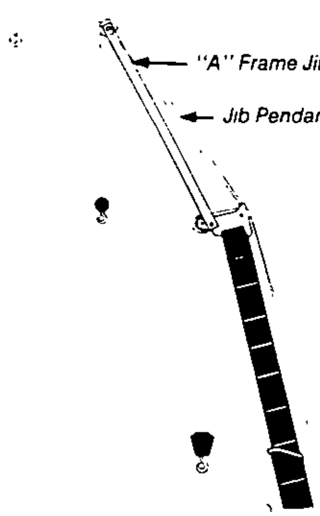


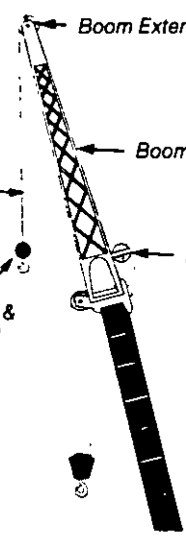
1.6 CARRIER-MOUNTED TELESCOPIC BOOM CRANES

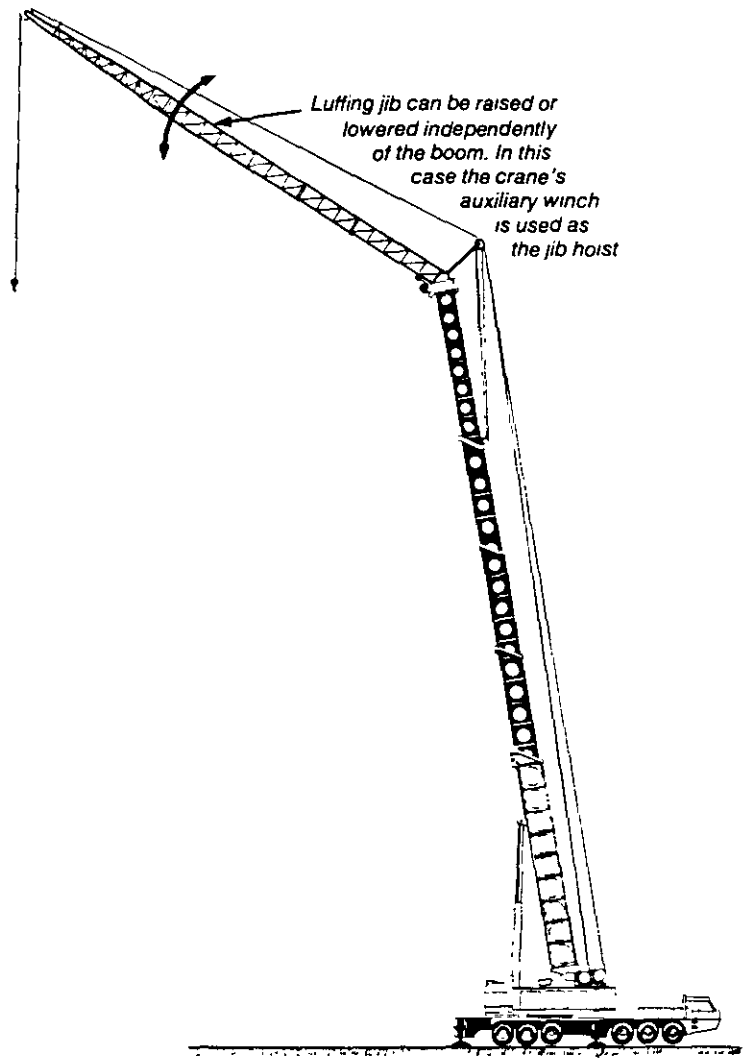
These machines are also mounted on specially designed carriers. They can be equipped with a variety of jibs and boom extensions which can be stowed on or under the heel section of the main boom (see page 15, Sect. 1.8).

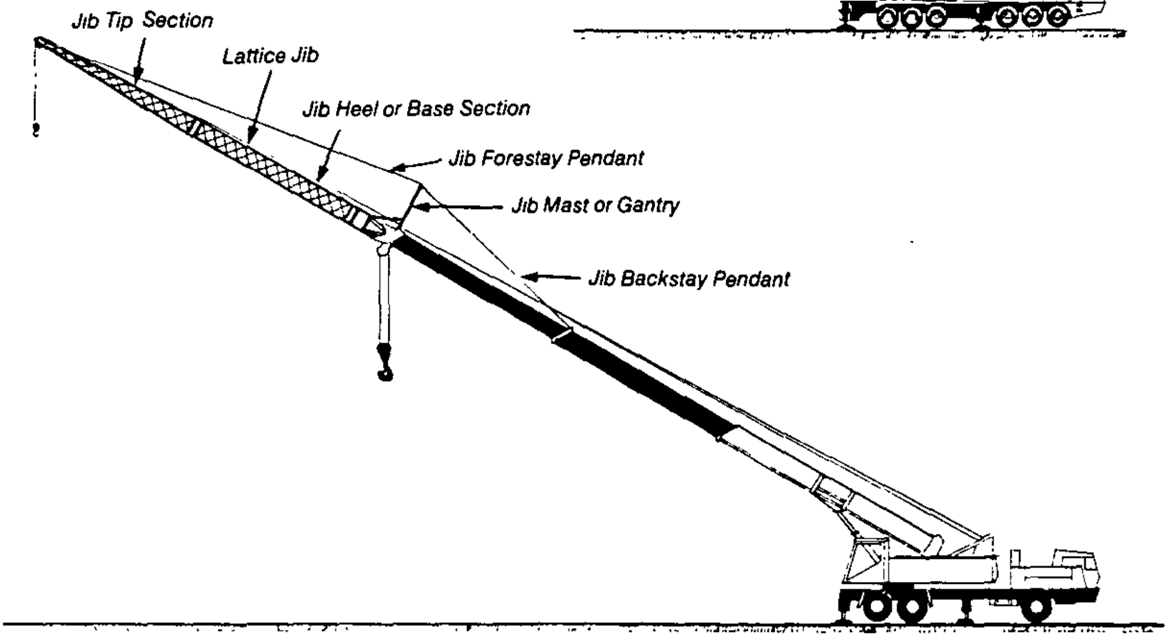






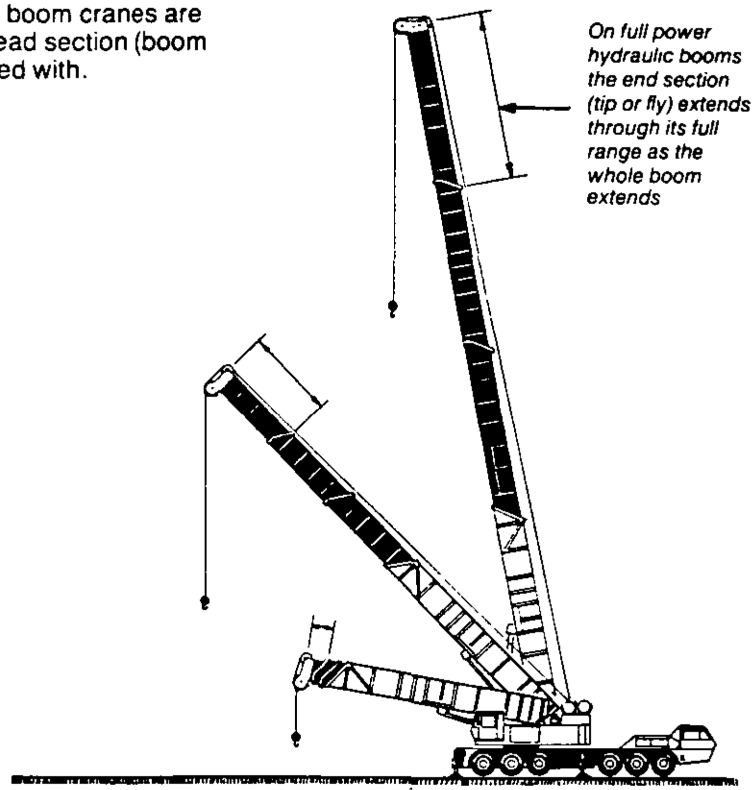



Luffing jib can be raised or lowered independently of the boom. In this case the crane's auxiliary winch is used as the jib hoist

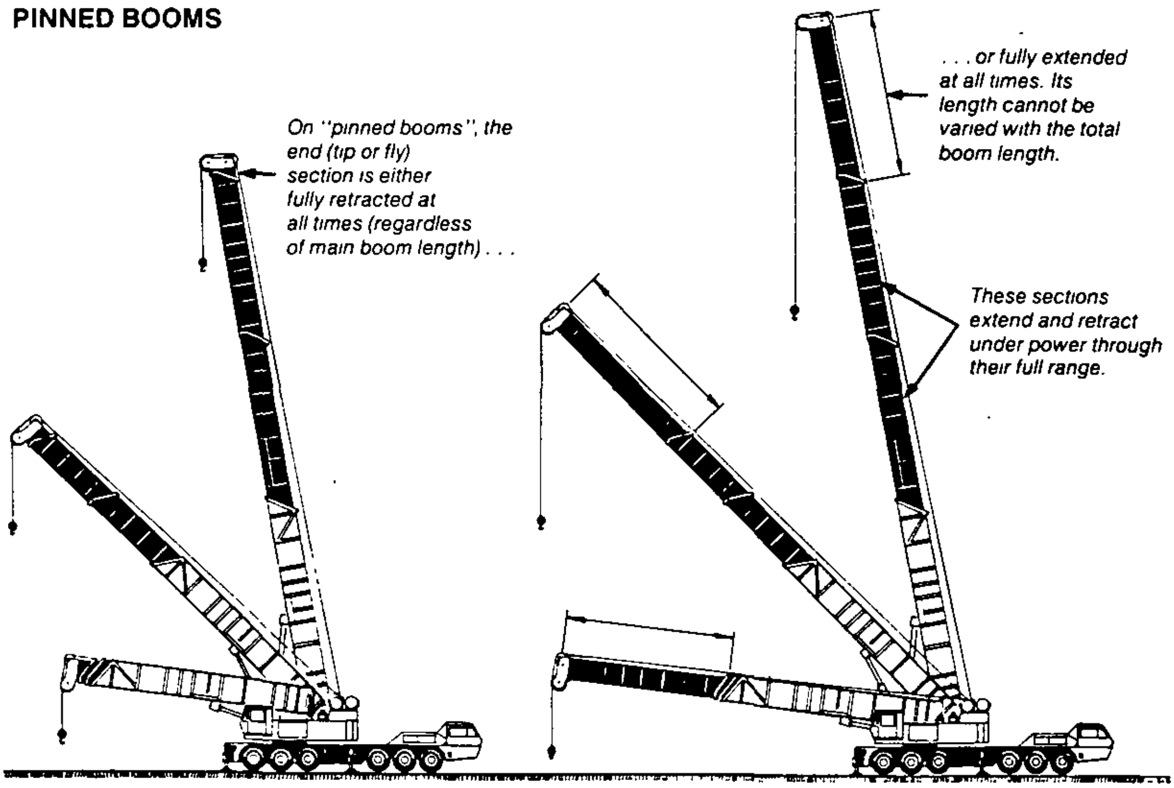

Jib Tip Section
Lattice Jib
Jib Heel or Base Section
Jib Forestay Pendant
Jib Mast or Gantry
Jib Backstay Pendant

Carrier-mounted telescopic boom cranes are subdivided by the type of head section (boom tip section) they are equipped with.

FULL POWER BOOMS

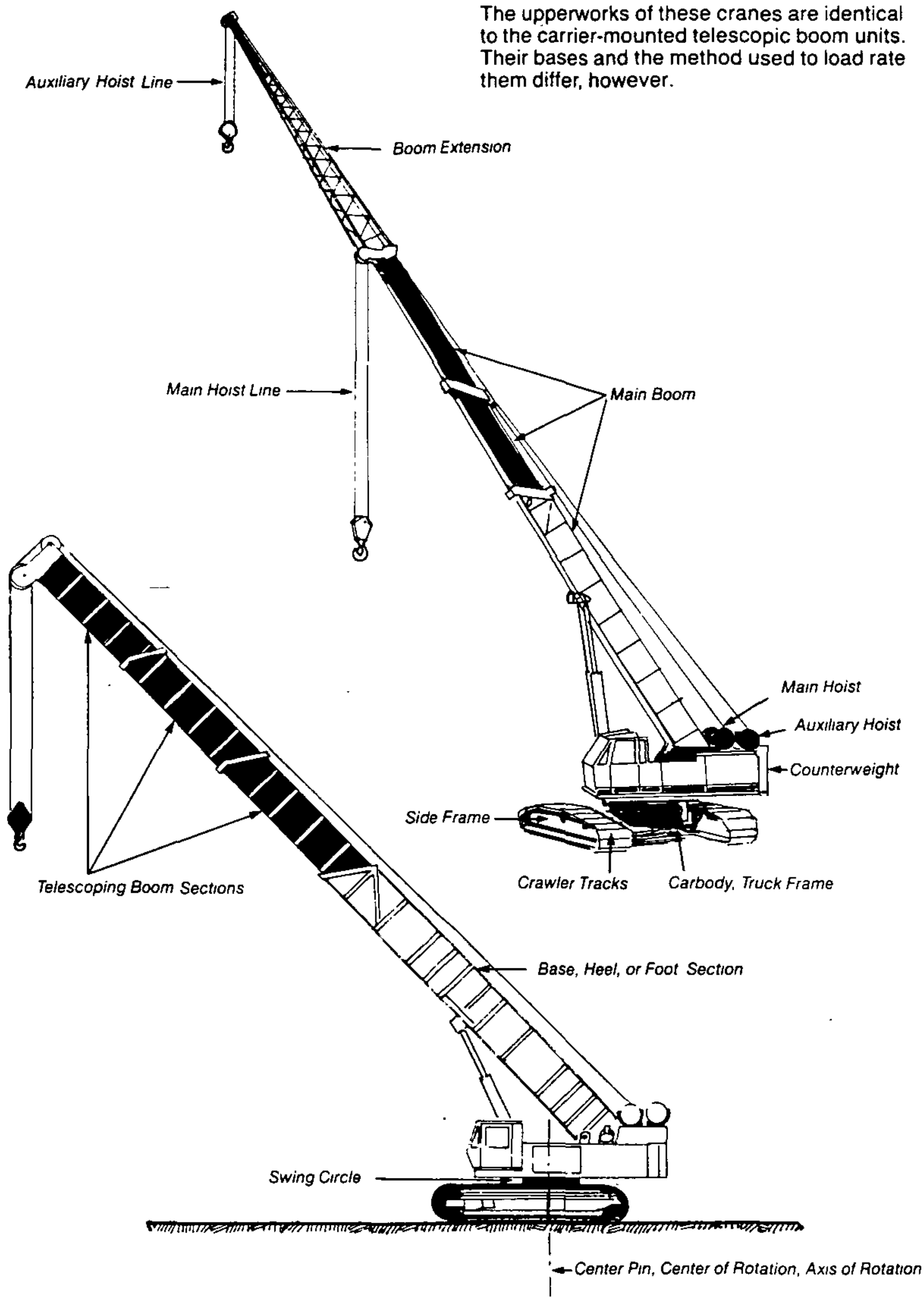


PINNED BOOMS



1.7 CRAWLER-MOUNTED TELESCOPIC BOOM CRANES

The upperworks of these cranes are identical to the carrier-mounted telescopic boom units. Their bases and the method used to load rate them differ, however.

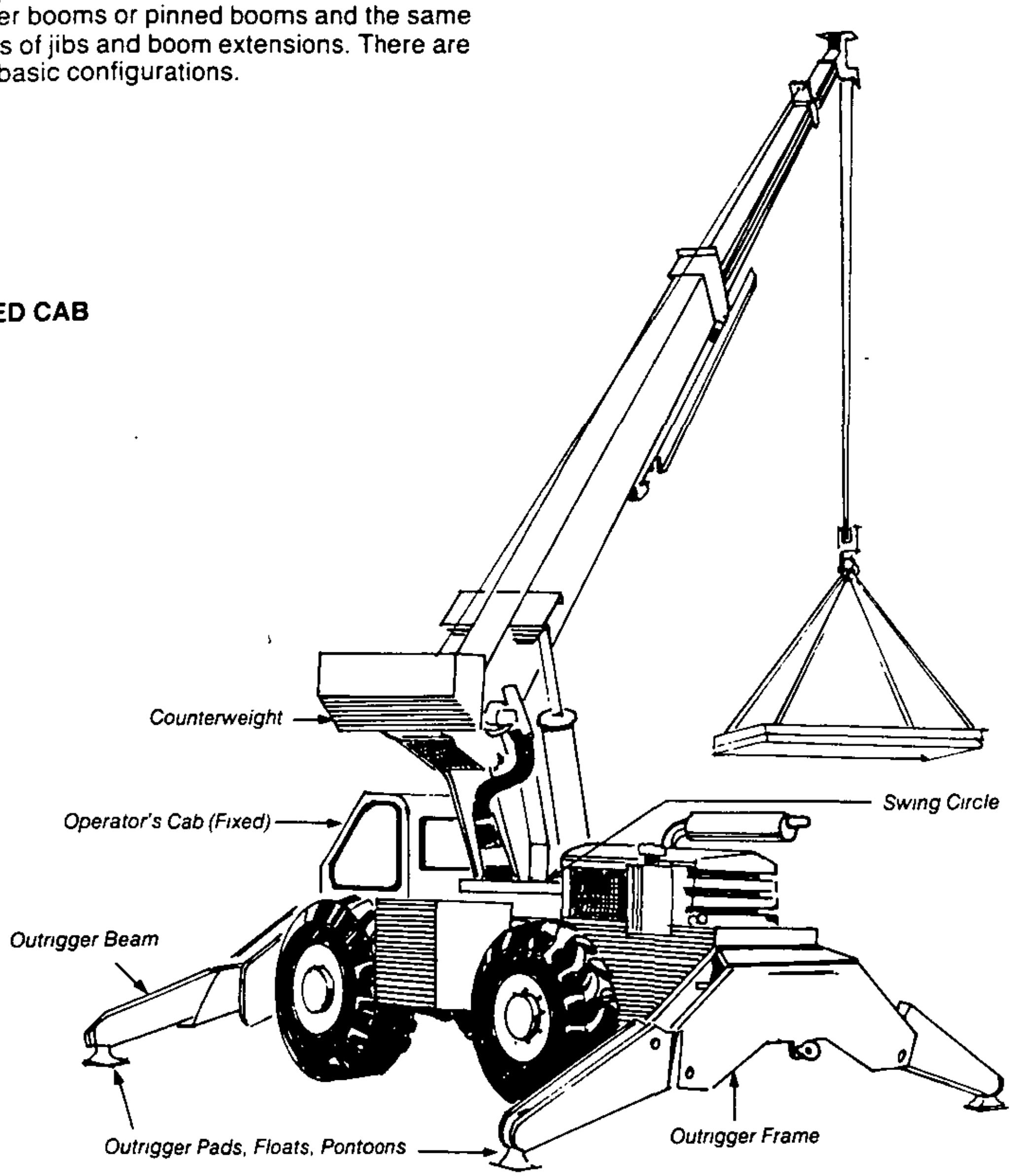


1.8 ROUGH TERRAIN CRANES

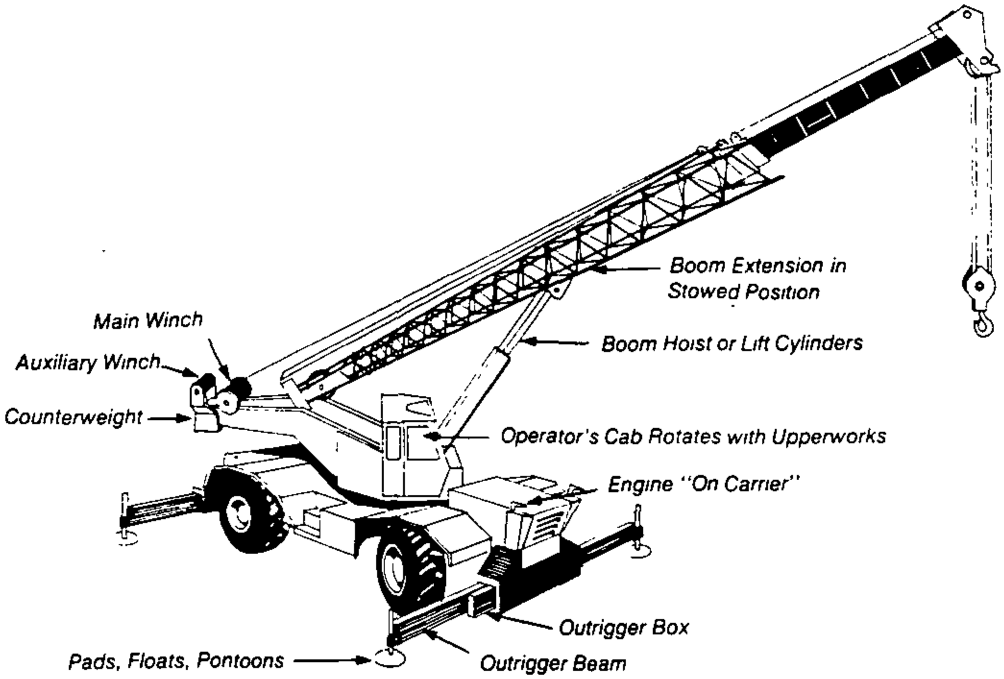
The rough terrain crane's oversized tires facilitate movement across the rough terrain of construction sites and other broken ground. Their short wheel base and crab-steering improve maneuverability. In "pick and carry" operations on rough terrain, however, they are still subject to the same operating restrictions that apply to other cranes.

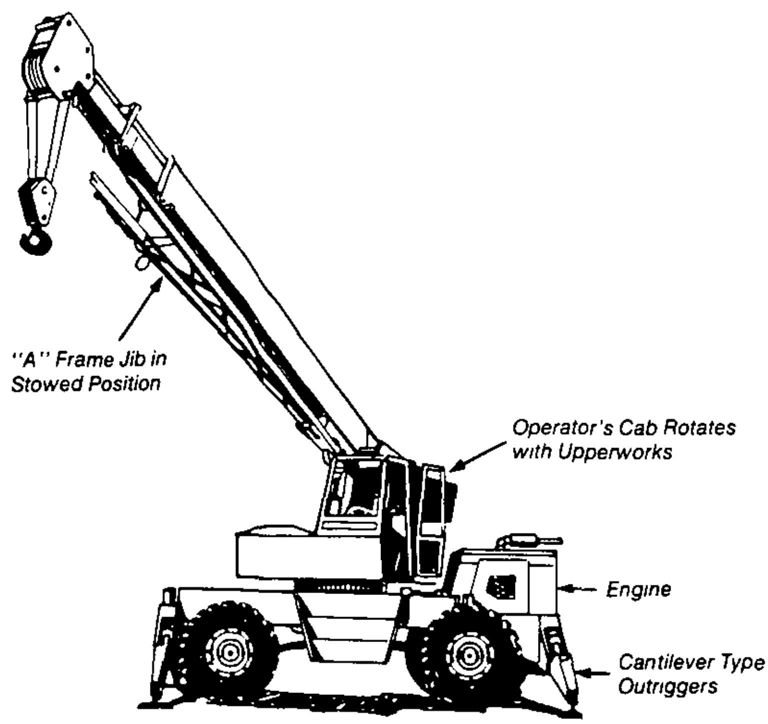
Like carrier-mounted telescopic boom cranes, rough terrain units are available with either full power booms or pinned booms and the same types of jibs and boom extensions. There are two basic configurations.

FIXED CAB

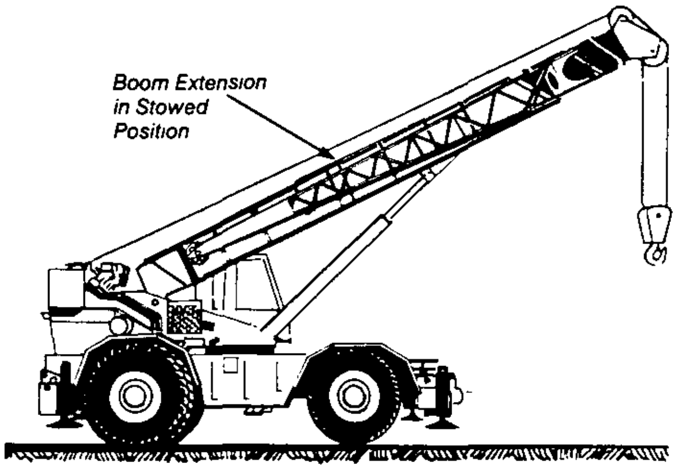


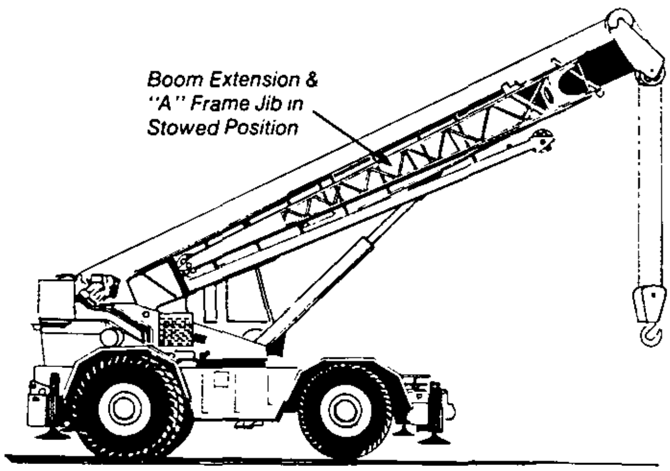
ROTATING CAB

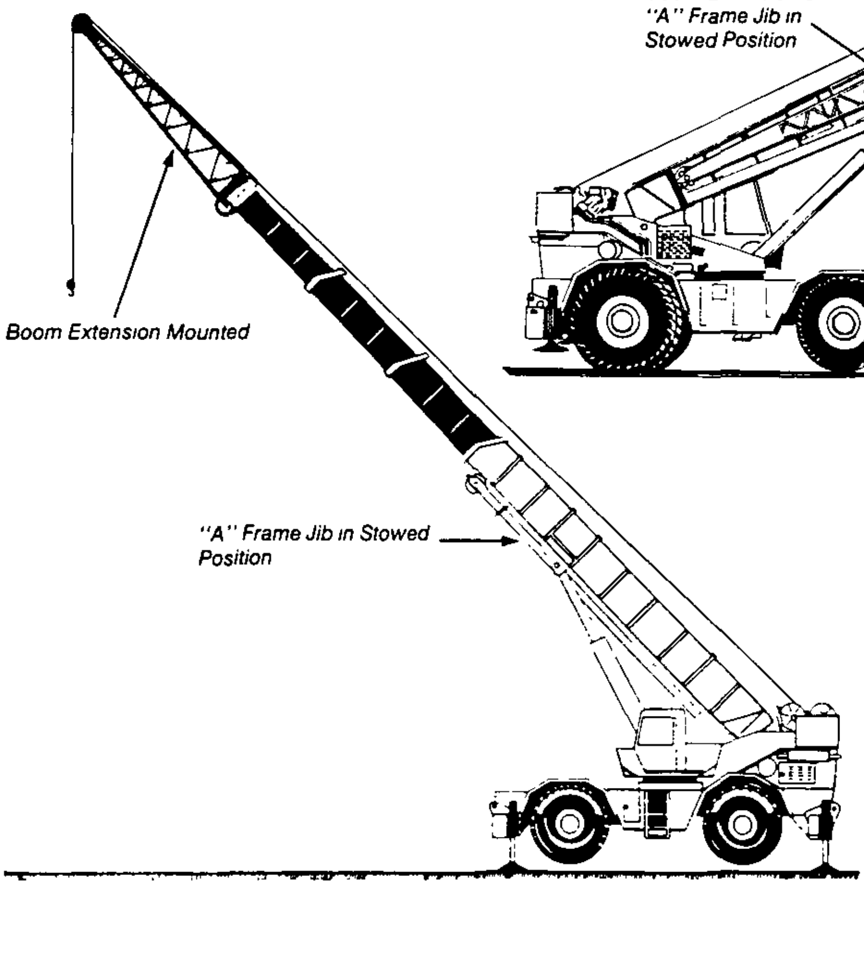




Like the carrier-mounted telescopic boom cranes, rough terrain cranes can be equipped with either full power booms or pinned booms as well as with a variety of jibs and boom extensions which can also be stowed on or under the heel section of the main boom.

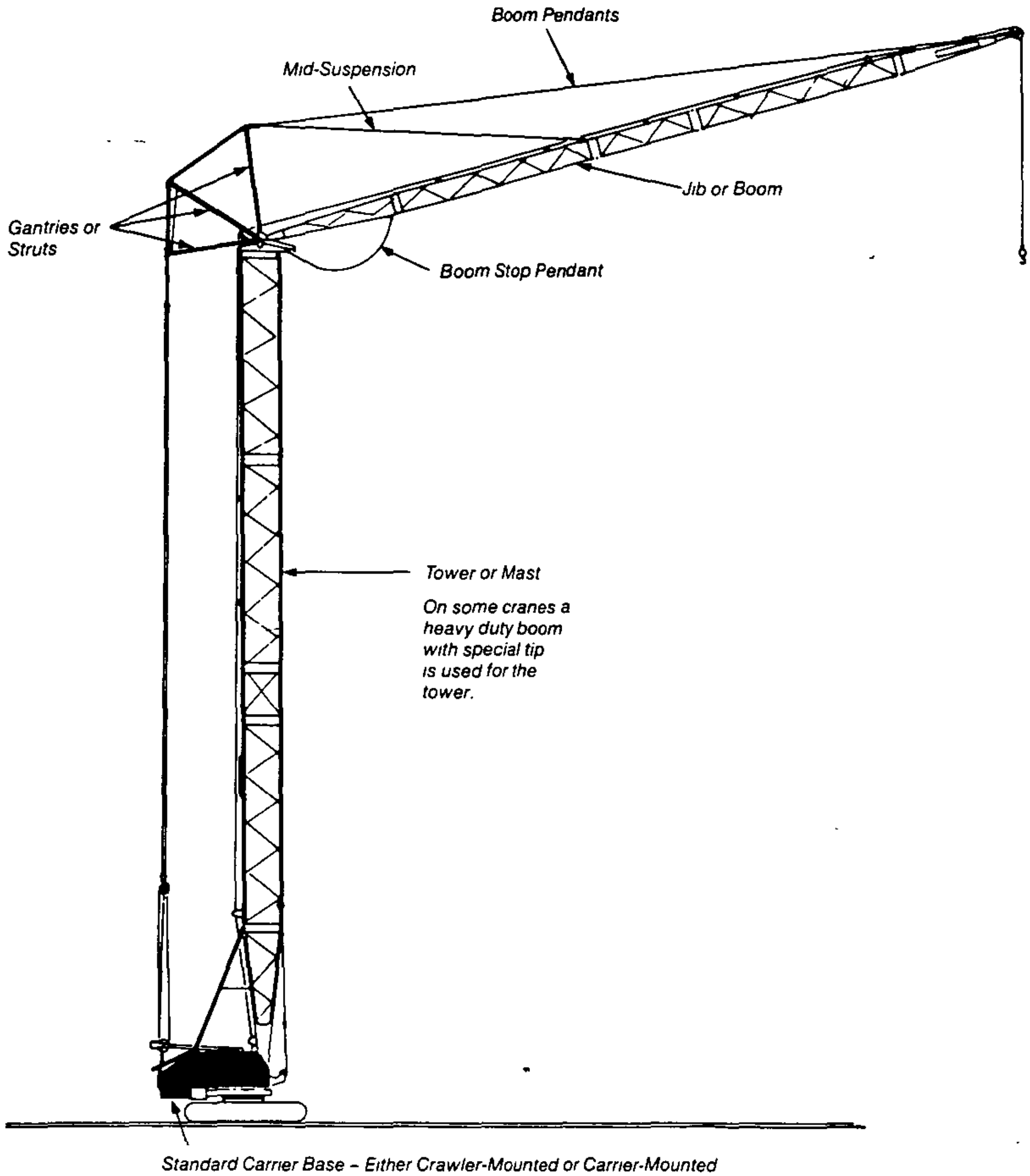






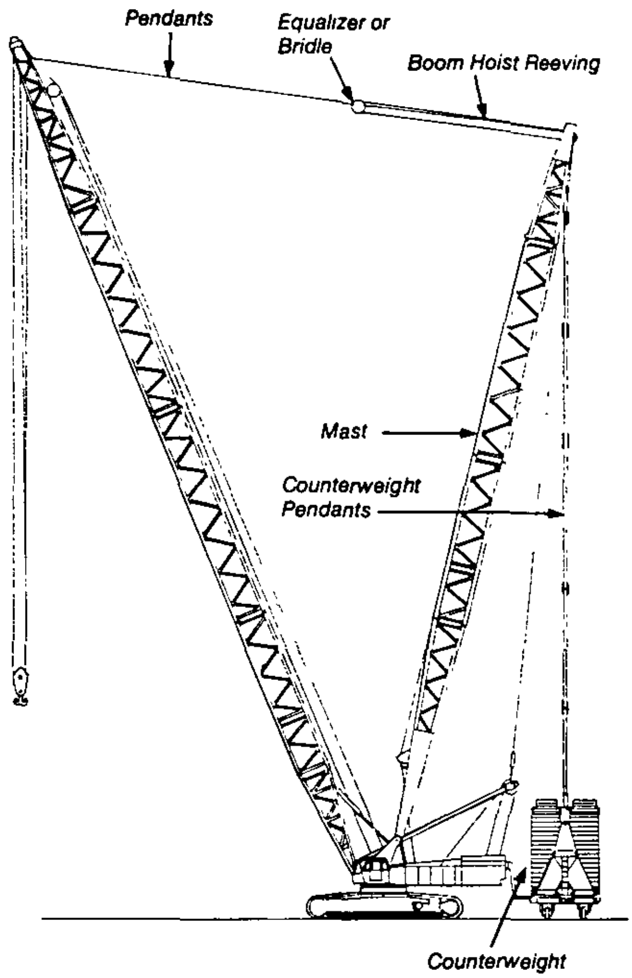
1.9 MOBILE TOWER CRANES

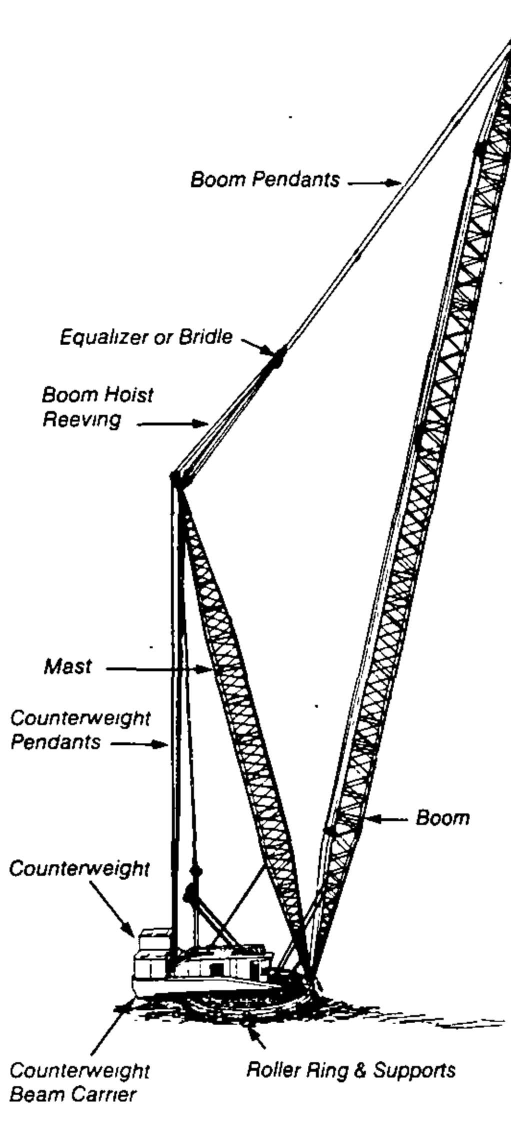
Some manufacturers of carrier-mounted lattice boom cranes offer optional tower attachments for their machines.



1.10 HEAVY LIFT MOBILE CRANES

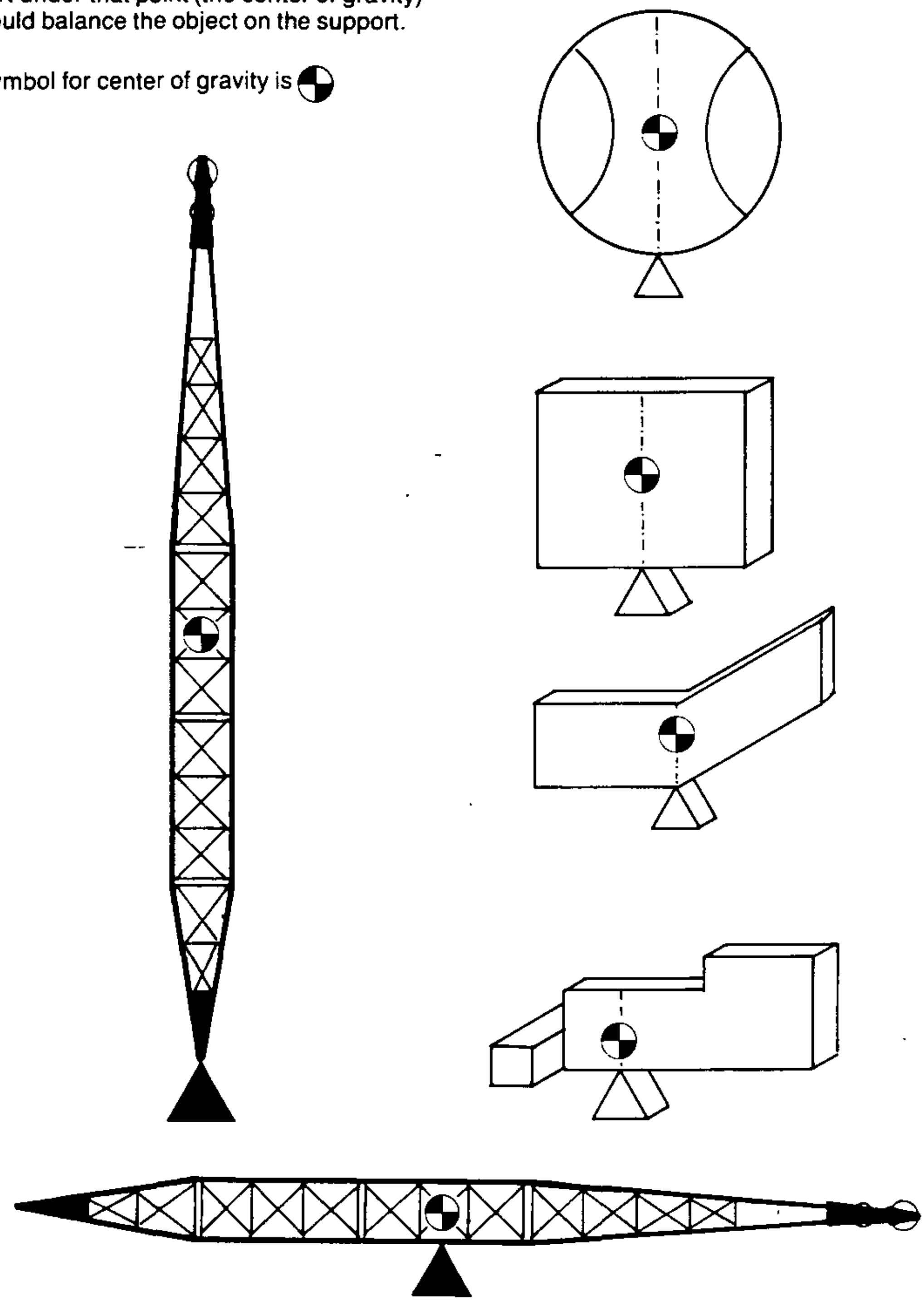
These cranes combine the best features of derricks and lattice boom mobile cranes. Typically they use very large extended counterweights, masts and often roller rings that move the boom's fulcrum and the crane's tipping axis further away from the center of gravity.





The center of gravity of any object is the point in the object where its weight can be assumed to be concentrated or, stated in another way, it is the point in the object around which its weight is evenly distributed. If you could put a support under that point (the center of gravity) you could balance the object on the support.

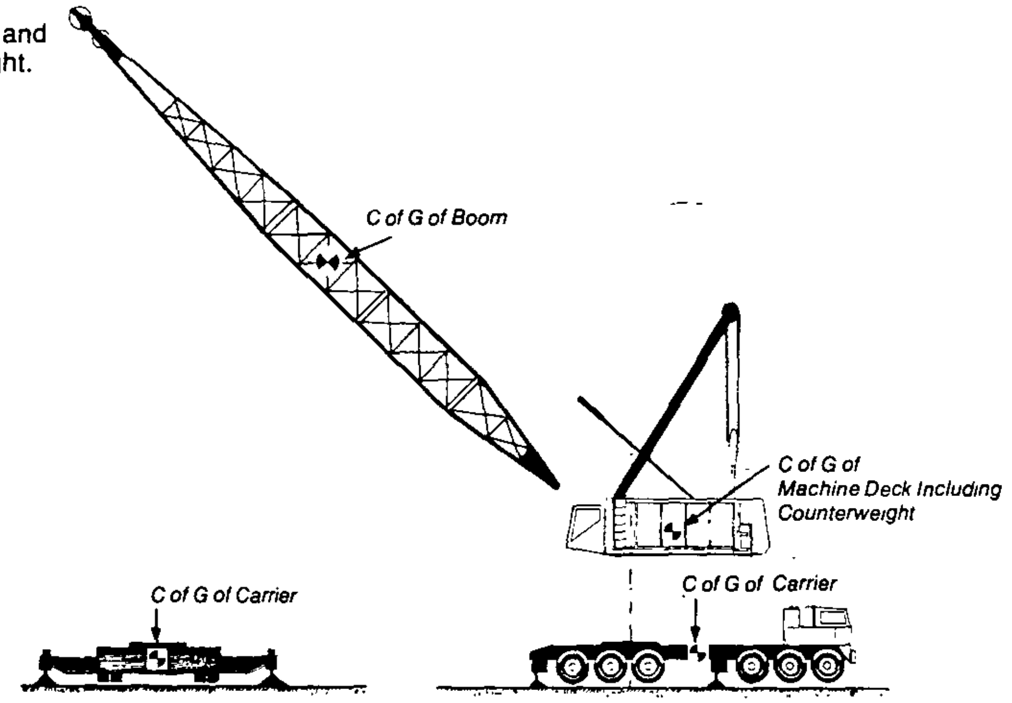
The symbol for center of gravity is



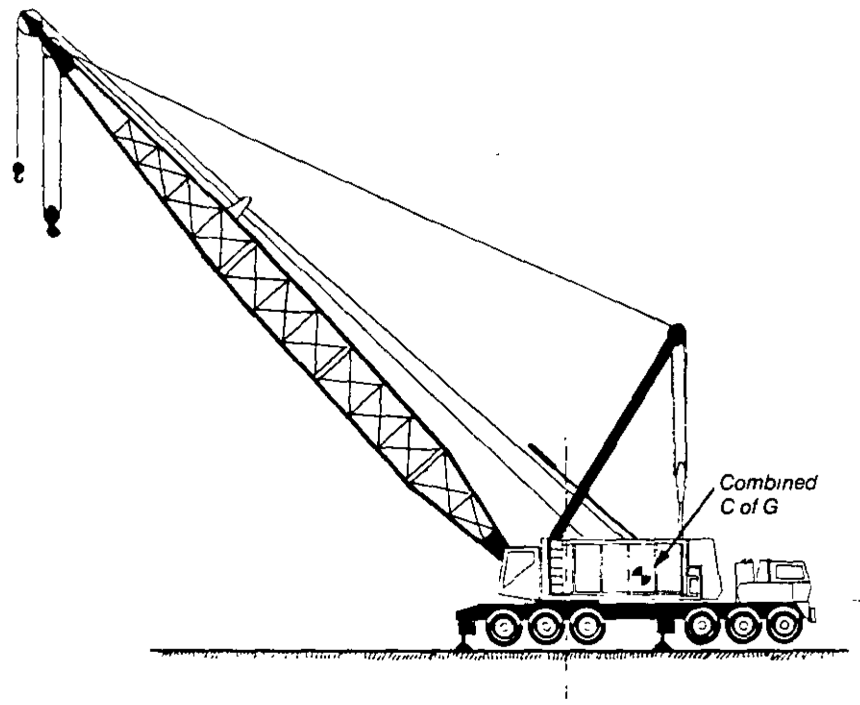
2.2 CRANE'S CENTER OF GRAVITY

The location of the center of gravity of a mobile crane depends on the weight and location of its heaviest components. We need only be concerned with the effect of the

- boom
- carrier
- upperworks and counterweight.

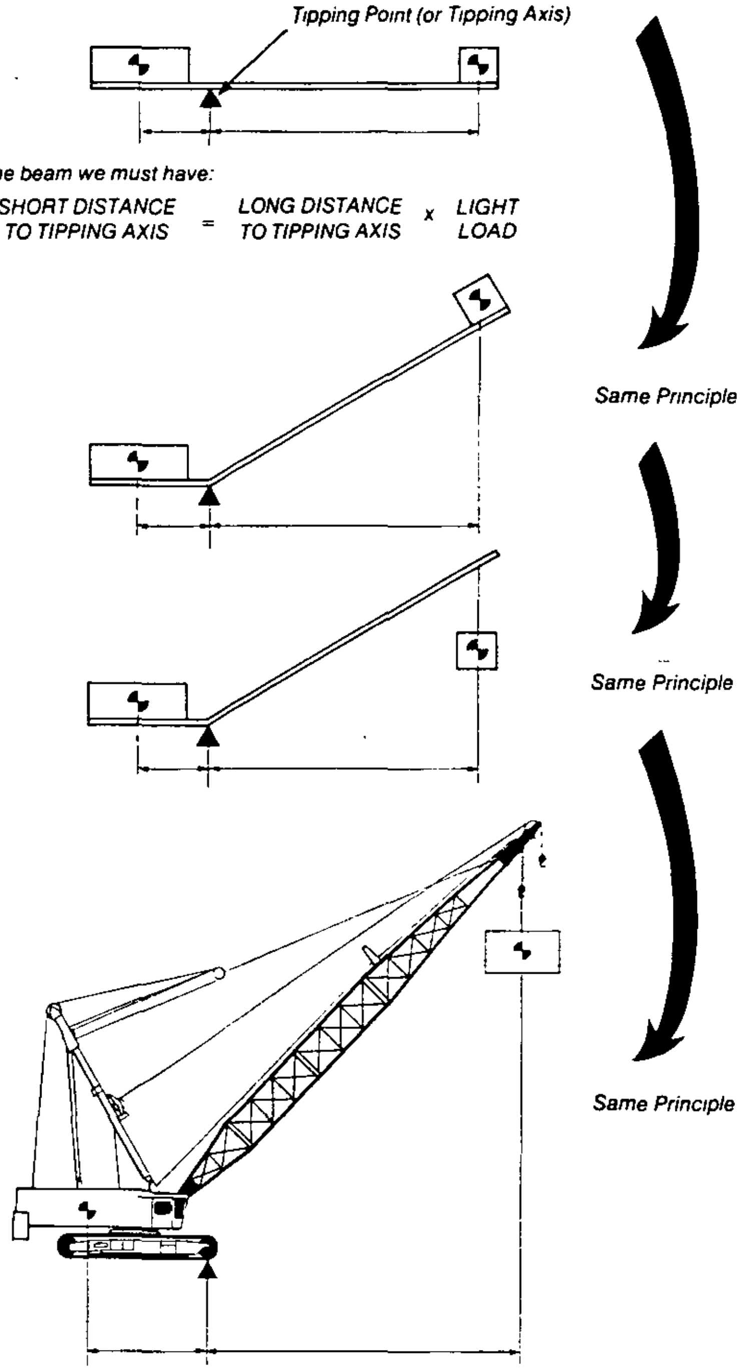


When the pieces are assembled, we can determine the location of the center of gravity of the entire crane.



2.3 PRINCIPLE OF LEVERAGE

Cranes use the principle of leverage to lift loads.



To balance the beam we must have:

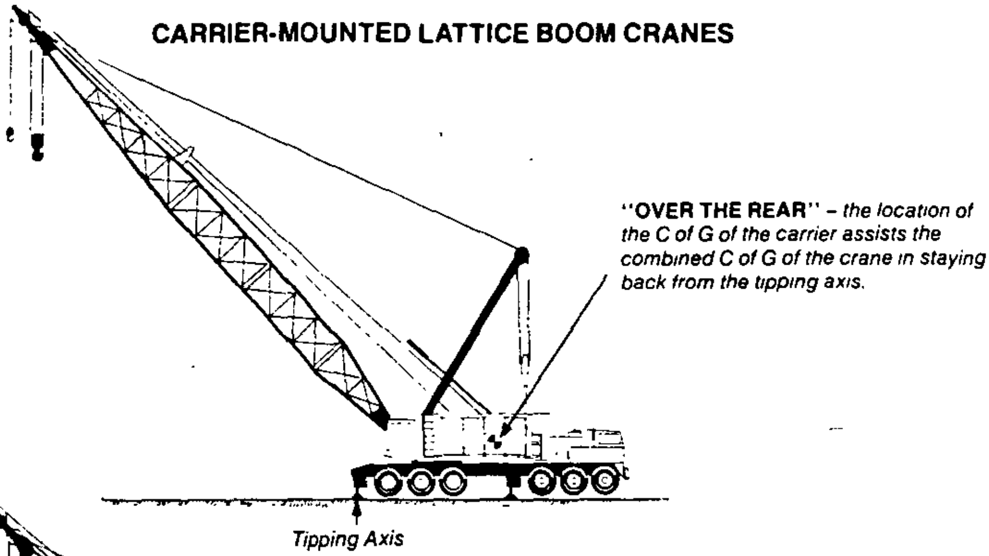
HEAVY LOAD × SHORT DISTANCE TO TIPPING AXIS = LONG DISTANCE TO TIPPING AXIS × LIGHT LOAD

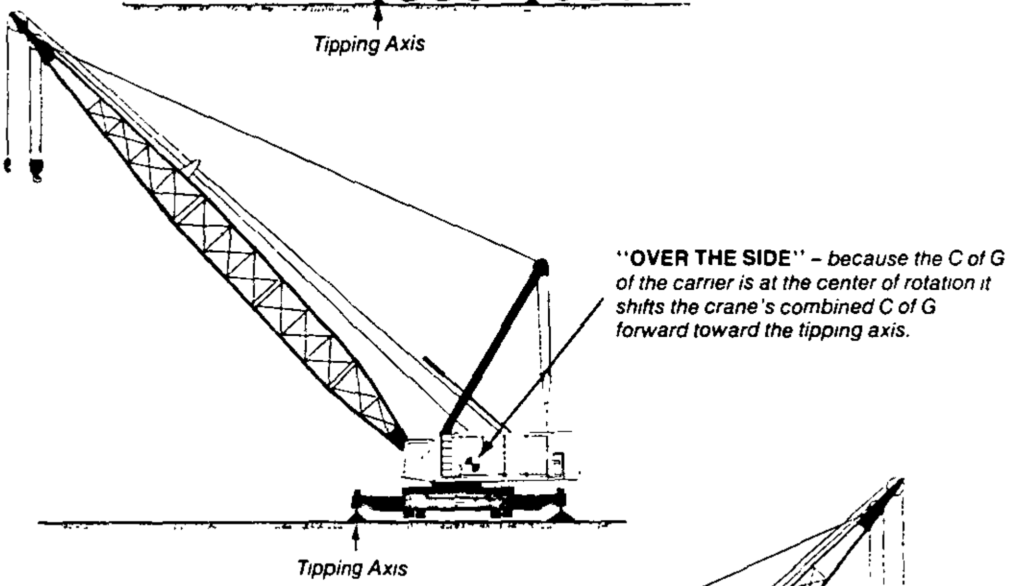
$$\underbrace{\text{CRANE WEIGHT} \times \text{HORIZONTAL DISTANCE FROM C OF G TO TIPPING AXIS}}_{\text{CRANE'S LEVERAGE}} = \underbrace{\text{HORIZONTAL DISTANCE FROM C OF G OF LOAD TO TIPPING AXIS} \times \text{LOAD WEIGHT}}_{\text{LOAD'S LEVERAGE}}$$

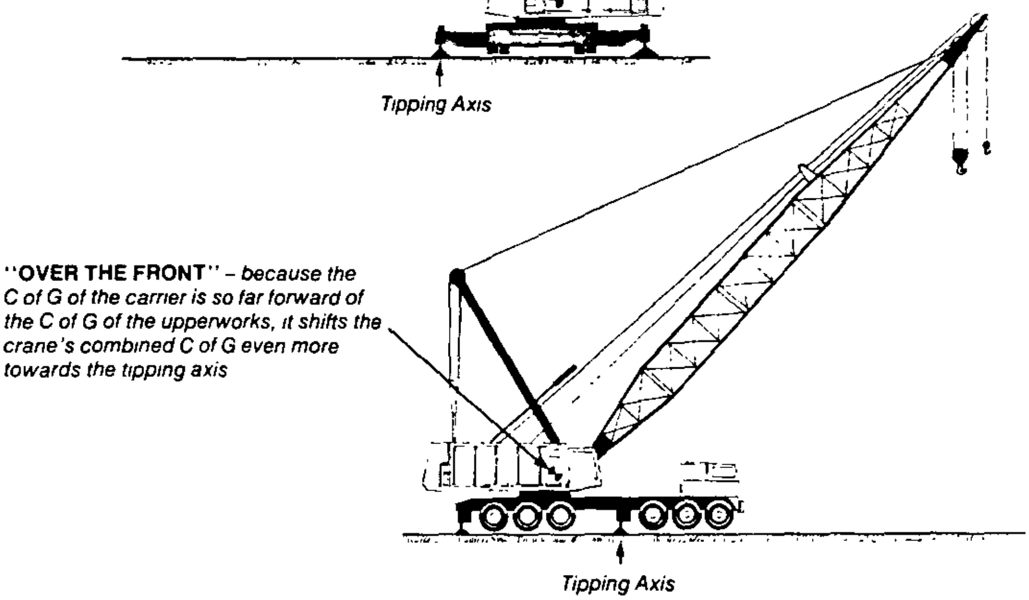
2.4 CHANGES IN LOCATION OF C OF G DURING ROTATION OF UPPERWORKS

Rotation of the upperworks changes the location of the crane's center of gravity.

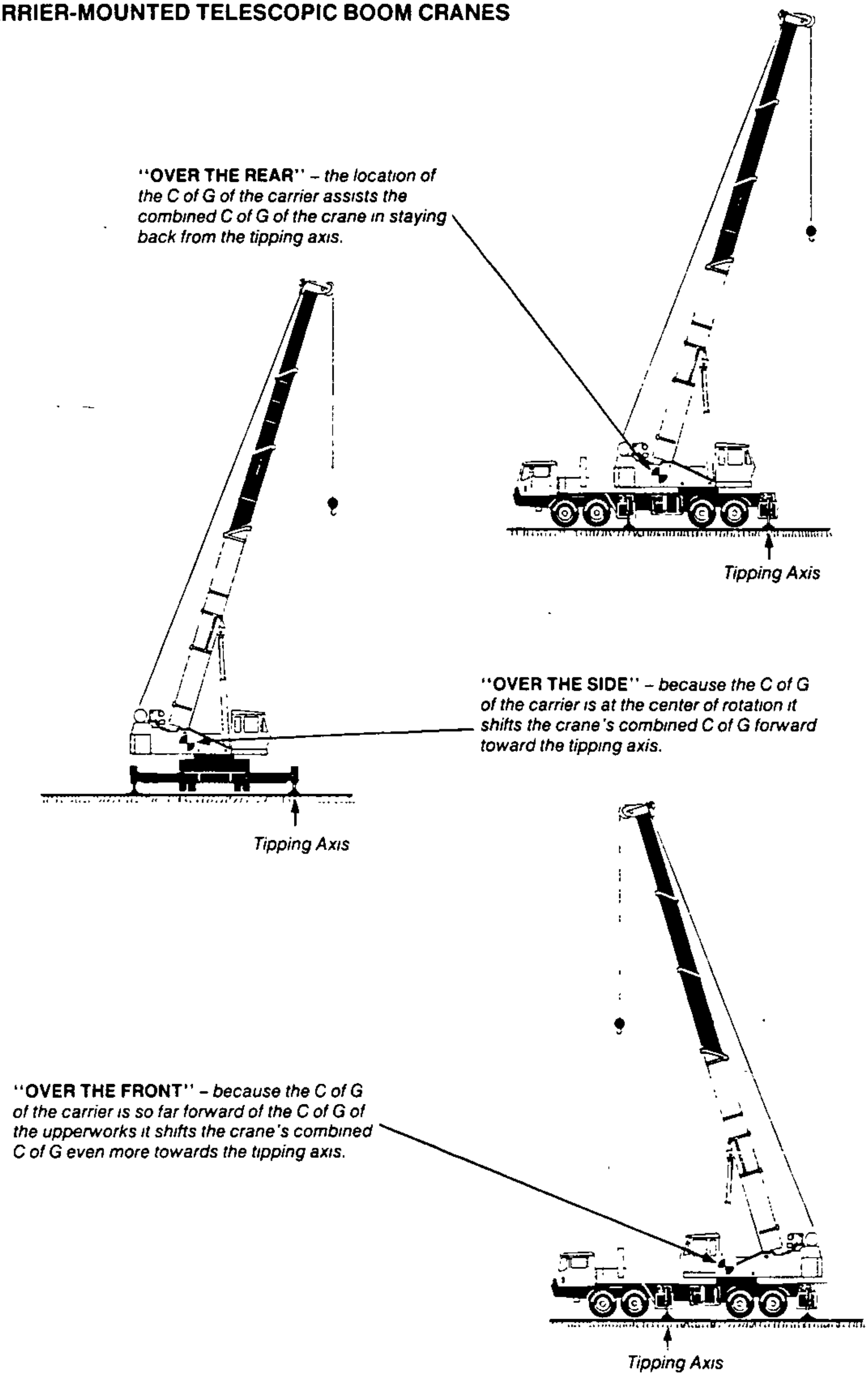
CARRIER-MOUNTED LATTICE BOOM CRANES





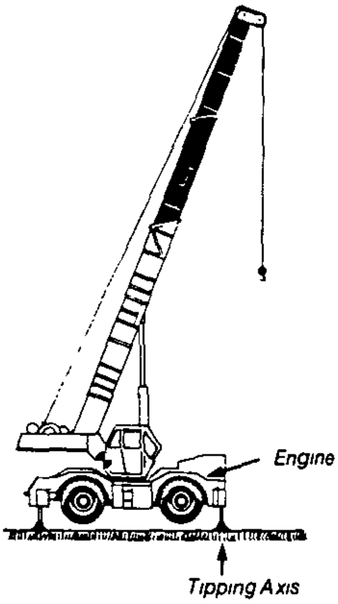


CARRIER-MOUNTED TELESCOPIC BOOM CRANES

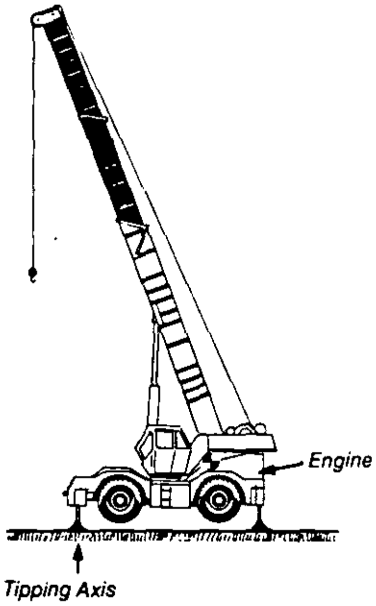


ROUGH TERRAIN CRANES

The location of the engine in the carrier unit affects the location of the center of gravity.

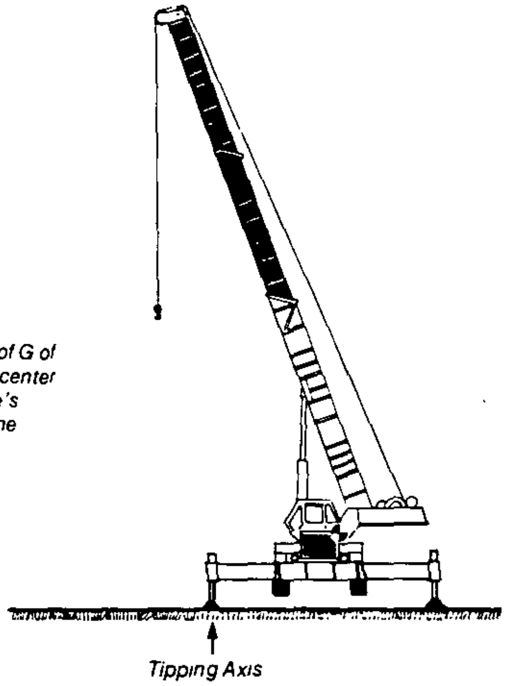


When lifting over the engine the combined C of G of the crane is closer to the tipping axis . . .



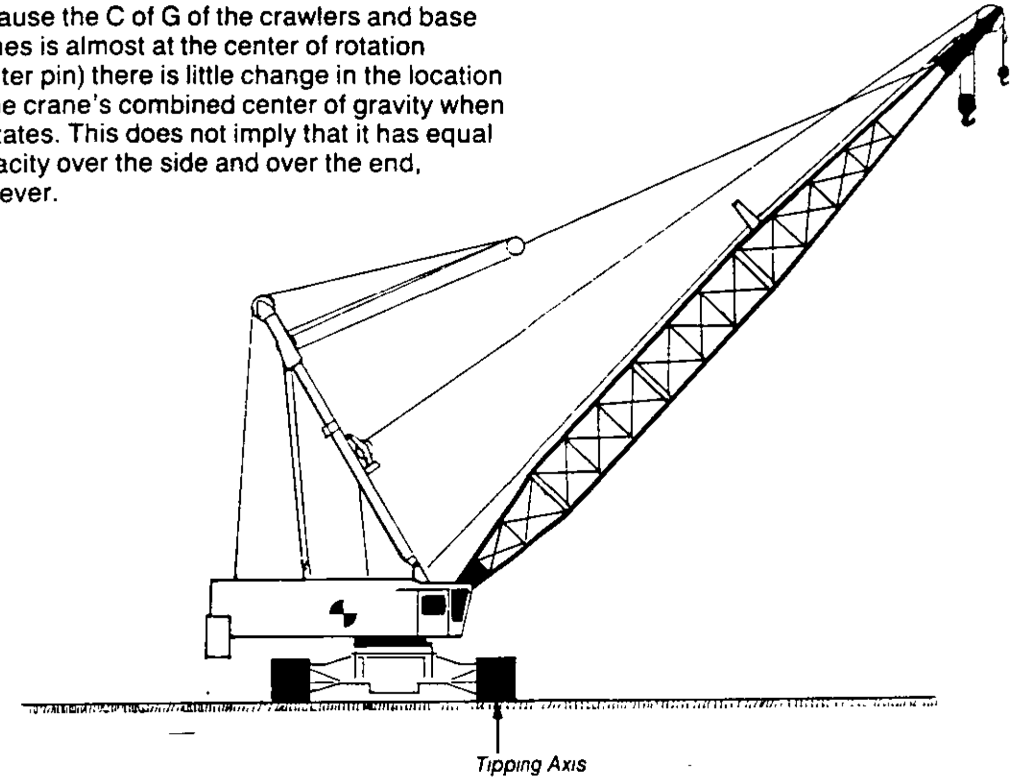
. . than is the case when lifting over the other end. In this case the engine weight shifts the combined C of G further away from the tipping axis.

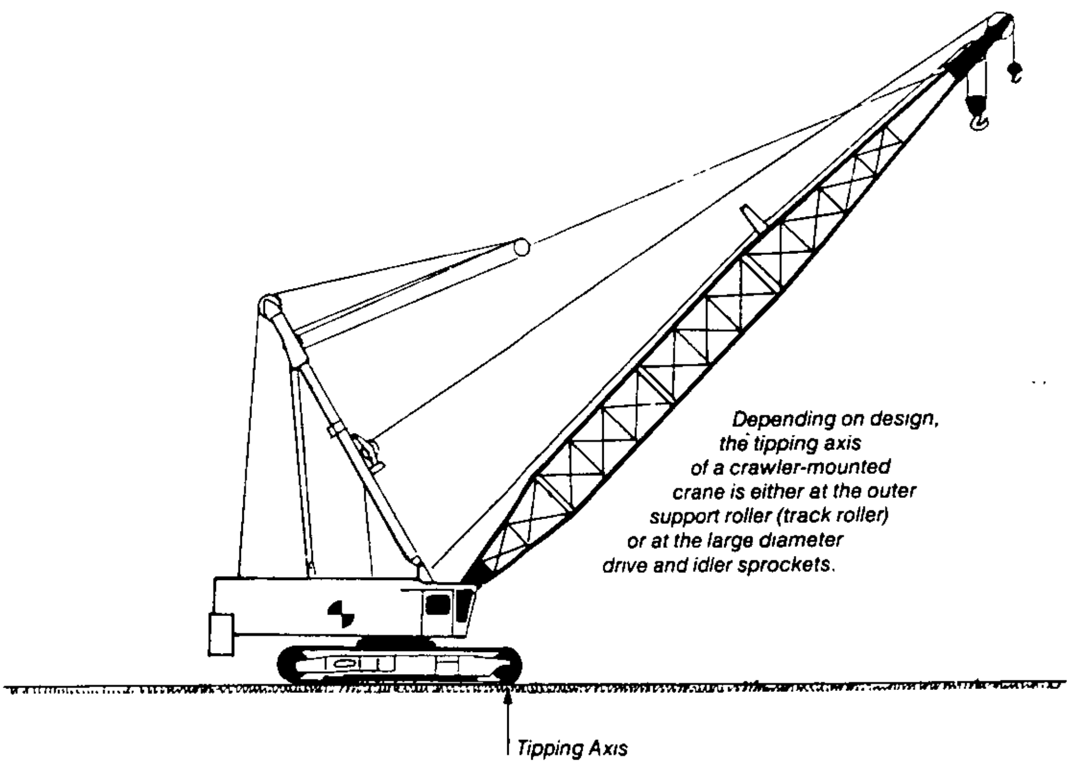
When lifting "over the side" the C of G of the carrier (which is located at the center line) shifts the location of the crane's combined C of G forward toward the tipping axis.



CRAWLER-MOUNTED CRANES

Because the C of G of the crawlers and base frames is almost at the center of rotation (center pin) there is little change in the location of the crane's combined center of gravity when it rotates. This does not imply that it has equal capacity over the side and over the end, however.

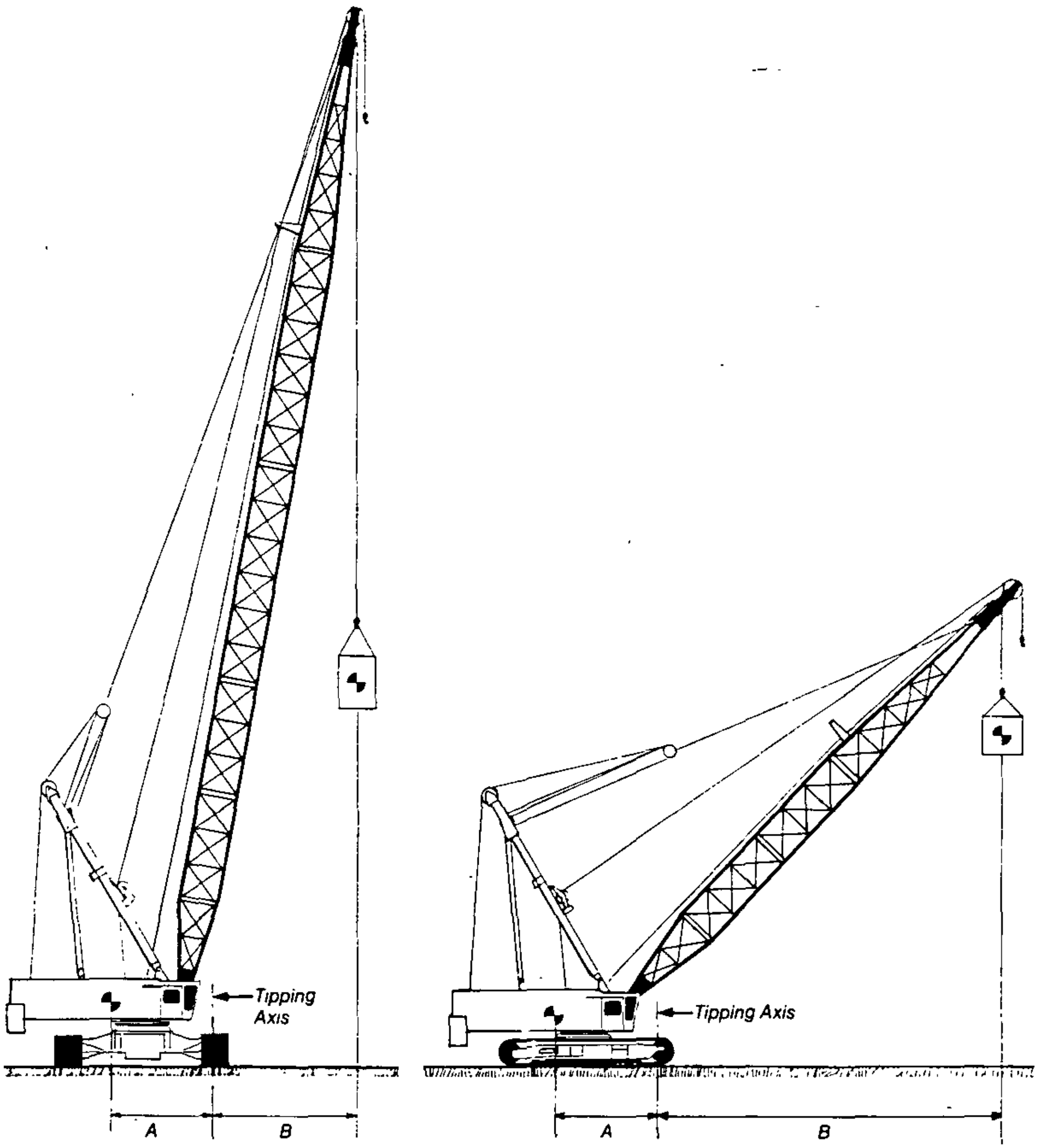




2.5 LEVERAGE AND STABILITY

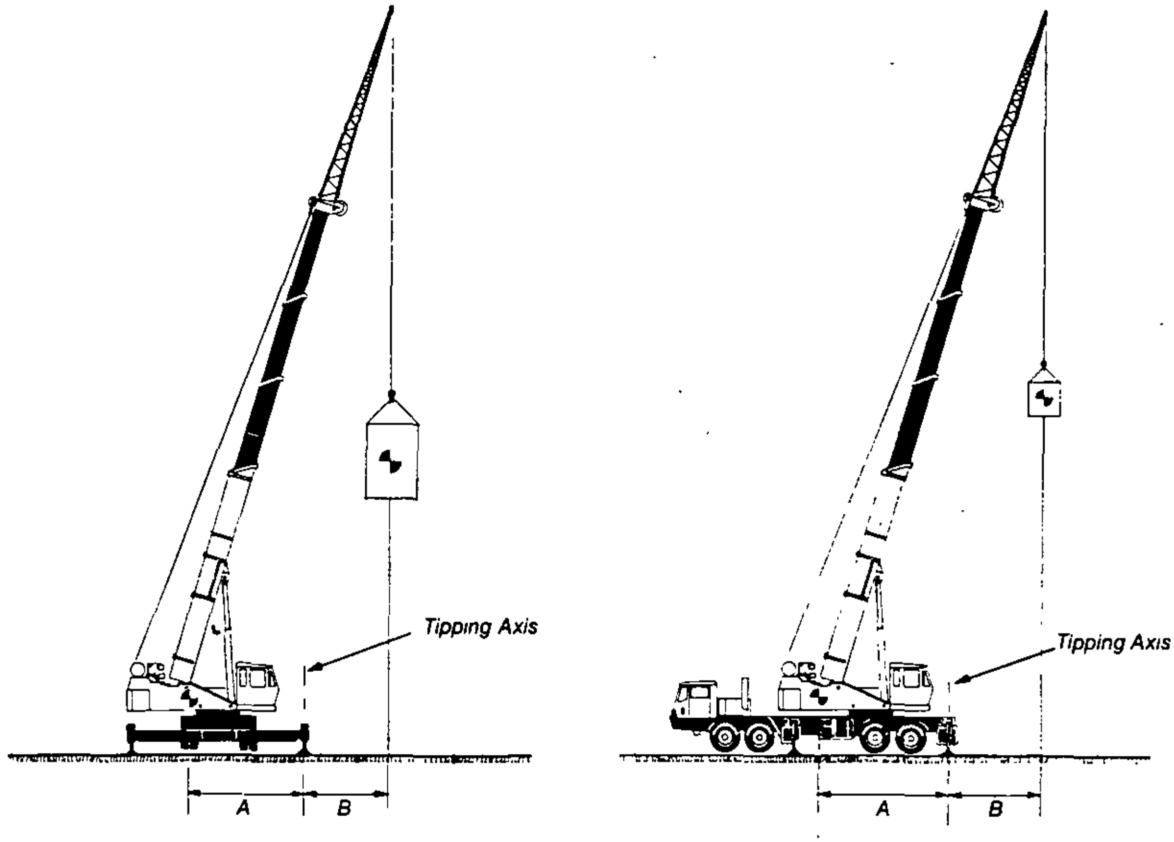
In Section 2.3 we saw that the crane exerts leverage on the load (its weight x the distance of its C of G to the tipping axis) *but* the load also exerts leverage on the crane (load weight x the distance of its C of G to the tipping axis).

A mobile crane is stable when its leverage on the load is *greater* than the load's leverage on the crane. So for simple balance the functions are equal. But for a lift, the crane's leverage must be greater than the load's.

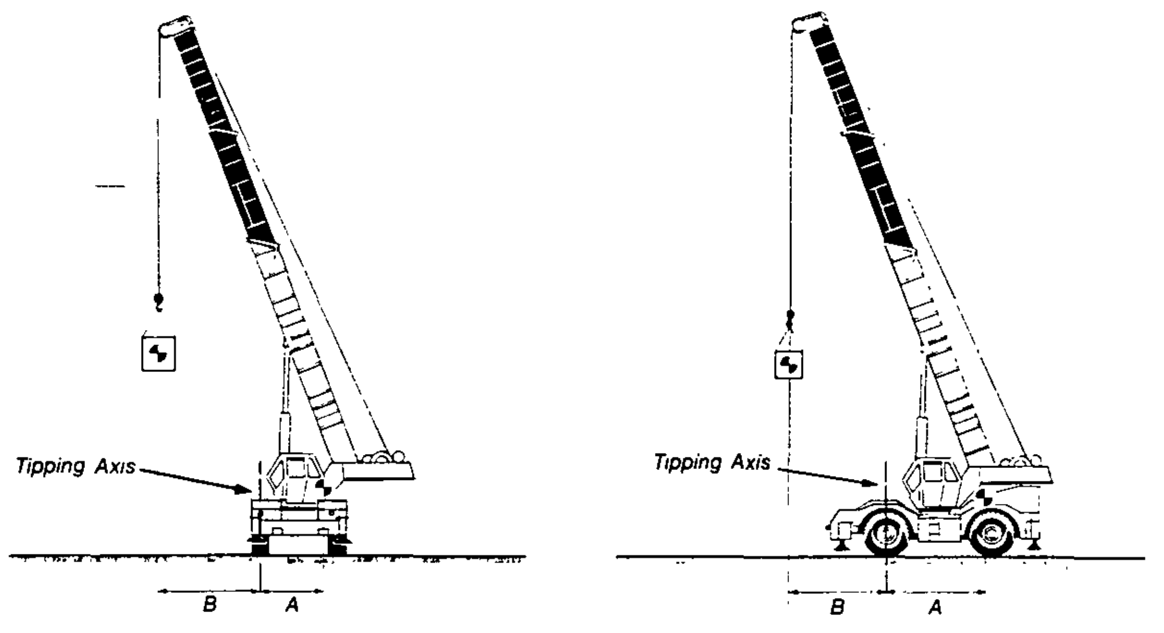


STABLE WHEN: (CRANE WEIGHT x A) GREATER THAN (LOAD WEIGHT x B)

2.5 CONTINUED



STABLE WHEN: (CRANE WEIGHT x A) GREATER THAN (LOAD WEIGHT x B)



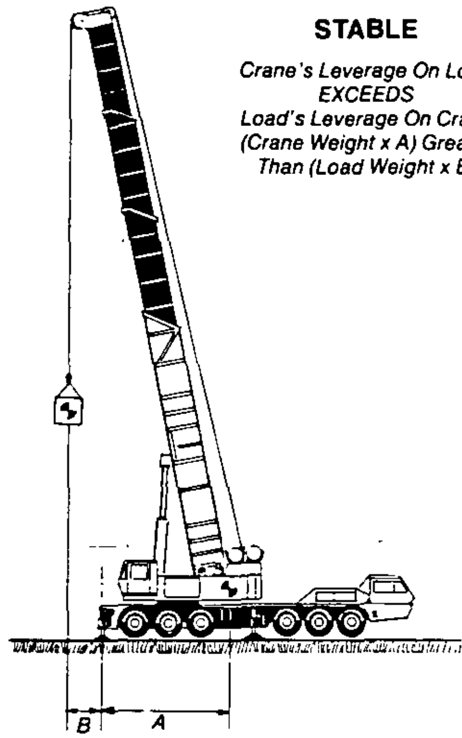
STABLE WHEN. (CRANE WEIGHT x A) GREATER THAN (LOAD WEIGHT x B)

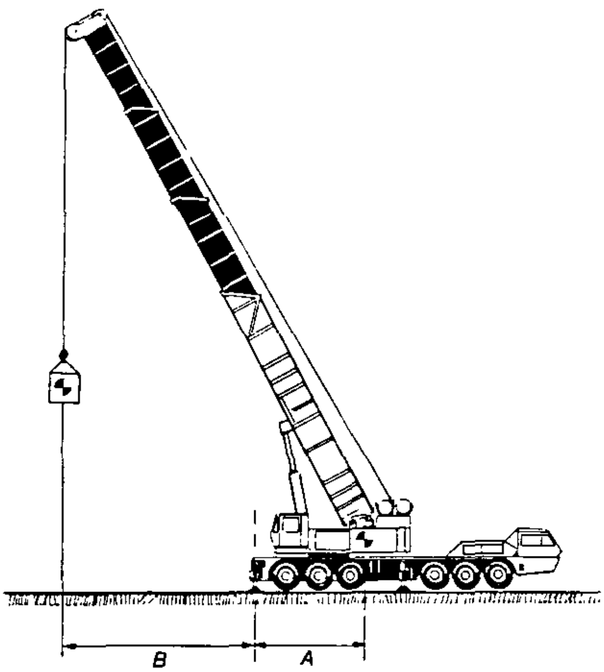
2.6 STABILITY VS INSTABILITY

A crane's stability decreases as the load radius increases. It also decreases as the weight of the load increases.

STABLE

Crane's Leverage On Load
EXCEEDS
Load's Leverage On Crane
(Crane Weight x A) Greater
Than (Load Weight x B)



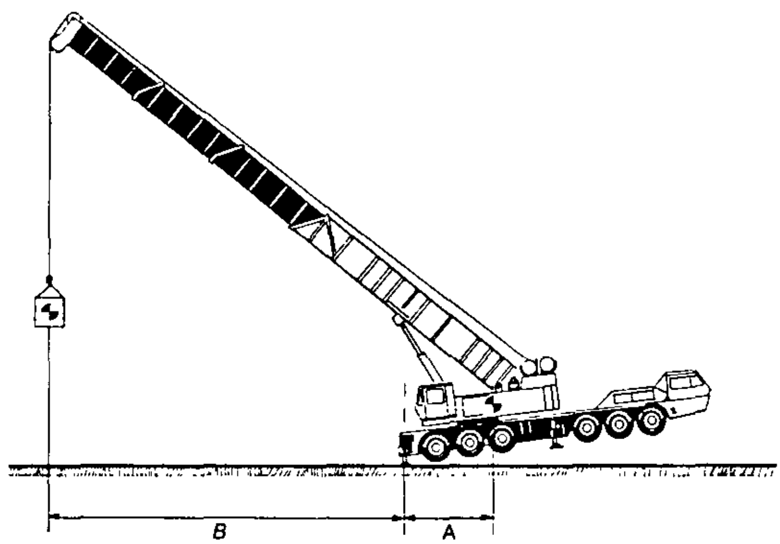


ON THE BRINK OF INSTABILITY

Crane's Leverage On Load
EQUALS
Load's Leverage On Crane
(Crane Weight x A) = (Load Weight x B)

UNSTABLE

Crane's Leverage On Load
LESS THAN
Load's Leverage On Crane
(Crane Weight x A) Less Than
(Load Weight x B)



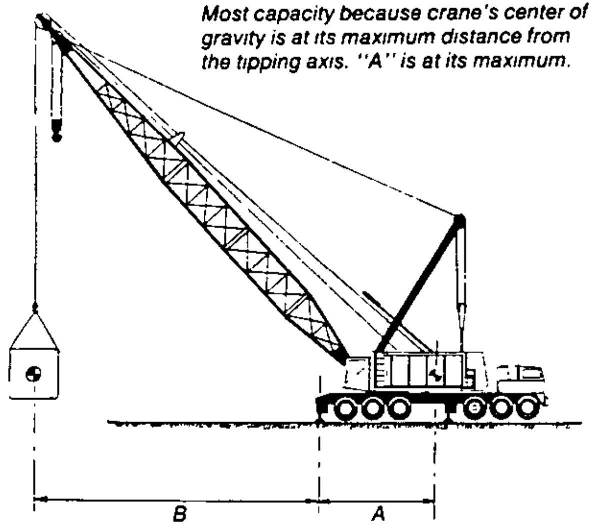
2.7 CHANGES IN CRANE LEVERAGE AND CAPACITY DURING ROTATION OF UPPERWORKS

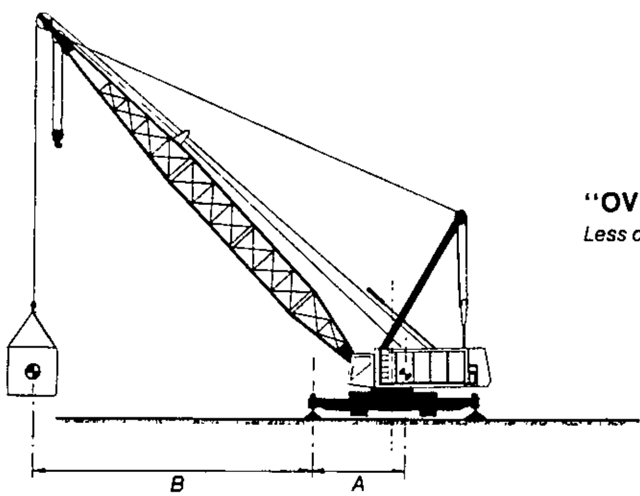
The leverage of a mobile crane changes as the upperworks rotates. This is because the location of the crane's center of gravity changes during rotation (see Section 2.4), and because the distance of the crane's center of gravity to its tipping axis also changes. This means in turn that the leverage the crane exerts on the load changes as it swings. This can affect stability.

The crane's rated capacity is therefore altered in the load chart to compensate for the change in leverage.

"OVER THE REAR"

Most capacity because crane's center of gravity is at its maximum distance from the tipping axis. "A" is at its maximum.



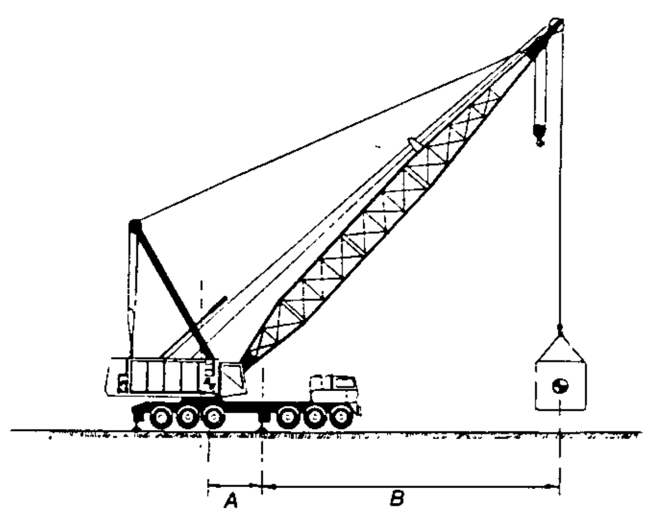


"OVER THE SIDE"

Less capacity because "A" is less than it is for "over the rear".

"OVER THE FRONT"

Least capacity because "A" is at its minimum. This situation changes totally, however, if the crane is equipped with a front bumper outrigger. "A" increases in length considerably and the capacity increases proportionally.



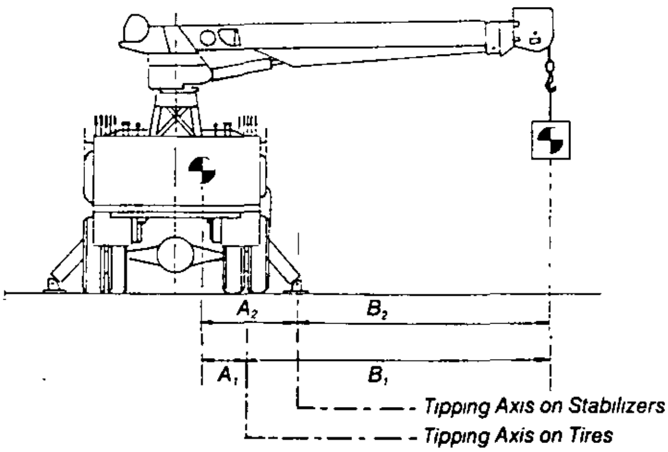
2.8 EFFECT OF LOCATION OF TIPPING AXIS ON STABILITY AND CAPACITY

Providing the ground is capable of supporting the load, a crane can be made more stable by moving the tipping axis further away from its C of G. The extra stability gained by moving the tipping axis can then be used to carry more load.

Increased Stability = More Load

INCREASE STABILITY BY:

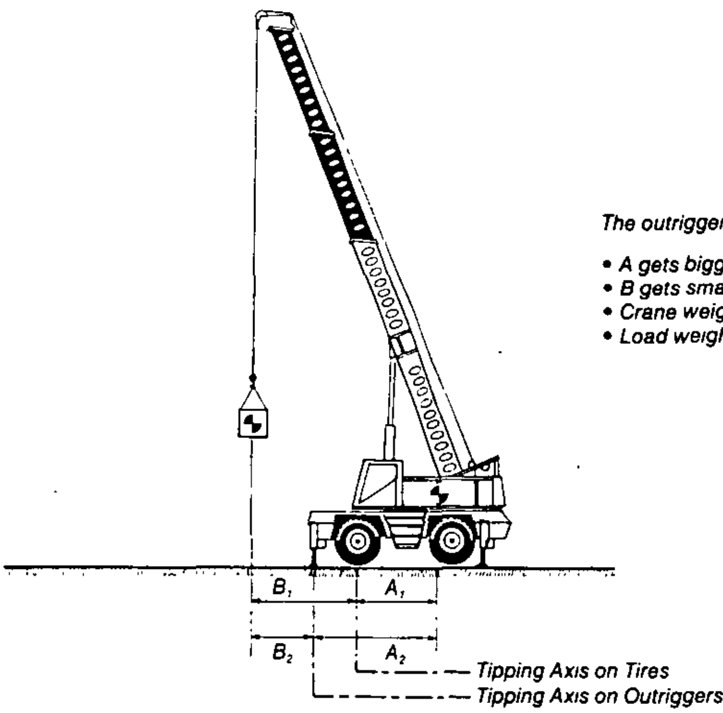
(1) USING STABILIZERS



The stabilizer moves the tipping axis out

- *A gets bigger* $(A_1 \longrightarrow A_2)$
- *B gets smaller* $(B_1 \longrightarrow B_2)$
- *Crane weight stays same.*
- *Load weight can therefore be increased.*

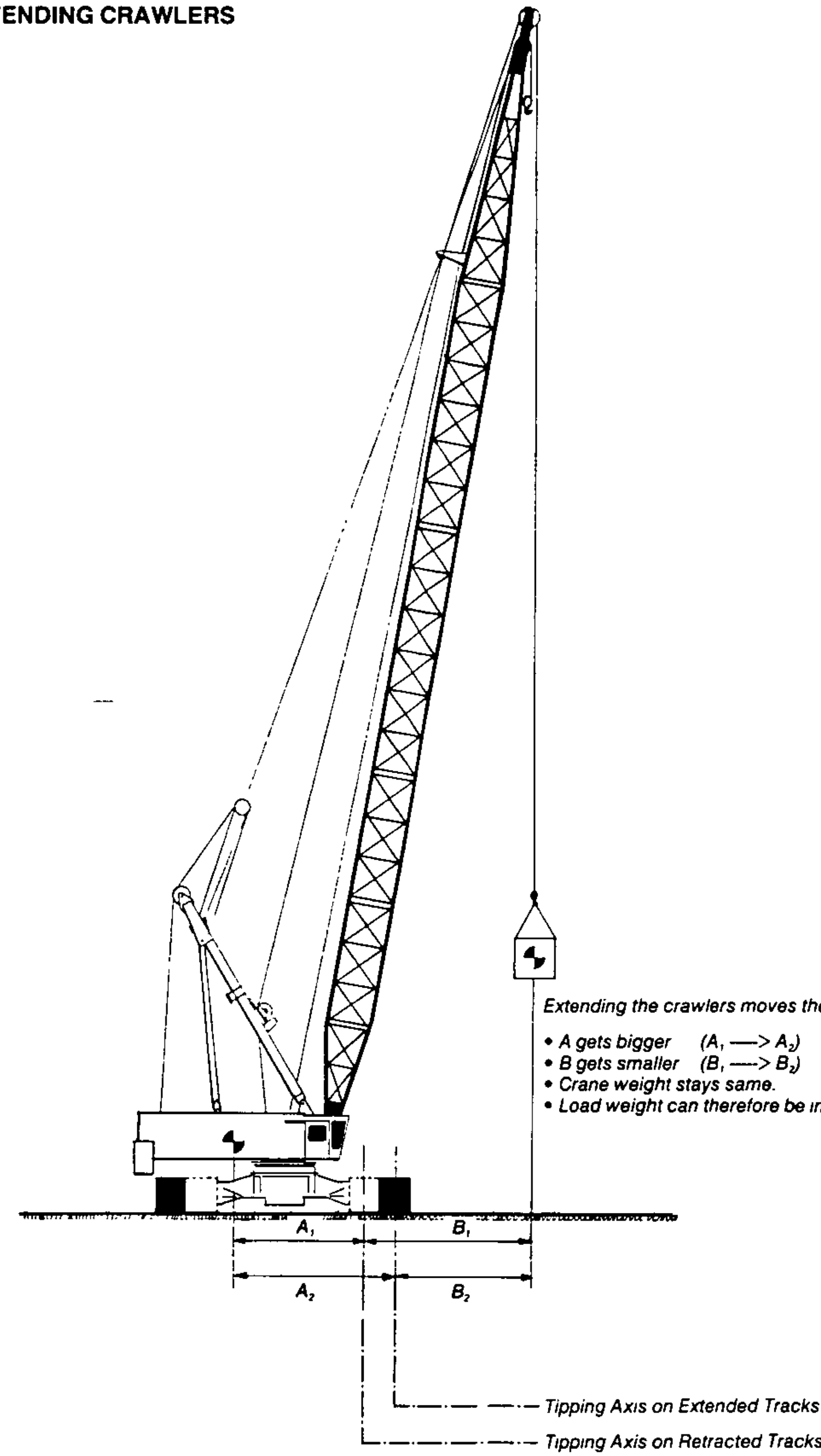
(2) USING OUTRIGGERS



The outrigger moves the tipping axis out.

- *A gets bigger* $(A_1 \longrightarrow A_2)$
- *B gets smaller* $(B_1 \longrightarrow B_2)$
- *Crane weight stays same.*
- *Load weight can therefore be increased.*

(3) EXTENDING CRAWLERS



Extending the crawlers moves the tipping axis out.

- *A gets bigger (A_1 ——> A_2)*
- *B gets smaller (B_1 ——> B_2)*
- *Crane weight stays same.*
- *Load weight can therefore be increased.*

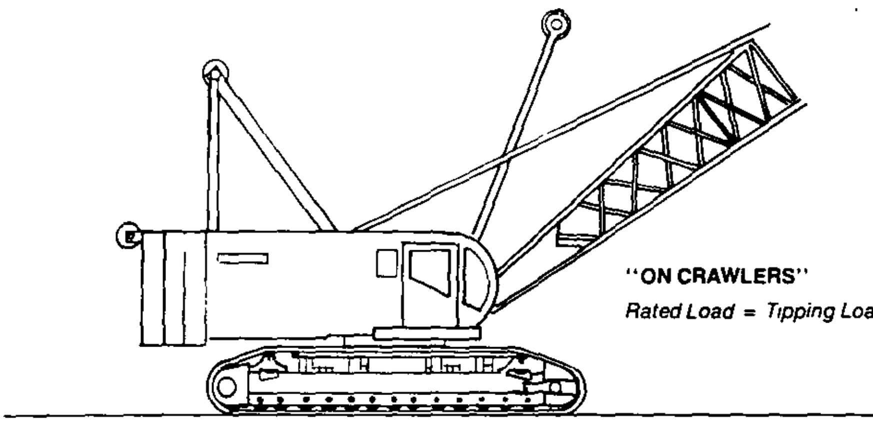
2.9 FORWARD STABILITY FACTORS

To maintain a margin of safety against forward tipping (forward stability failures) all mobile cranes are capacity rated at levels *below* the point at which the load will begin to tip the machine.

The manufacturer loads the crane and determines for every situation listed in the load chart how much weight it takes to make the crane tip. These loads are called the tipping loads.

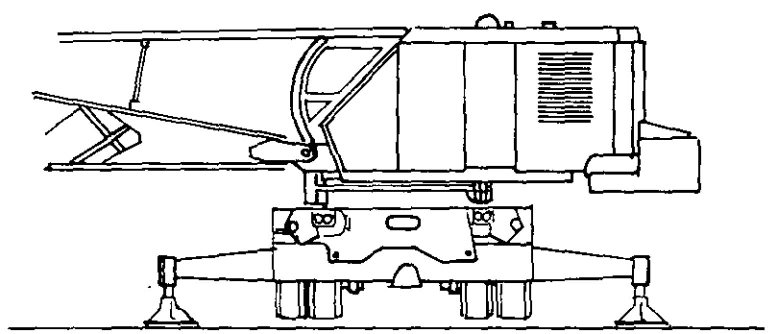
Tipping loads are then reduced by a percentage set by national standards to develop the rated loads listed in the load chart of the machine for every situation.

CRAWLER CRANES

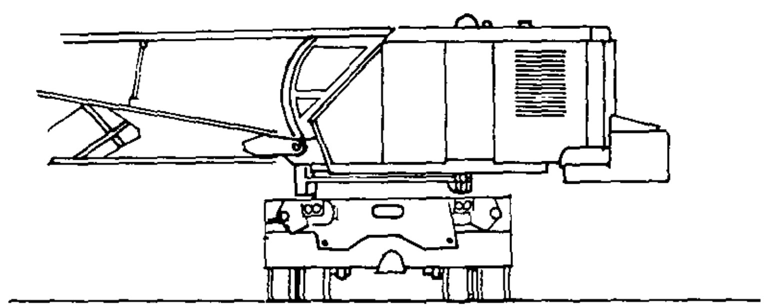


"ON CRAWLERS"
Rated Load = Tipping Load x 0.75

CARRIER-MOUNTED LATTICE AND TELESCOPIC BOOM CRANES



"ON OUTRIGGERS"
Rated Load = Tipping Load x 0.85

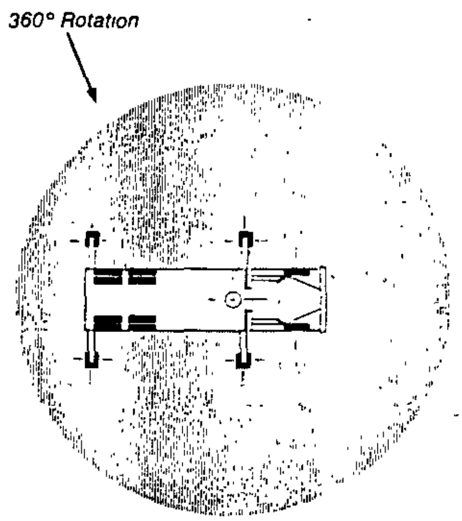


"ON RUBBER"
*Rated Load = Tipping Load x 0.75**

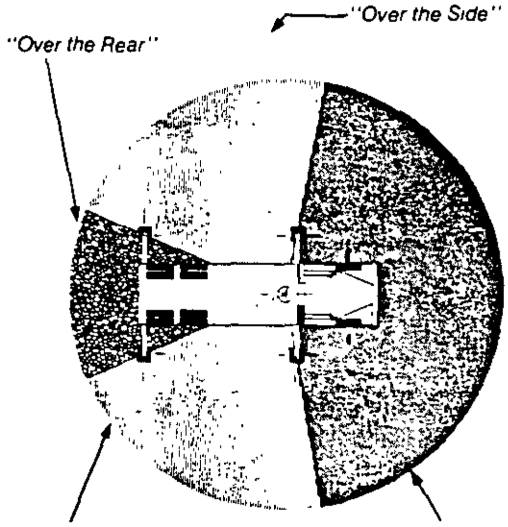
**ANSI Standards B30.5 and B30.15 currently specify 85% but will be lowered to 75% in the next editions Canada specifies 75% in CSA Standard Z150.*

Stabilizers Front and Rear, Turret Behind Cab

Case 1

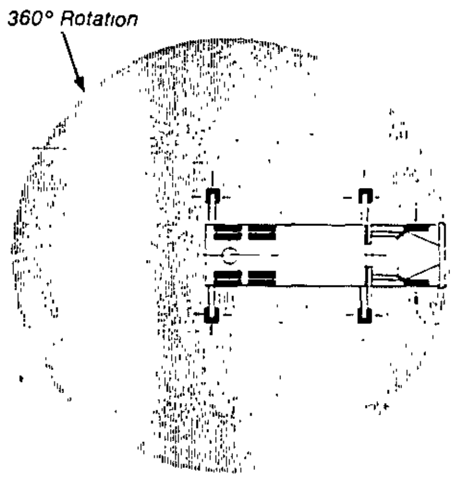


Case 2

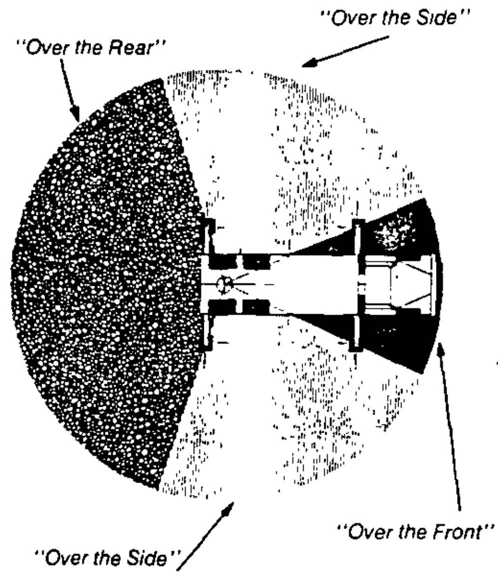


Stabilizers Front and Rear, Turret at Rear of Truck Bed

Case 1



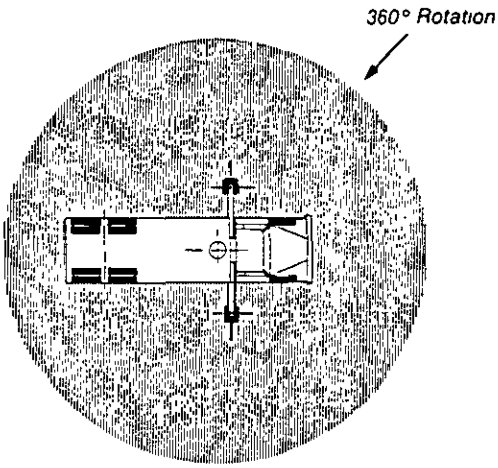
Case 2



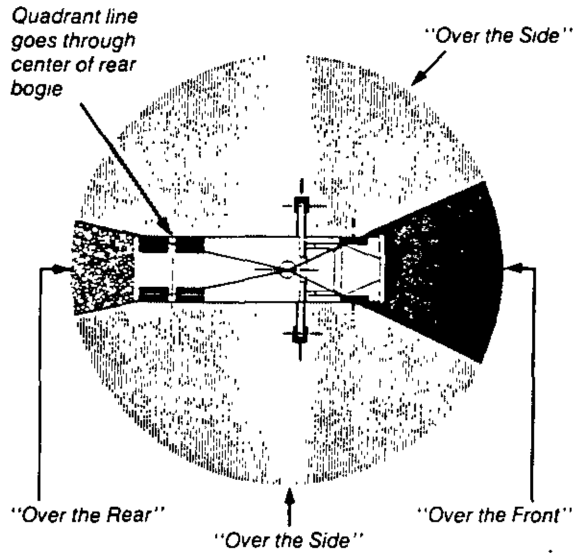
3.7 QUADRANTS FOR BOOM TRUCKS

Stabilizers and Turret Behind Truck Cab

Case 1

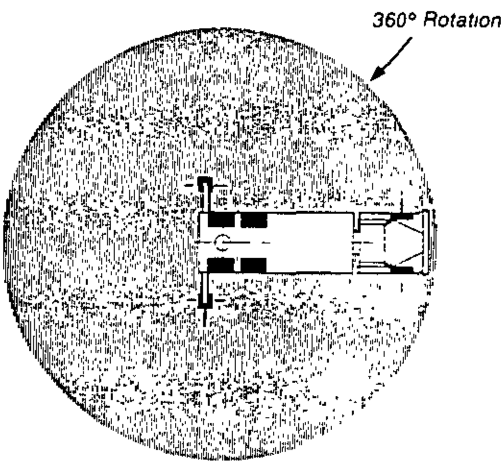


Case 2

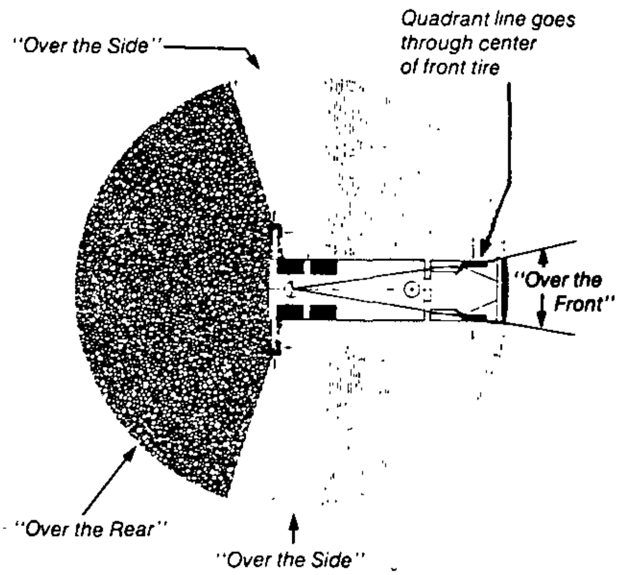


Stabilizers and Turret at Rear of Truck Bed

Case 1



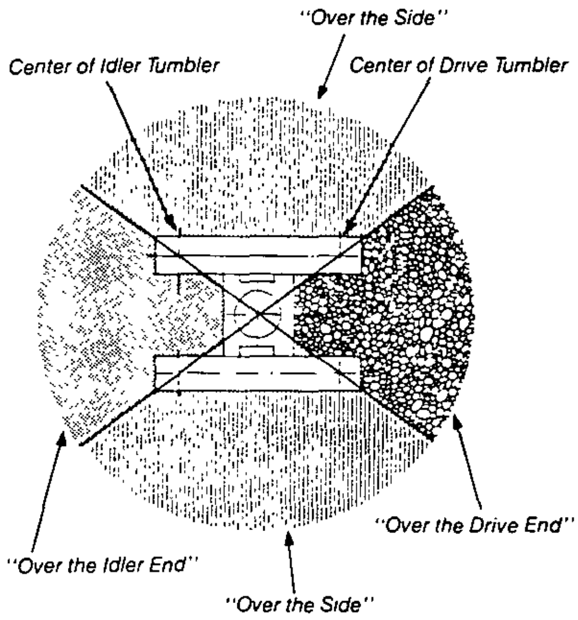
Case 2



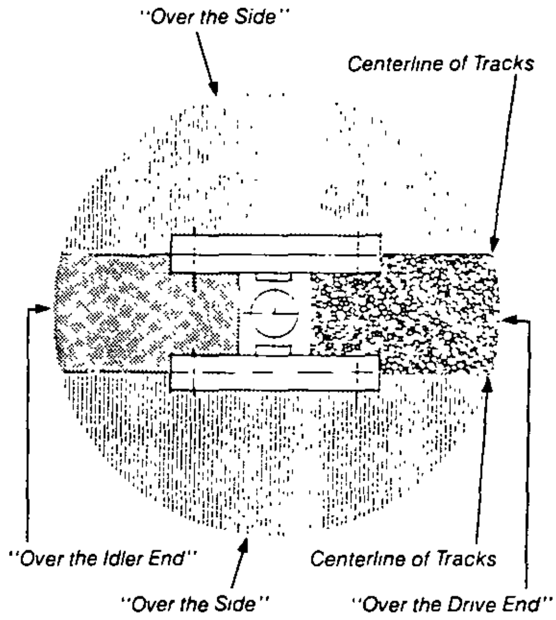
There are two types of base mounting for crawler rigs – center-mounted and offset-mounted. When operating "over the end", operators must always be aware of whether they are over the idler end or the drive end. Capacity may differ accordingly.

Center-Mounted Crawler

Case 1

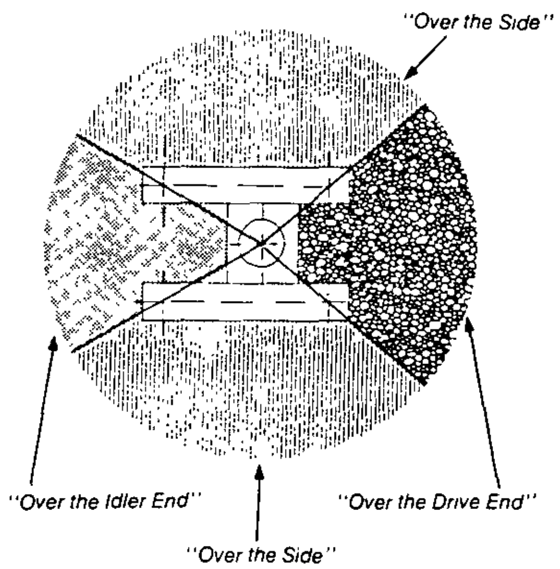


Case 2

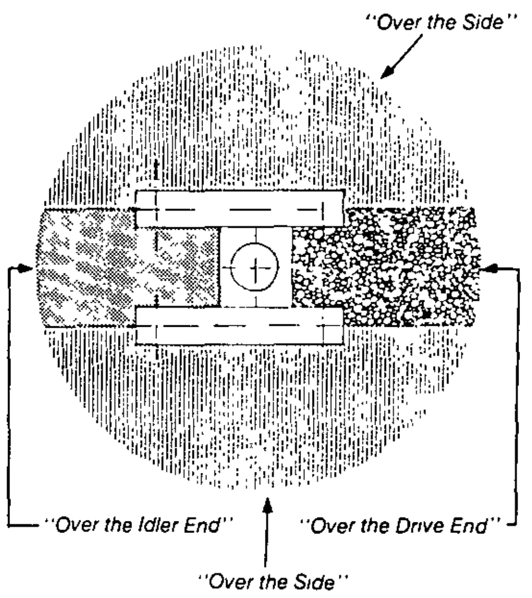


Offset-Mounted Crawler

Case 1



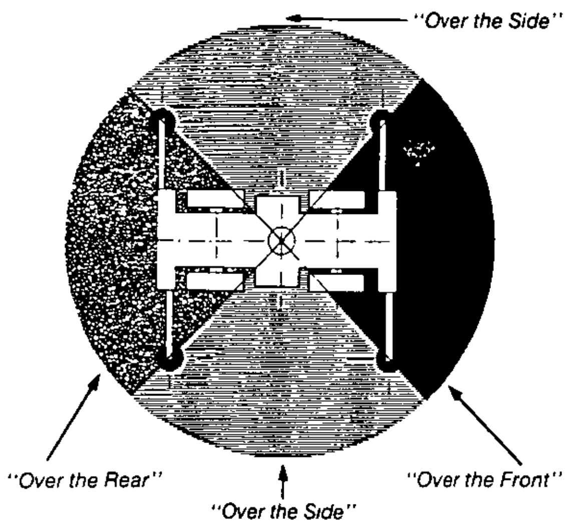
Case 2



TYPE 2 - ENGINE MOUNTED ON UPPERWORKS

"On Outriggers"

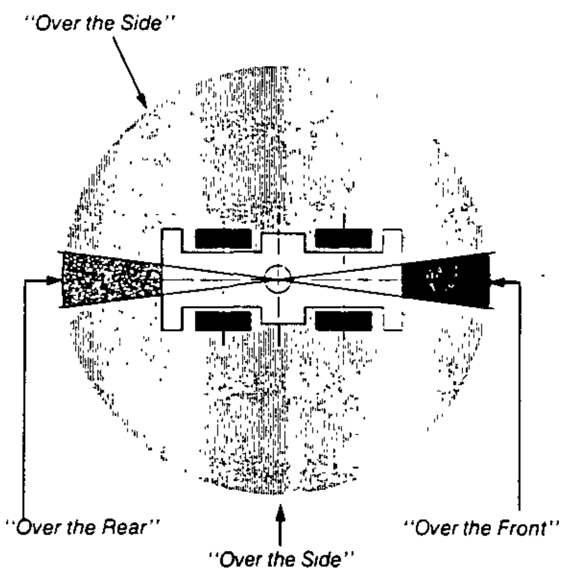
Case 1



Case 2

Similar in layout to Case 2 (page 57) where the engine is mounted on the carrier. The manufacturer will provide a chart for 360° rotation and possibly a second higher capacity chart for "over the front" or "over the rear".

"On Rubber"

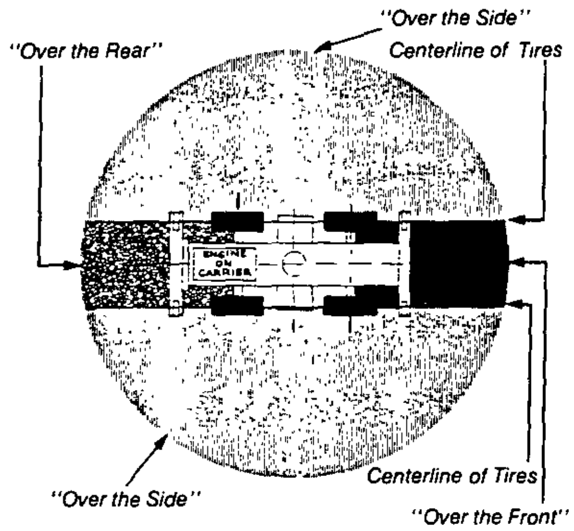


You may also find quadrant layouts similar to the four cases listed for machines with the engine mounted on the carrier.

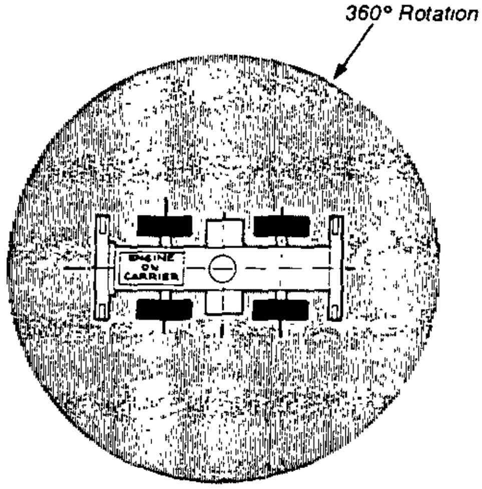
ENGINE MOUNTED ON CARRIER (continued)

"On Rubber"

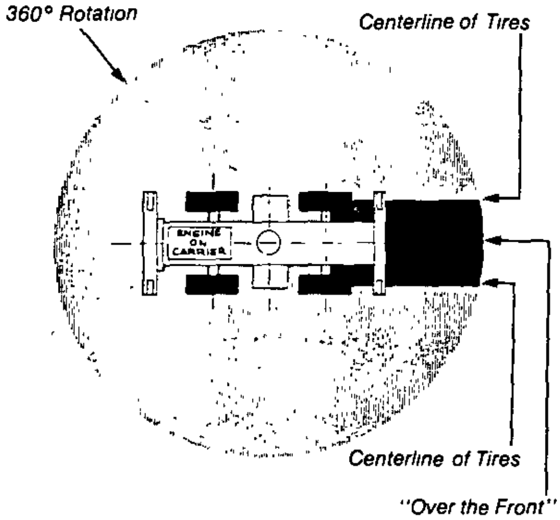
Case 1



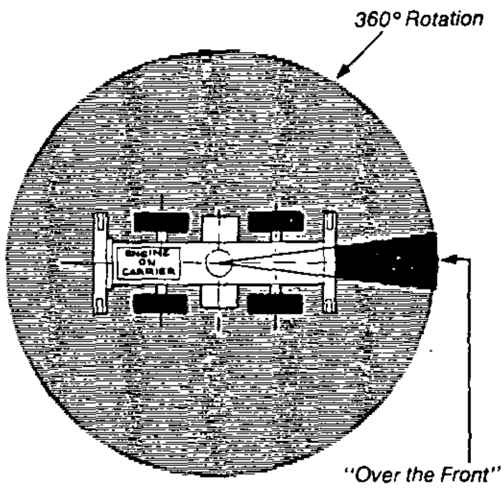
Case 2



Case 3



Case 4



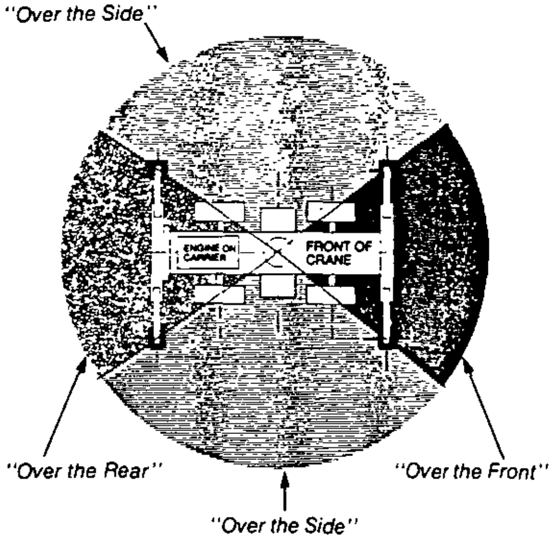
3.5 QUADRANTS FOR ROUGH TERRAIN CRANES

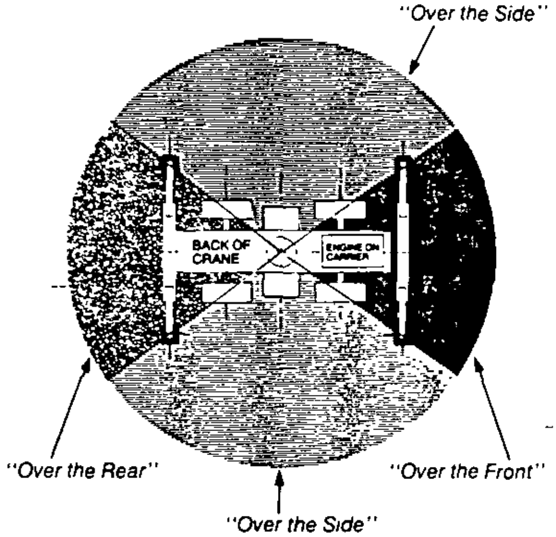
The quadrants for these machines are very similar to those for carrier-mounted units but the operator must be very careful in determining which end of the crane is designated "front" and "rear" by the manufacturer. Terms will differ between manufacturers and types.

TYPE 1—ENGINE MOUNTED ON CARRIER

"On Outriggers"

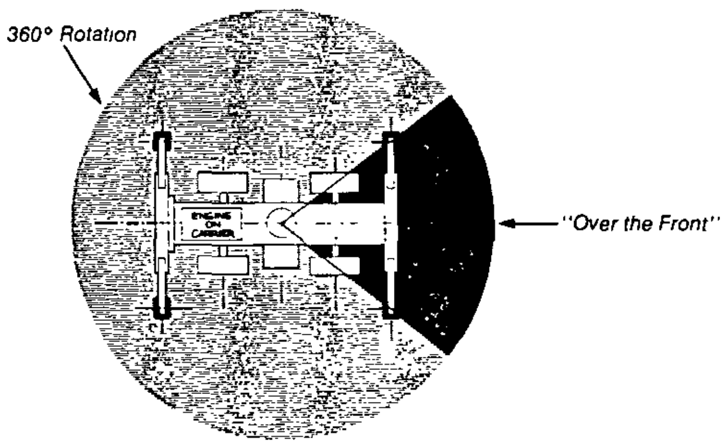
Case 1





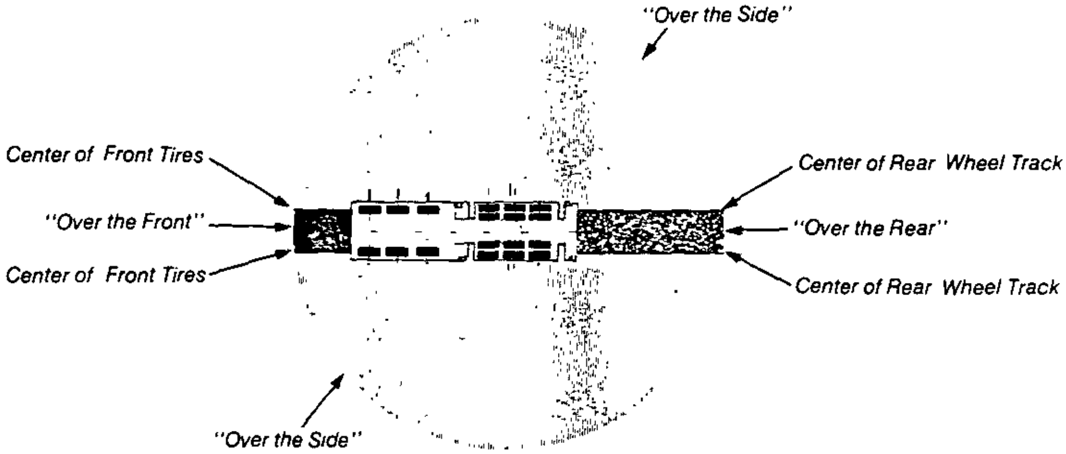
Note:
If the engine is located in the front of the crane, the quadrant designations do not change but the high capacity area changes from "front" to "rear".

Case 2

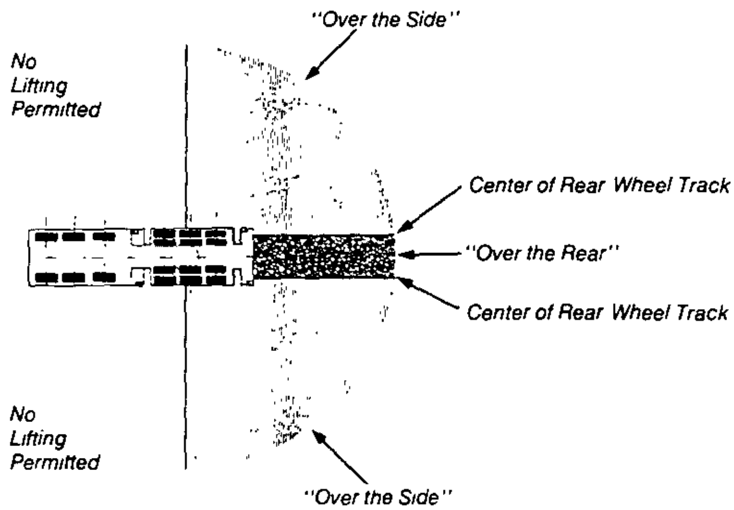


"On Rubber"

Case 1

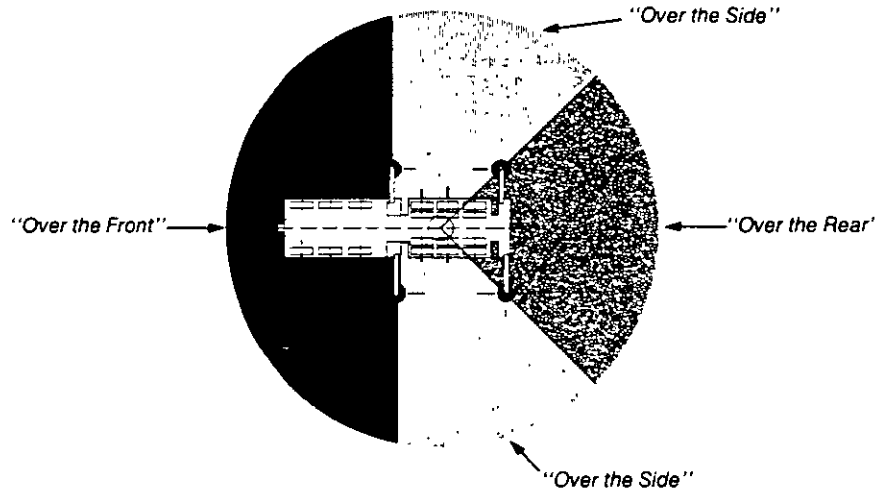


Case 2

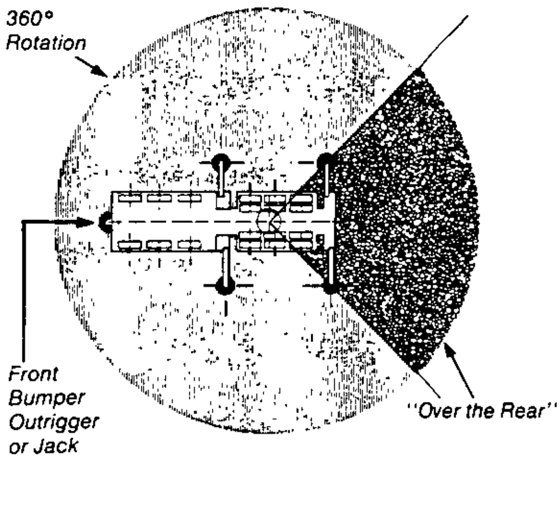


"On Outriggers" (continued)

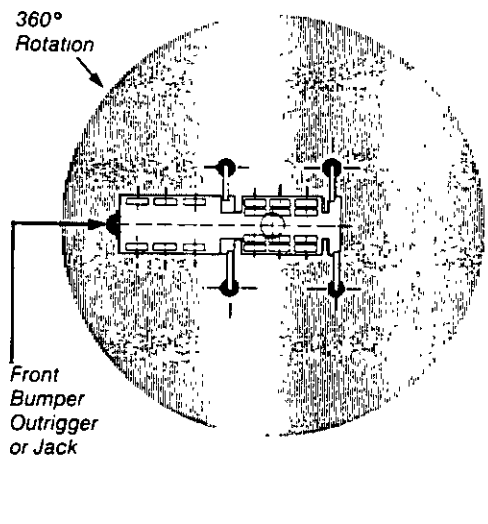
Case 4
with front bumper outrigger or jack



Case 5
with front bumper outrigger or jack



Case 6
with front bumper outrigger or jack

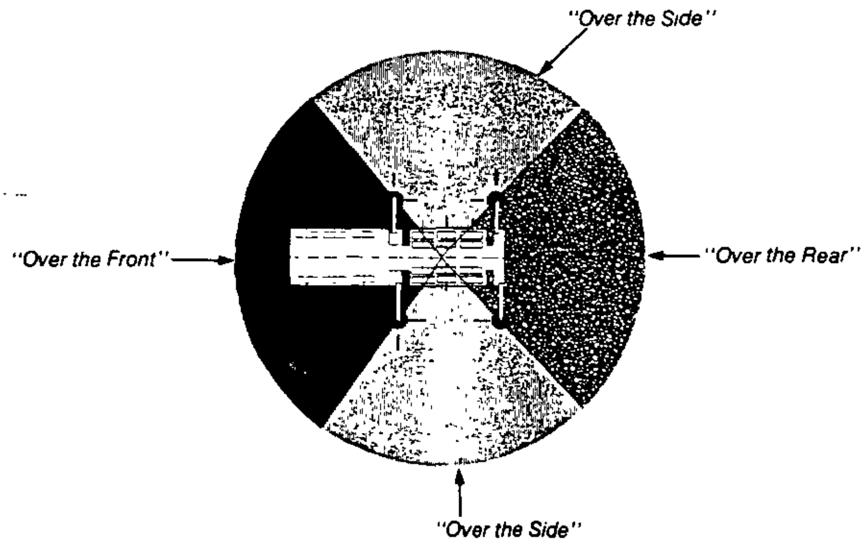


3.4 QUADRANTS FOR CARRIER-MOUNTED CRANES

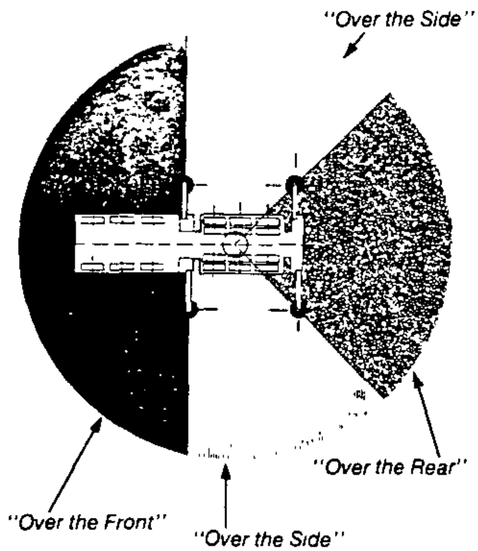
Most load charts will include a small drawing of the crane showing the shape and location of the machine's quadrants of operation. Quadrants will differ depending on crane manufacturer and model type. Use caution and check the specific quadrants developed for the crane being operated. The most common quadrants for carrier-mounted cranes are the following:

"On Outriggers"

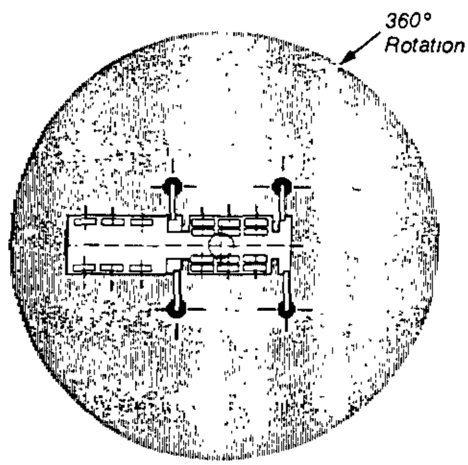
Case 1



Case 2



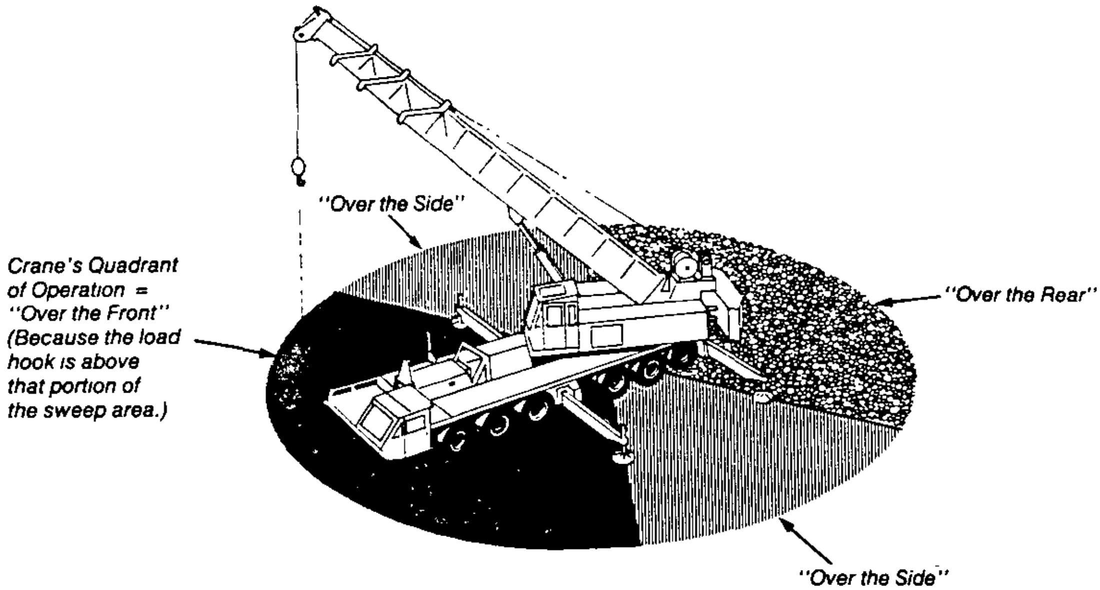
Case 3

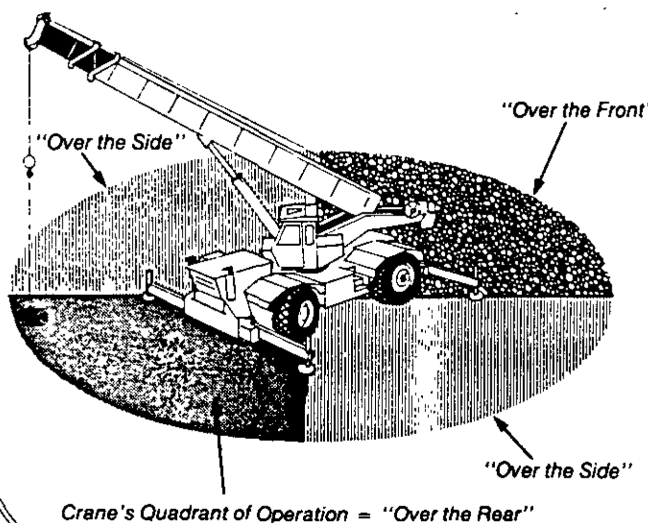


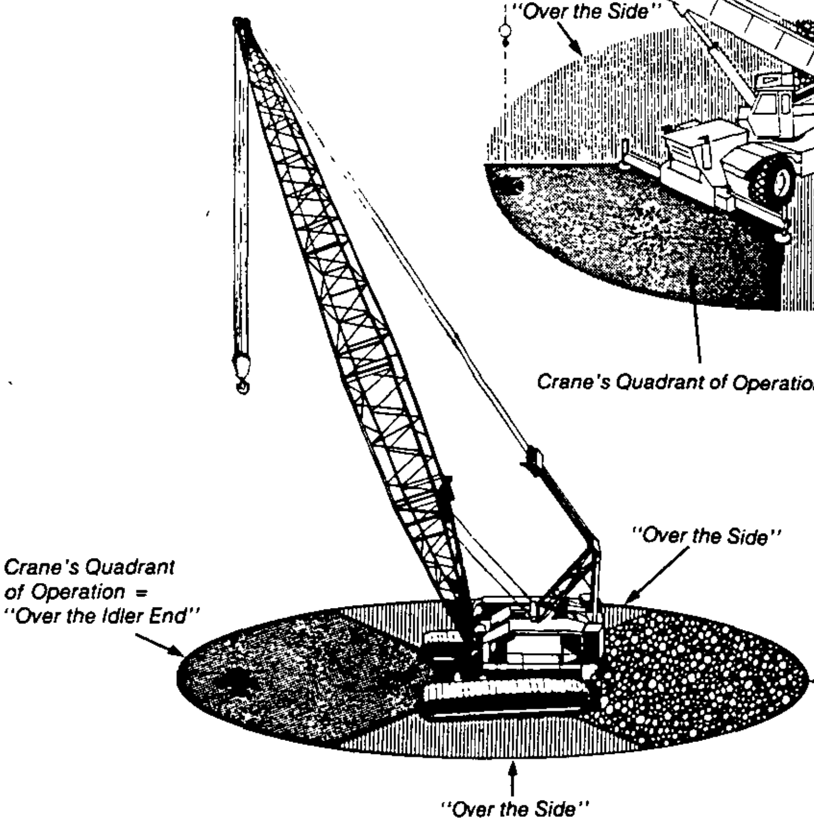
Note: SAE J1028 specifies the quadrants approved for use. The 1982 edition of this standard does not reference all the quadrant shapes included on this and the following pages but because these quadrants have been used in the past and the cranes may still be in operation they are included for reference.

3.3 DIVISION OF SWEEP AREA INTO QUADRANTS

The crane is said to be in a particular quadrant of operation when the load hook is located over that portion of the sweep area.



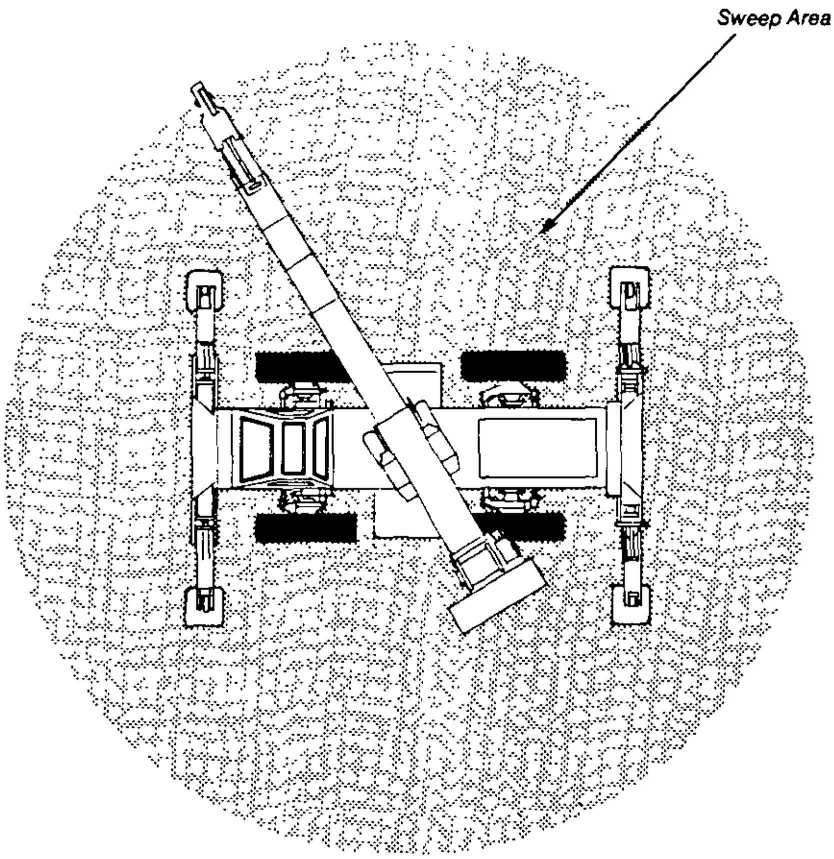




3.2 SWEEP AREA

The sweep area is the total area that the crane boom can swing over.

The sweep area is divided into operating areas called quadrants of operation. The crane's capacity is then based on the quadrants.

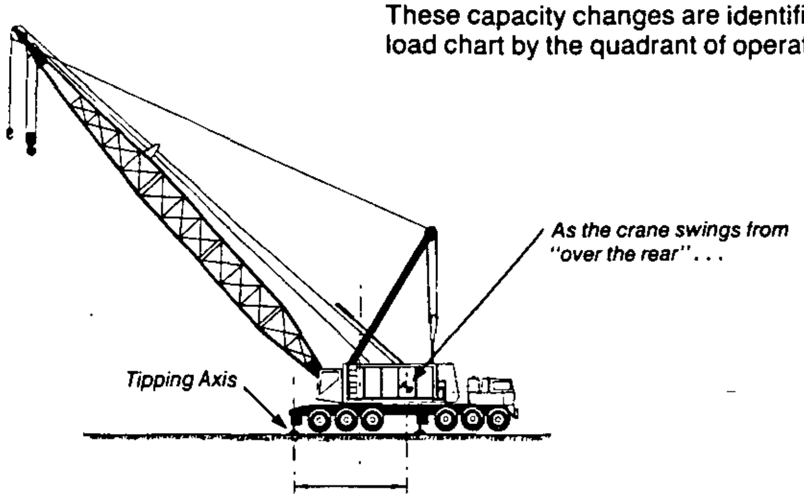


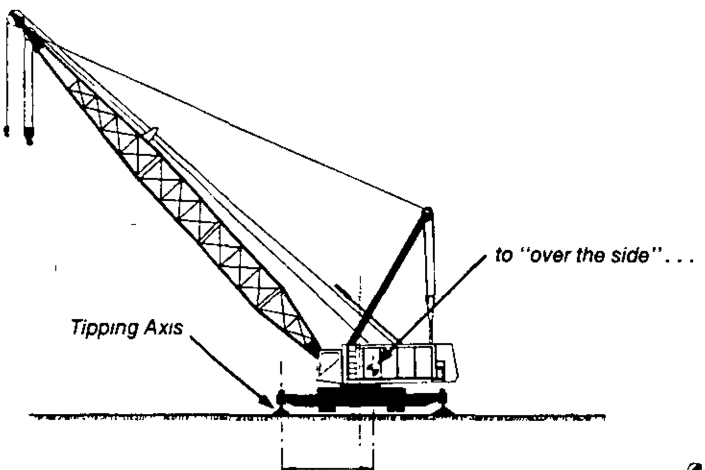
3.1 IMPORTANCE OF QUADRANTS

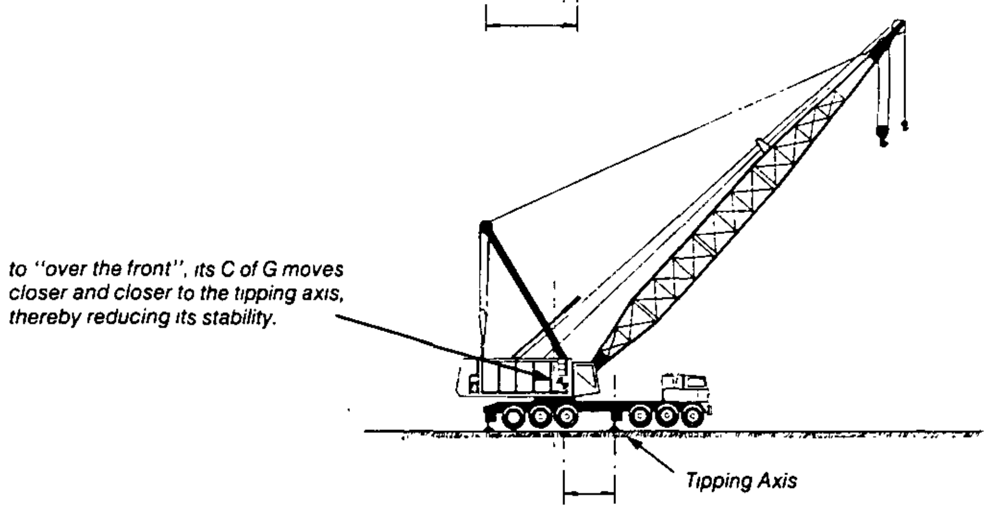
The leverage and capacity of a crane change during rotation of the upperworks. Leverage and capacity are also affected by the location of the tipping axis. For these reasons the crane's stability can change during operation.

To provide uniform stability, regardless of the position of the upperworks relative to the carrier, the crane's capacity is adjusted by the manufacturer according to the quadrant of operation.

These capacity changes are identified in the load chart by the quadrant of operation.







4.4 QUADRANTS OF OPERATION

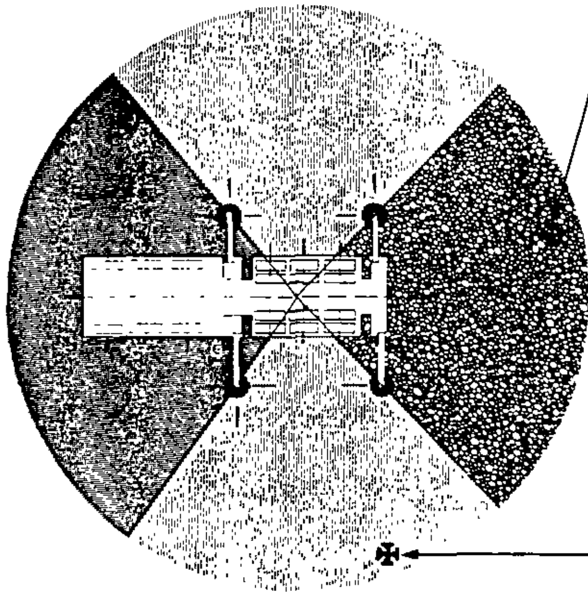
The capacities listed in the load chart are also based on the quadrant of operation of the crane.

Operator must know:

1) Quadrant where pick is being made;
2) Quadrants the load may be carried through; and
3) Quadrant where load is being placed.

> *Caution:* Never put any crane into a position where the load chart shows no capacity for the particular quadrant, boom length, boom angle or load radius. The mere weight of the boom and empty load block can be enough to cause damage or tipping.

Although the quadrant lines are easily distinguished on sketches such as this, they are not so evident when operating the crane. The operator will have to sight along his outrigger cylinders to stay within the range of the quadrant.



Picking the load from the rear quadrant

Placing the load in the side quadrant

RATED LIFTING CAPACITIES IN POUNDS

(Charts not shown in their entirety)

ON OUTRIGGERS FULLY EXTENDED - OVER SIDE

Radius in Feet	Boom Length in Feet							84 ft + 32 ft Ext
	*34	40	44	54	64	74	84	**116
10	100,000 (70)	74,000 (73)	72,000 (76)					
12	90,000 (66.5)	70,000 (70)	67,500 (73.5)	64,000 (76.5)				
15	72,000 (61)	63,700 (65.5)	61,000 (69)	55,000 (73)	44,700 (76)			
20	53,000 (50.5)	52,200 (57.5)	49,800 (62)	44,000 (67.5)	37,900 (71)	35,000 (74)	31,000 (76.5)	
25	39,800 (38.5)	39,800 (48)	39,800 (54)	36,300 (61.5)	31,900 (66)	29,200 (70)	27,500 (73.5)	17,500 (76.5)
30	27,030 (21.5)	27,030 (37.5)	27,030 (45)	27,030 (55.5)	27,000 (60.5)	25,000 (65.5)	23,900 (69.5)	16,600 (75)
35		20,280 (23)	20,280 (34.5)	20,280 (48.5)	20,280 (55)	20,280 (61)	20,280 (66)	14,500 (72.5)
40			15,950 (19)	15,950 (41)	15,950 (49)	15,950 (56.5)	15,950 (62)	12,800 (70)
45				12,840 (31.5)	12,840 (42)	12,840 (51.5)	12,840 (58)	11,400 (67)

ON OUTRIGGERS FULLY EXTENDED - OVER REAR

Radius in Feet	Boom Length in Feet							84 ft + 32 ft Ext
	*34	40	44	54	64	74	84	**116
10	100,000 (70)	74,000 (73)	72,000 (76)					
12	90,000 (66.5)	70,000 (70)	67,500 (73.5)	64,000 (76.5)				
15	72,000 (61)	63,700 (65.5)	61,000 (69)	55,000 (73)	44,700 (76)			
20	53,000 (50.5)	52,200 (57.5)	49,800 (62)	44,000 (67.5)	37,900 (71)	35,000 (74)	31,000 (76.5)	
25	41,000 (38.5)	41,000 (48)	41,000 (54)	36,300 (61.5)	31,900 (66)	29,200 (70)	27,500 (73.5)	17,500 (76.5)
30	29,690 (21.5)	29,690 (37.5)	29,690 (45)	29,690 (55.5)	27,000 (60.5)	25,000 (65.5)	23,900 (69.5)	16,600 (75)
35		22,650 (23)	22,650 (34.5)	22,650 (48.5)	22,650 (55)	21,800 (61)	20,500 (66)	14,500 (72.5)
40			18,090 (19)	18,090 (41)	18,090 (49)	18,090 (56.5)	17,900 (62)	12,800 (70)
45				14,840 (31.5)	14,840 (42)	14,840 (51.5)	14,840 (58)	11,400 (67)

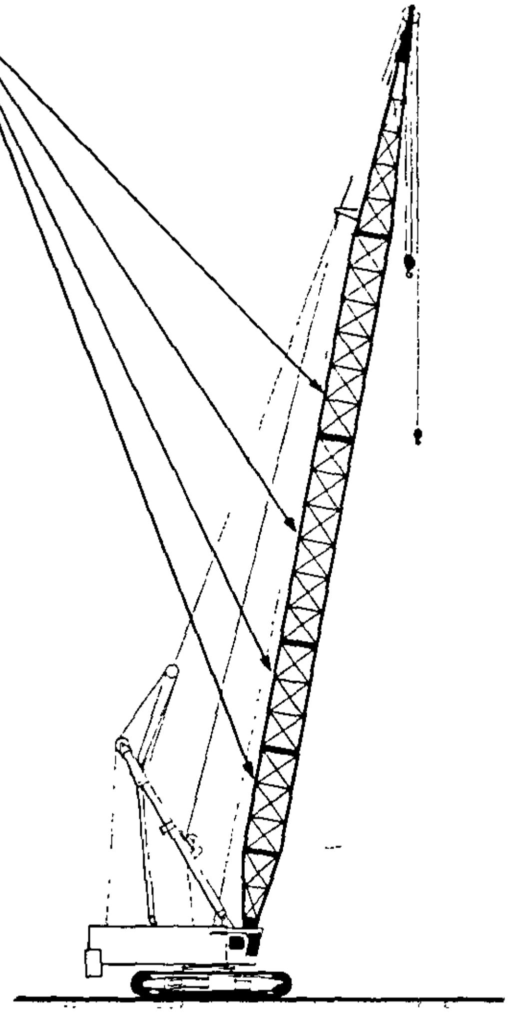
Courtesy Grove Manufacturing Co.

In this case check the load chart capacities in both quadrants. The lower of the two will be the maximum crane rating. Note that in this case it is not possible to swing the load into the front quadrant because the crane is not rated in this area.

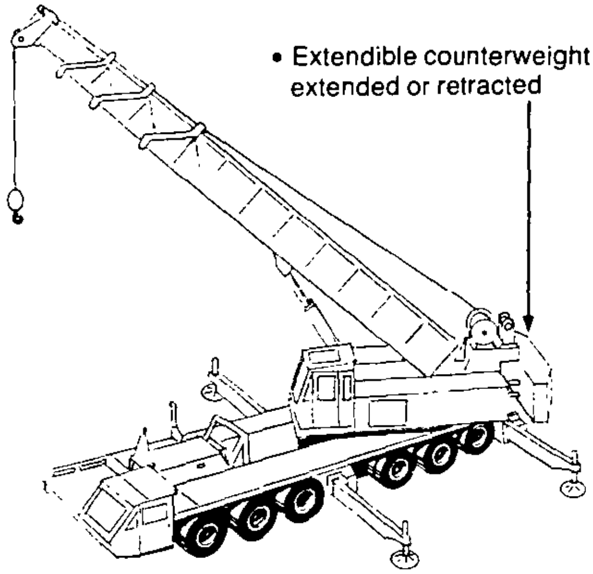
• Location of boom inserts

77H TAPERED TIP BOOM COMPOSITION						
Boom Length (In Feet)	30 Ft. 77S Inner	10 Ft. 77S Center	20 Ft. 77S Center	50 Ft. 77S Center	40 Ft. 77H Outer Base	30 Ft. 77H Tapered Tip
100'	1	–	–	–	1	1
110'	1	1	–	–	1	1
120'	1	–	1	–	1	1
130'	1	1	1	–	1	1
140'	1	–	2	–	1	1
150'	1	–	–	1	1	1
160'	1	1	–	1	1	1
170'	1	–	1	1	1	1
180'	1	1	1	1	1	1
190'	1	–	2	1	1	1
200'	1	–	–	2	1	1
210'	1	1	–	2	1	1
220'	1	–	1	2	1	1
230'	1	1	1	2	1	1
240'	1	–	2	2	1	1
250'	1	–	–	3	1	1
260'	1	1	–	3	1	1
270'	1	–	1	3	1	1
280'	1	1	1	3	1	1
290'	1	–	2	3	1	1

Courtesy American Hoist & Derrick Co.



• Extendible counterweight extended or retracted



Know how your crane's configuration affects its capacity and follow the manufacturer's instructions on the load chart. The following is a typical load chart note.

Notes – lifting crane capacities

• 54'' x 60'' (1 37 x 1 52 m) boom with open throat top section – for lifting 220.000# (99 792 kg) with 1'' (25 40 mm) dia wire rope 8 parts of 1'' (25 45 mm) dia Type "N" wire rope required

• Lifting capacities shown are based on machine equipped with 54'' x 60'' (1 37 x 1 52 m) tubular boom with open throat top section and
  a With a boom live mast 1¼'' (31 75 mm) dia pendants, and boom midpoint suspension pendants, for boom lengths up to and including 230 (70 10 m) (Boom live mast may be used throughout entire range of boom lengths but must be used as follows with booms 50' (15 24 m) long and over when jib is used, with all boom lengths over 110') (33 53 m) when ctwt "A" is used with all boom lengths when ctwt "AB" is used Midpoint suspension pendants must be used on boom lengths over 150' (45 72 m)
  b Without boom live mast and 1½'' (38 10 mm) dia pendants for boom lengths up to and including 110' (33 53 m) **Note**: 1¼'' (31 75 mm) dia pendants must not be used unless machine is equipped with boom live mast

• When machine is equipped with boom live mast and 1½'' (38 10 mm) dia pendants instead of 1¼'' (31 75 mm) dia pendants reduce all capacities shown by 400# (181 kg)

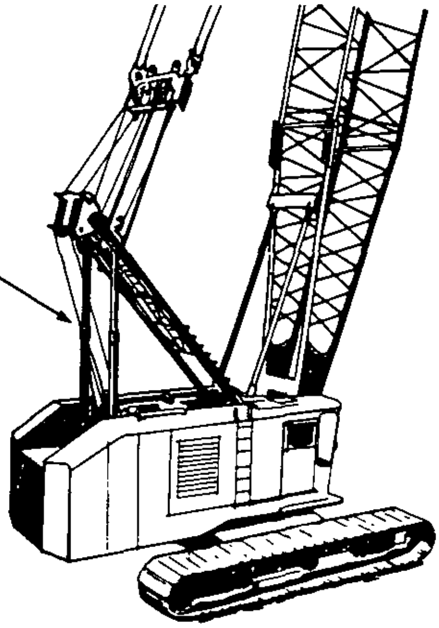
• When using 30'0'' (9 14 m) boom live mast as short boom, maximum lifting capacity of the mast is 47,000# (21 319) kg) at radii from 13'0'' (3 96 m) minimum to 20 0'' (6 10 m) maximum and live mast stops in position and operative
  a For lifting 47,000# (21 319 kg) on boom live mast with ¾' (19 05 mm) dia wire rope 4 parts of ¾'' (19 05 mm) Type "N" wire rope are required
  b Boom live mast may be used as a short boom for machine assembly/ disassembly only. Boom live mast is not to be used for general lift crane service

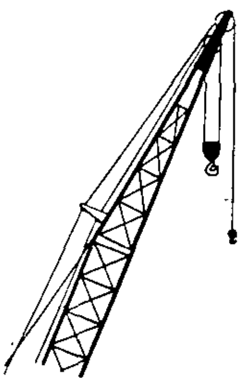
Courtesy FMC Corp.

4.3 CONFIGURATION OF CRANE AND BOOM

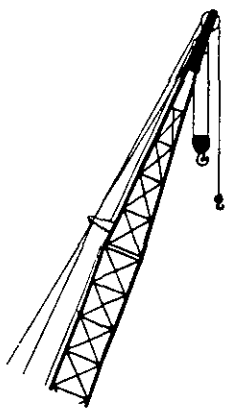
The actual configuration of the crane and the boom can affect the lifting capacity. Some of the more common factors to consider include:

- Backhitch extended or retracted
- Live mast installed
- Pendant size
- Boom type (angle, tubular, tower, heavy duty, "highlite")
- Type of boom tip installed

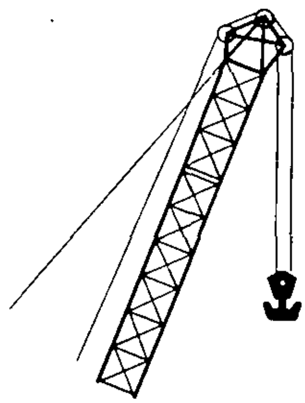




Offset Tip

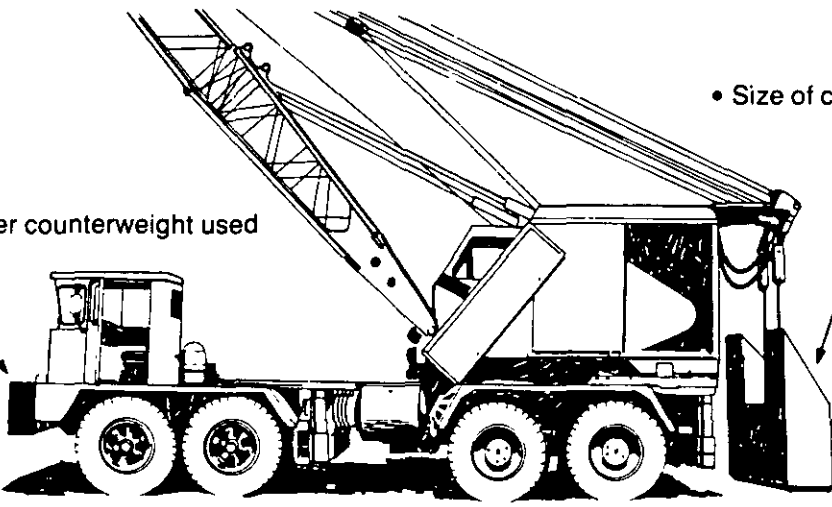


Tapered Tip



Hammerhead Tip

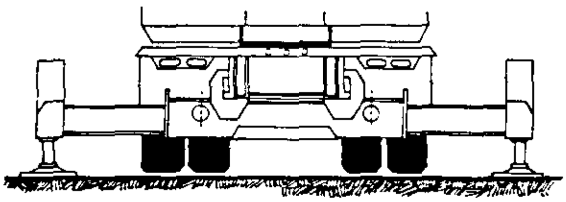
- Size of counterweight
- Front bumper counterweight used

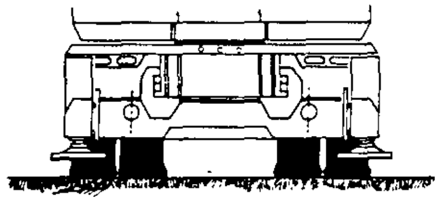


4.2 CONFIGURATION OF CRANE BASE

The capacities listed in the load chart depend on the crane's base (how it is set up) as follows:

- **Carrier-Mounted Cranes** (including lattice boom, hydaulic boom and rough terrain units)

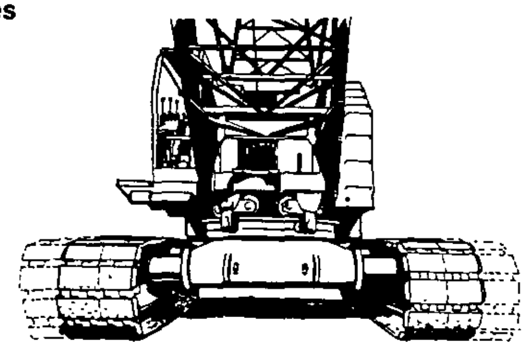




The base will be either:

"ON OUTRIGGERS"	OR	"ON RUBBER"

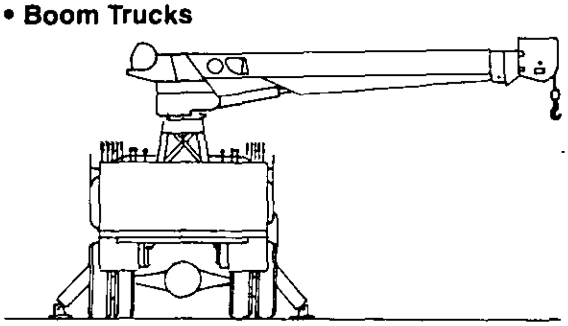
- **Crawler-Mounted Cranes**

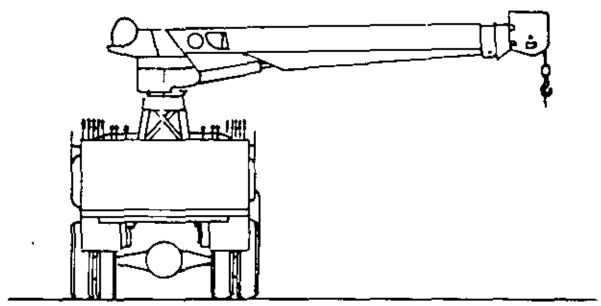


The base will be either:

CRAWLERS RETRACTED	OR	CRAWLERS EXTENDED

- **Boom Trucks**





The base will be either:

"ON STABILIZERS"	OR	"ON RUBBER"

4.1 LOAD CHARTS

The crane's load chart specifies the rated (maximum) capacity of the machine for every permissible configuration and situation. The load chart also specifies the machine's operational limitations and the conditions necessary for safe operation.

With so much information provided, the load chart can be difficult to understand. Chapters 4, 5, 6 and 7 are intended to assist in the understanding, use and application of load charts.

The ability to understand and correctly use the machine's load chart is critical to the safe operation of a mobile crane. When operators are unable to do so they rely on guesswork and the highly dangerous practice of attempting lifts and relying on signs of tipping to warn of overload.

Never use signs of tipping to determine capacity limits.

- A crane can be overloaded before any signs of tipping are evident.
- A crane can be overstressed or can fail structurally before tipping occurs.
- A crane may go from a stable to an unstable condition with no marked change in the operator's perception of machine condition.
- Once tipping starts, it may happen so quickly that recovery is impossible. The only recourse is to cut the load loose but this will not always work, particularly when multiple-part hoist lines are reeved.

Use the load chart to determine capacity.

The load charts of all mobile cranes are based on the configuration of the crane at the time of the lift. These configurations start with one of three basic situations:

1) Jibs and boom extensions not installed. (Covered in Chapter 5)
2) Jib and/or boom extensions installed but load lifted from main boom. (Covered in Chapter 6)
3) Jibs and/or boom extensions installed and load lifted from jib or boom extension. (Covered in Chapter 7)

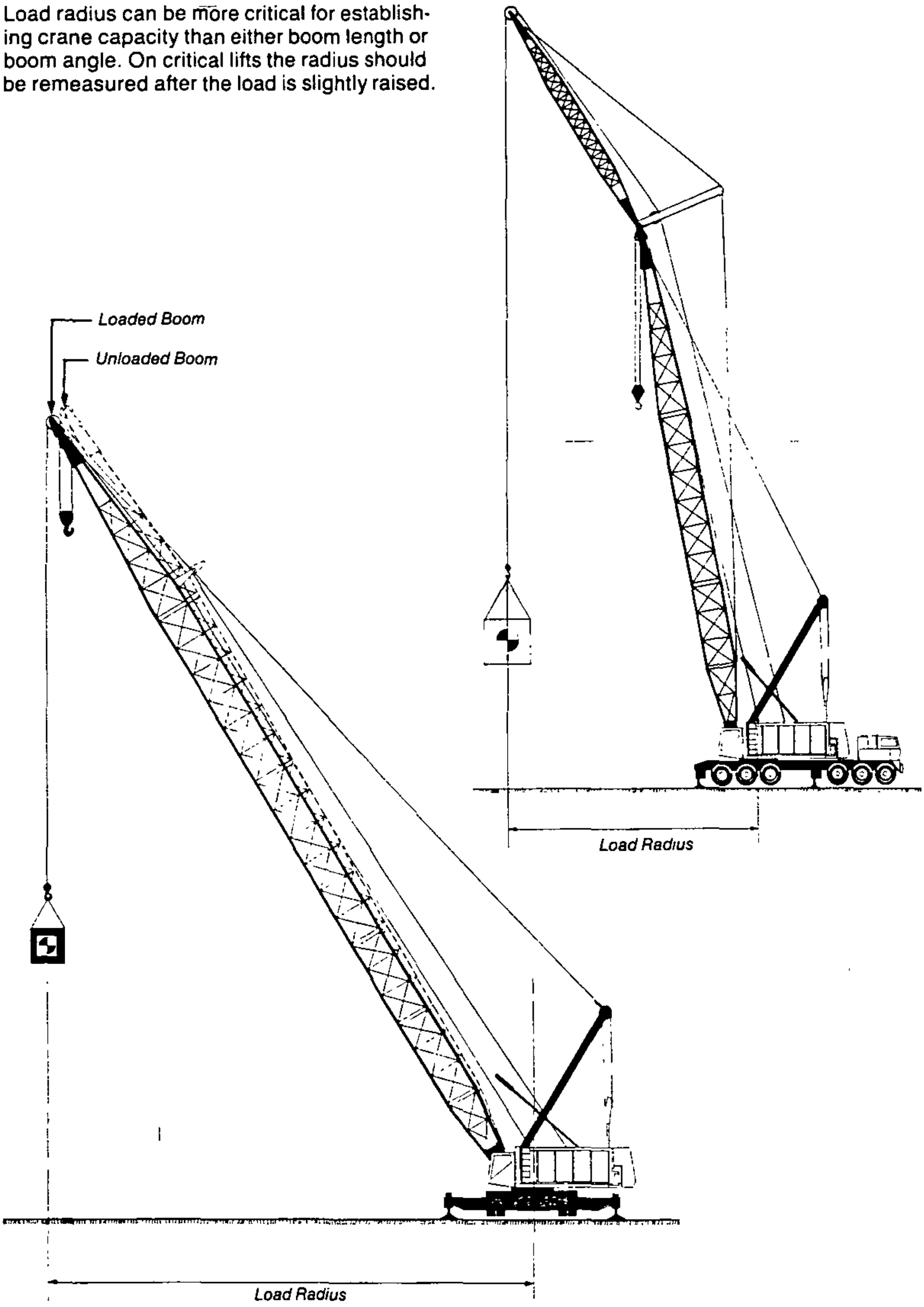
All other aspects of the crane's configuration are considered in relation to these three basic cases.

The principal factors influencing the machine's capacity and how the chart is read include:

- Geometry and configuration of crane base
- Configuration of crane
- Quadrant(s) of operation
- Boom length
- Boom angle
- Load radius
- Deductions from gross capacity.

The meaning of each term and the value of each for every lift must be known in order to determine the net capacity of the crane.

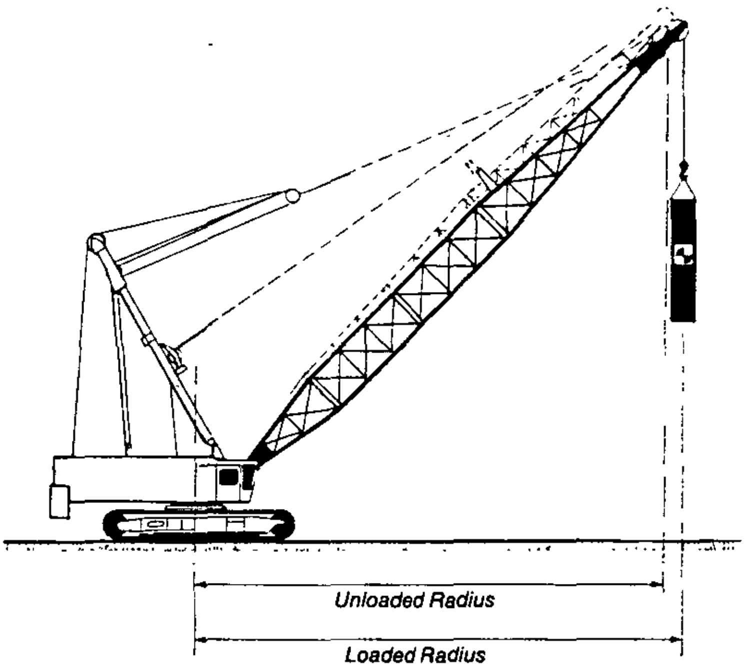
Load radius can be more critical for establishing crane capacity than either boom length or boom angle. On critical lifts the radius should be remeasured after the load is slightly raised.



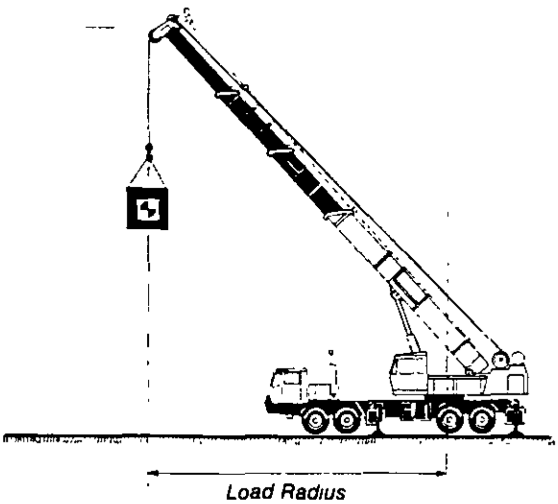
4.9 LOAD RADIUS

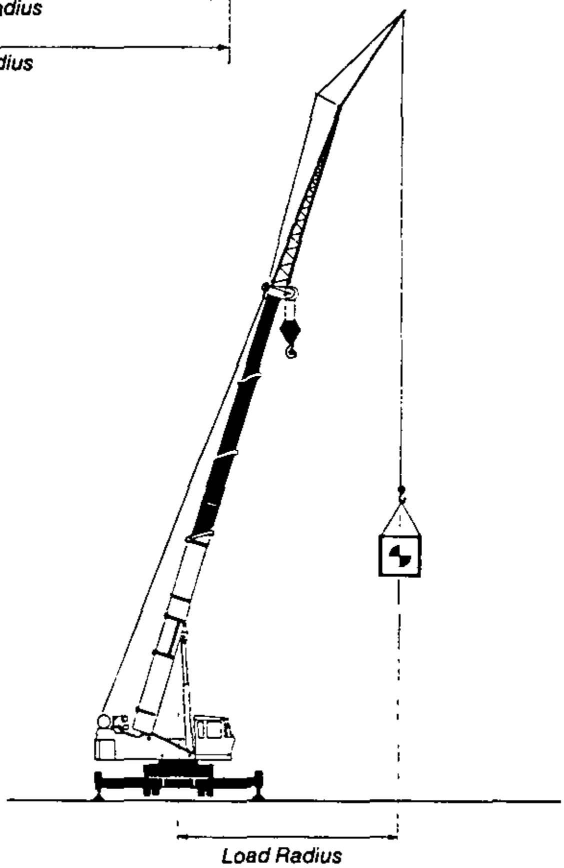
The capacities listed in the load chart also depend on and vary with the crane's load radius.

The load radius is the horizontal distance measured from the center of rotation of the crane (center pin) to the load hook (center of gravity of the load) *while the boom is loaded.*



Because of boom and machine deflection and pendant stretch, expect the load radius to increase when the load is lifted off the ground. Expect even larger increases in radius when the crane is "on rubber" because of tire deflection.

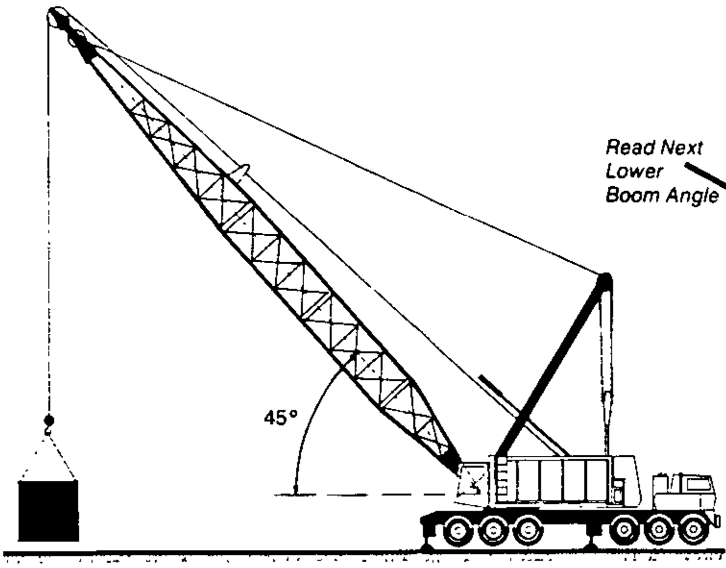




4.8 BOOM ANGLE BETWEEN CHART LISTINGS

If the actual boom angle falls between the values listed in the load chart use the gross capacity rating for the *next lower boom angle* listed in the chart.

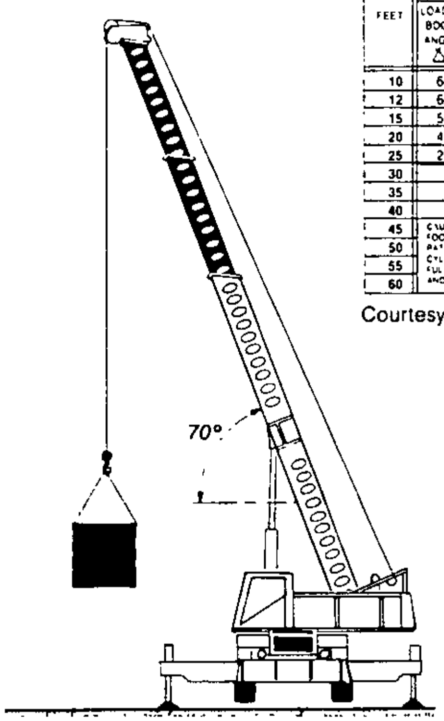
Don't interpolate the capacity between the chart listings. Use the value for the next lower boom angle.



Read Next Lower Boom Angle

Boom Length In Feet	Radius In Feet	Boom Angle Degrees	Outriggers Free: Over Side	Outriggers Free: Over Rear	Outriggers Extended and Set: Over Side	Outriggers Extended and Set: Over Rear	Ft. From Boom Pt. to Ground
100 feet	21	80 8	-	129970	222850	222850	106
	25	78 5	-	106560	222850	222850	106
	30	75 6	-	86120	198730	198730	104
	35	72 6	-	71980	163860	166360	103
	40	69 6	-	61610	134130	142790	101
	50	63 3	41680	47400	97810	110750	97
	60	56 7	34100	38120	76440	89980	91
	[illegible]	[illegible]	28530	31570	62350	75410	84
	80	41 5	24260	26700	52360	64610	74
	90	31.9	20860	22910	44880	55710	60
	100	18 3	18080	19870	39060	48610	39

Courtesy American Hoist & Derrick Co.



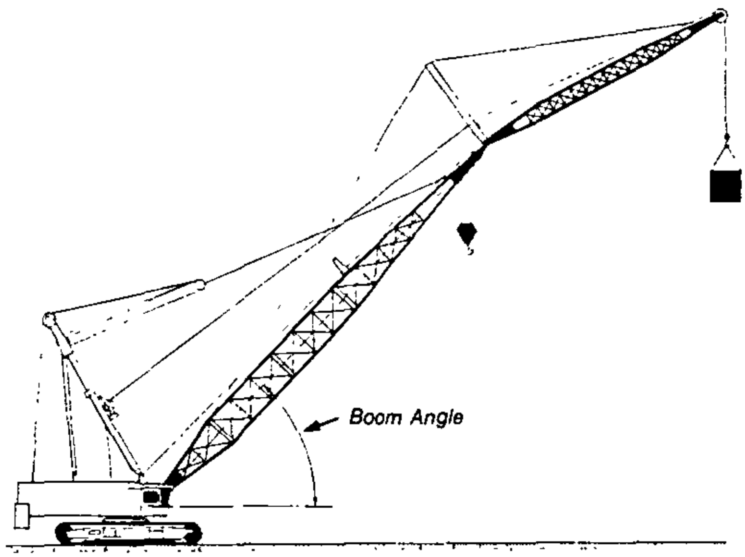
POWERED BOOM LENGTH IN FEET – MANUAL SECTION RETRACTED, BOTH POWERED SECTIONS EXTENDED EQUALLY

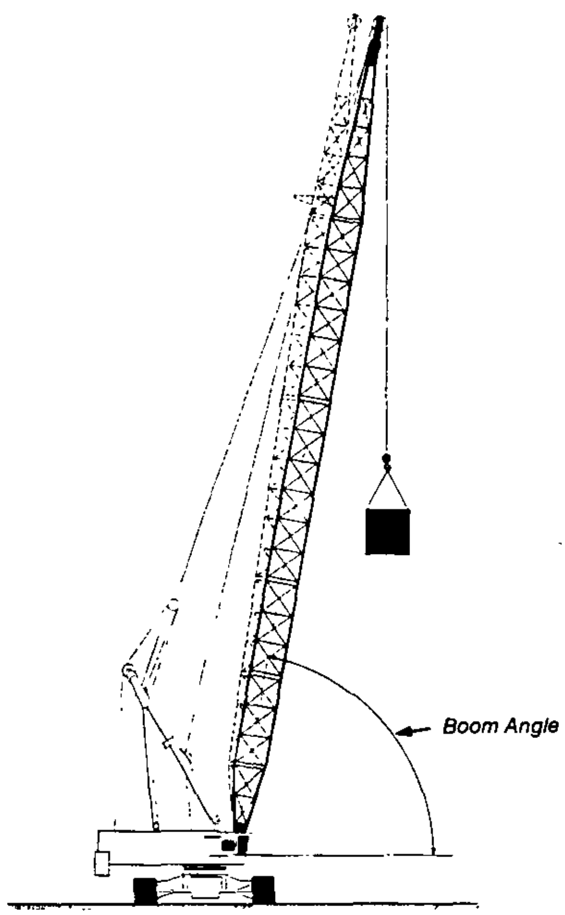
LOAD RADIUS IN FEET	32 5' Loaded Boom Angle ∠°	32 5' Rated Load Pounds 360°	32 5' Rated Load Pounds Front	40' Loaded Boom Angle ∠°	40' Rated Load Pounds 360°	40' Rated Load Pounds Front	48' Loaded Boom Angle ∠°	48' Rated Load Pounds 360°	48' Rated Load Pounds Front	56 Loaded Boom Angle ∠°	56 Rated Load Pounds 360°	56 Rated Load Pounds Front	64 Loaded Boom Angle ∠°	64 Rated Load Pounds 360°	64 Rated Load Pounds Front
10	64	80000	80000	70	70000	70000	73	67000	67000	76	62000	62000			
12	60	72000	72000	66	64000	64000	71	61000	61000	74	57000	57000	76	50000	50000
15	54	60000	60000	62	57000	57000	67	54000	54000	[illegible]	51000	51000	73	44000	44000
20	41	47000	47000	53	45000	45000	60	43000	43000	65	40000	40000	69	37000	37000
25	23	36000	36000	43	35000	35000	53	34000	34000	[illegible]	33000	33000	64	29500	29500
30				30	29000	29000	44	28000	[illegible]	53	27000	27000	58	25000	25000
35							35	23000	[illegible]	46	23000	23000	53	22000	22000
40							21	17700	[illegible]	38	17700	19200	47	17700	19000
45	CAUTION FOR 32 5 FOOT BOOM LENGTH RATINGS TELESCOPE CYLINDERS MUST BE FULLY RETRACTED AND AGAINST STOPS									28	13800	15100	40	13800	15100
50										9	11100	12100	32	11100	12100
55													21	9000	9900
60															

Courtesy Harnischfeger Corp.

Read Next Lower Boom Angle

On lattice boom cranes the boom angle is the angle between the center line of the boom (from the boom foot pins of the main boom to the boom tip sheave) and the horizontal *while the boom is under load*.

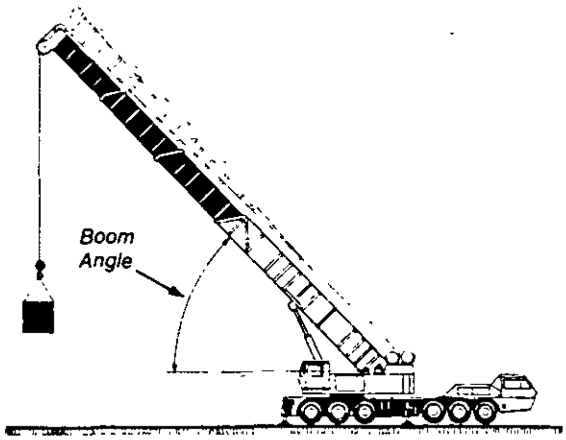


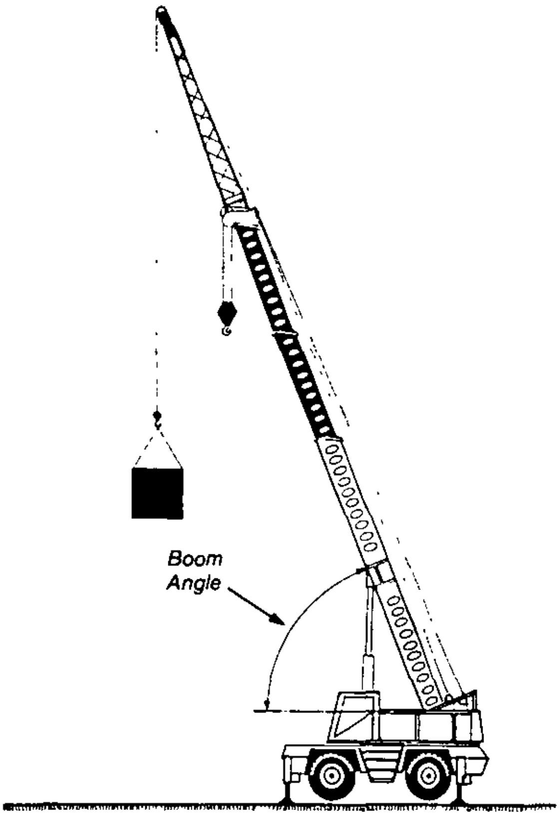


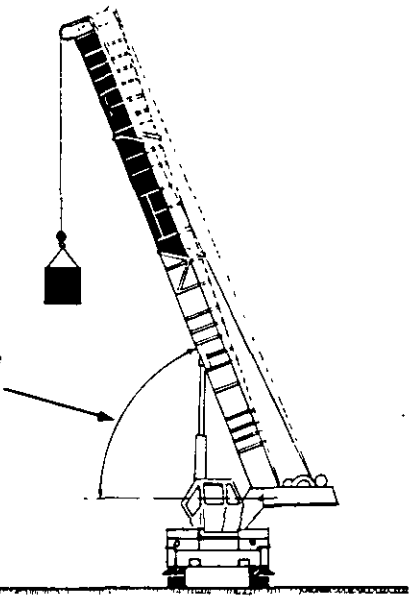
4.7 BOOM ANGLE

The capacities listed in the load chart are also based on and vary with the boom angle of the machine.

On telescopic boom cranes the boom angle is the angle between the base (bottom) of the heel section of the main boom and the horizontal *while the boom is under load.*





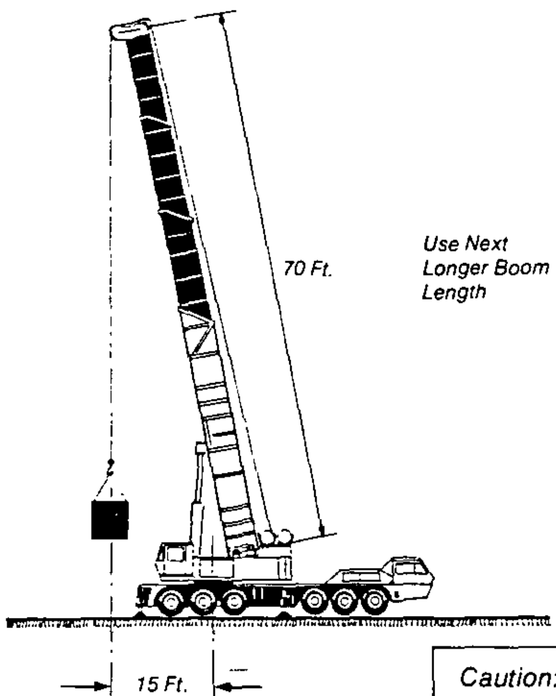


Because of boom and machine deflection (and pendant stretch on lattice booms) expect the boom angle to lower somewhat from its unloaded condition once a load is applied. Expect even larger boom angle reductions when the crane is "on rubber" because of tire deflection.

4.6 BOOM LENGTH BETWEEN CHART LISTINGS

Unless otherwise indicated, if the actual boom length falls between the values listed in the load chart, use the gross capacity rating for the *next longer boom length* listed in the chart.

Do not interpolate between the readings. Use the longer.

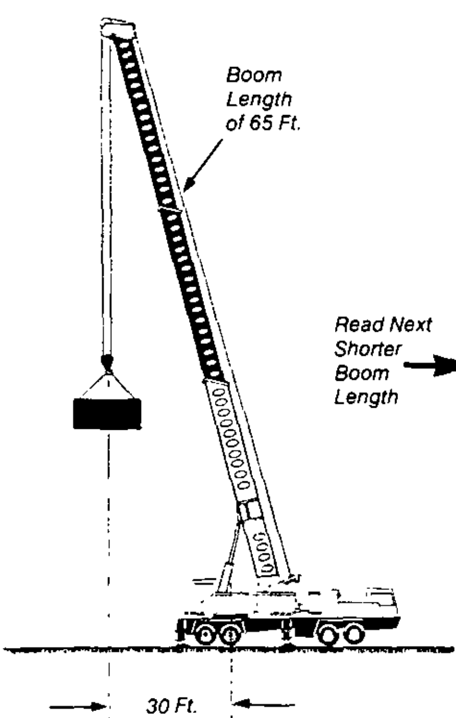


Use Next Longer Boom Length

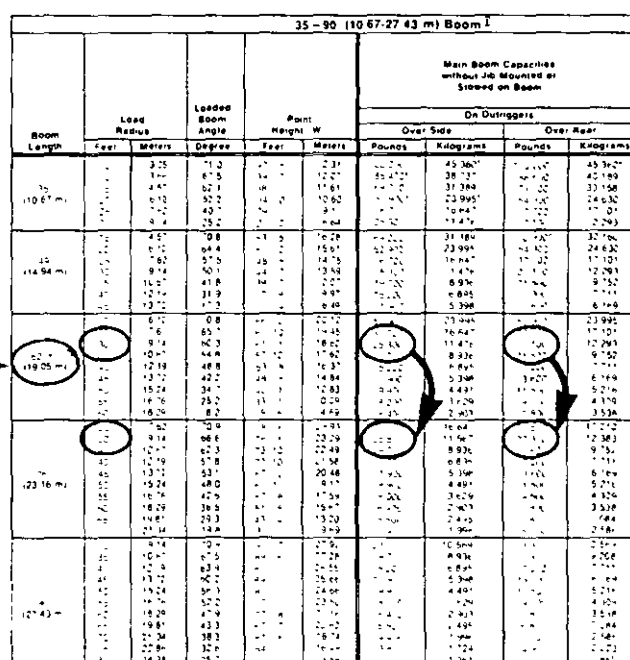
ON OUTRIGGERS FULLY EXTENDED - OVER REAR											
Radius in Feet	Main Boom Length in Feet (Power Pinned Fly Retracted)								[illegible]	[illegible]	[illegible]
	36	44	52	60	68	76	82	87	114	119	146
10	130 000 (67)	106 700 (71.5)	101 600 (74.5)	100 000 (77)	96 [illegible]						
12	123 500 (63)	106 700 (68.5)	101 600 (72)	96 500 ([illegible])	[illegible] 850 (77)	84 700 ([illegible])			See Warning Note [illegible]	See Warning Note [illegible]	See Warning Note [illegible]
15	105 000 (57.5)	105 000 (64)	95 300 ([illegible])	84 900 (72)	79 1[illegible] (74.[illegible])	77 550 (76)	70 250 (77.5)	64 500 (79)			
20	78 8[illegible]0 (47)	78 350 (56[illegible])	[illegible] 850 (62.5)	70 5[illegible] ([illegible])	[illegible] (70)	63 800 ([illegible])	59 400 (74)	55 000 (75.5)	38 750 (80)	23 600 (79.5)	
25	60 000 (34)	[illegible]	[illegible] (55.5)	60 000 (61)	54 000 (65.5)	49 700 (67.5)	47,150 (70.5)	45 600 (72)	34 000 (77)	21 300 (77)	22 500 (80)
30		47 900 (38)	47 900 (48.5)	47 900 (55.5)	46 650 (60.5)	42 750 (63.5)	40 450 (66.5)	39 150 (68.5)	30 300 (74.5)	19 500 (74.5)	21 700 (78.5)
35		38 800 (24.5)	38 800 (40.5)	38 800 (49.5)	38 800 (55.5)	37 300 (58.5)	35 200 (62.5)	34 050 (65)	27 250 (71.5)	17 950 (72)	18 300 (76.5)
40			30 080 (30.5)	30 080 (42.5)	30 080 (50)	30 080 (54)	30 080 (58.5)	29 550 (61.5)	24 750 (69)	16 600 (69.5)	16 000 (74.5)
45	See Warning Note [illegible]		23 970 (14.5)	23 970 (34.5)	23 970 (44)	23 970 (49)	23 970 (54)	23 970 (57.5)	22 650 (66)	15,500 (66.5)	14 620 (72.5)
50				19 470 (24)	19 470 (37.5)	19 470 (43.5)	19 470 (49.5)	19 470 (53.5)	20 800 (63)	14 500 (64)	13 730 (70)
60					13 200 (17.5)	13 200 (30.5)	13 200 (39)	13 200 (44)	15 650 (57)	12 850 (58.5)	11 450 (66)
70							9 110 (24.5)	9 110 (33)	11 590 (50.5)	11 550 (52.5)	9 540 (61.5)
80								6 220 (14)	8 430 (43)	9 030 (46)	8 090 (56.5)
90									6 000 (34.5)	6 470 (38.5)	7 080 (51.5)
100									4 050 (23)	4 520 (29.5)	5 660 (46)
110										2 870 (15.5)	4 080 (40)
120											2 820 (33)
130											1 720 (24)
Min. boom angle (deg.) for indicated length (no load)								0	0	0	0
Max. boom length (ft.) at 0 degree boom angle (no load)								87	114	119	146

Courtesy Grove Manufacturing Co.

> *Caution:* The gross capacity rating for the next longer boom might be higher (on some cranes) than it is for the shorter boom. In these cases use the gross capacity rating for the next *shorter* boom length listed in the chart.



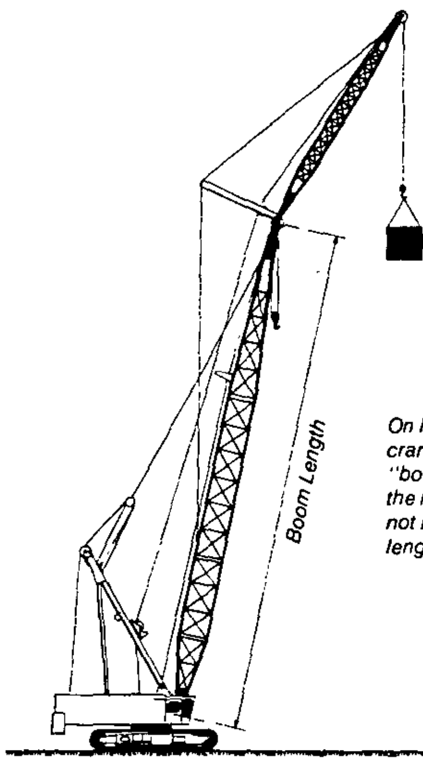
Read Next Shorter Boom Length



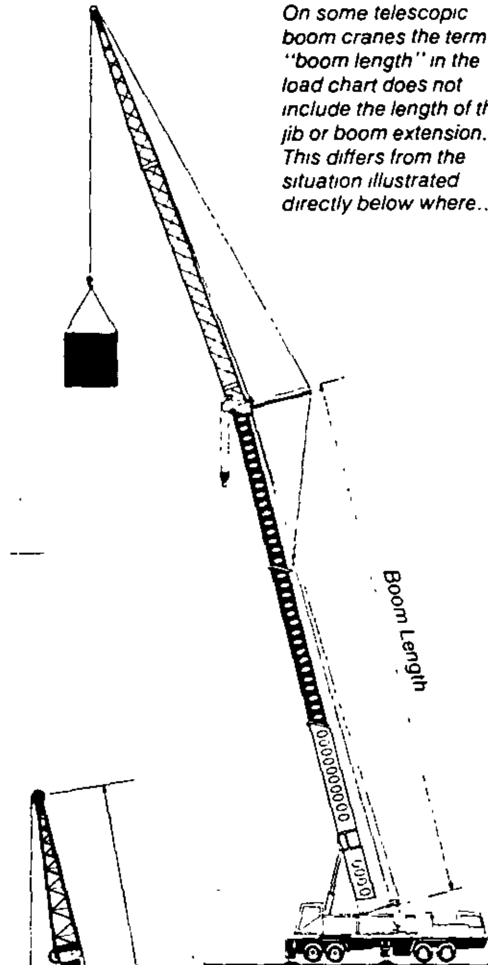
Note Capacity Increase

Courtesy FMC Corp.

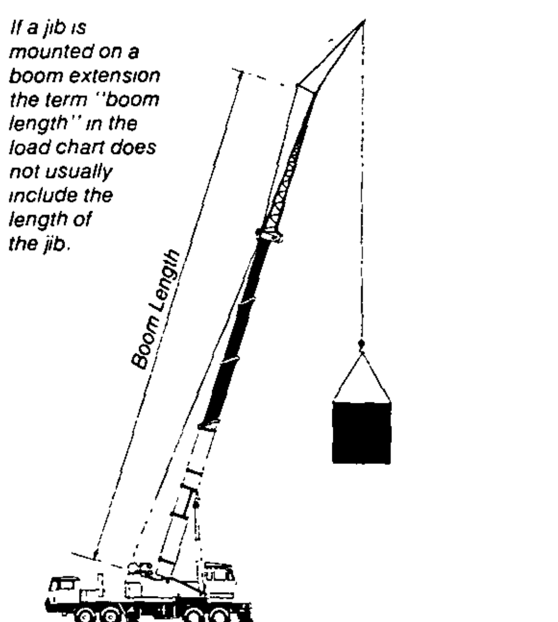
When lifting from a jib and/or boom extension the operator will have to be very careful to understand exactly what the manufacturer means by "boom length" because it may not be the overall length of the boom plus jib or boom extension.



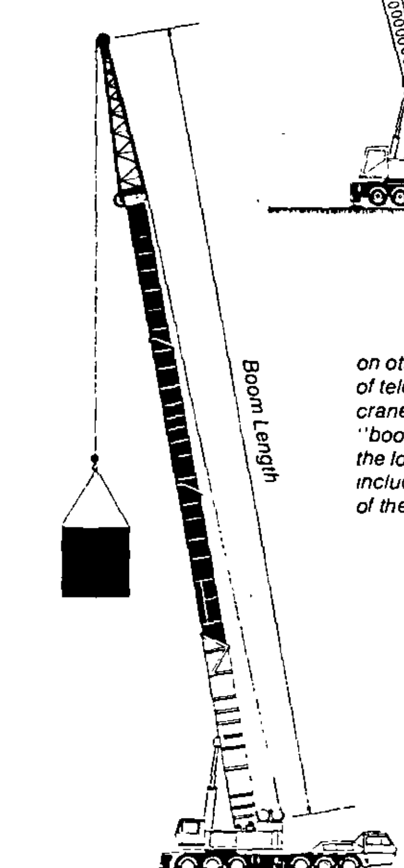
On lattice boom cranes the term "boom length" in the load chart does not include the length of the jib.



On some telescopic boom cranes the term "boom length" in the load chart does not include the length of the jib or boom extension. This differs from the situation illustrated directly below where...

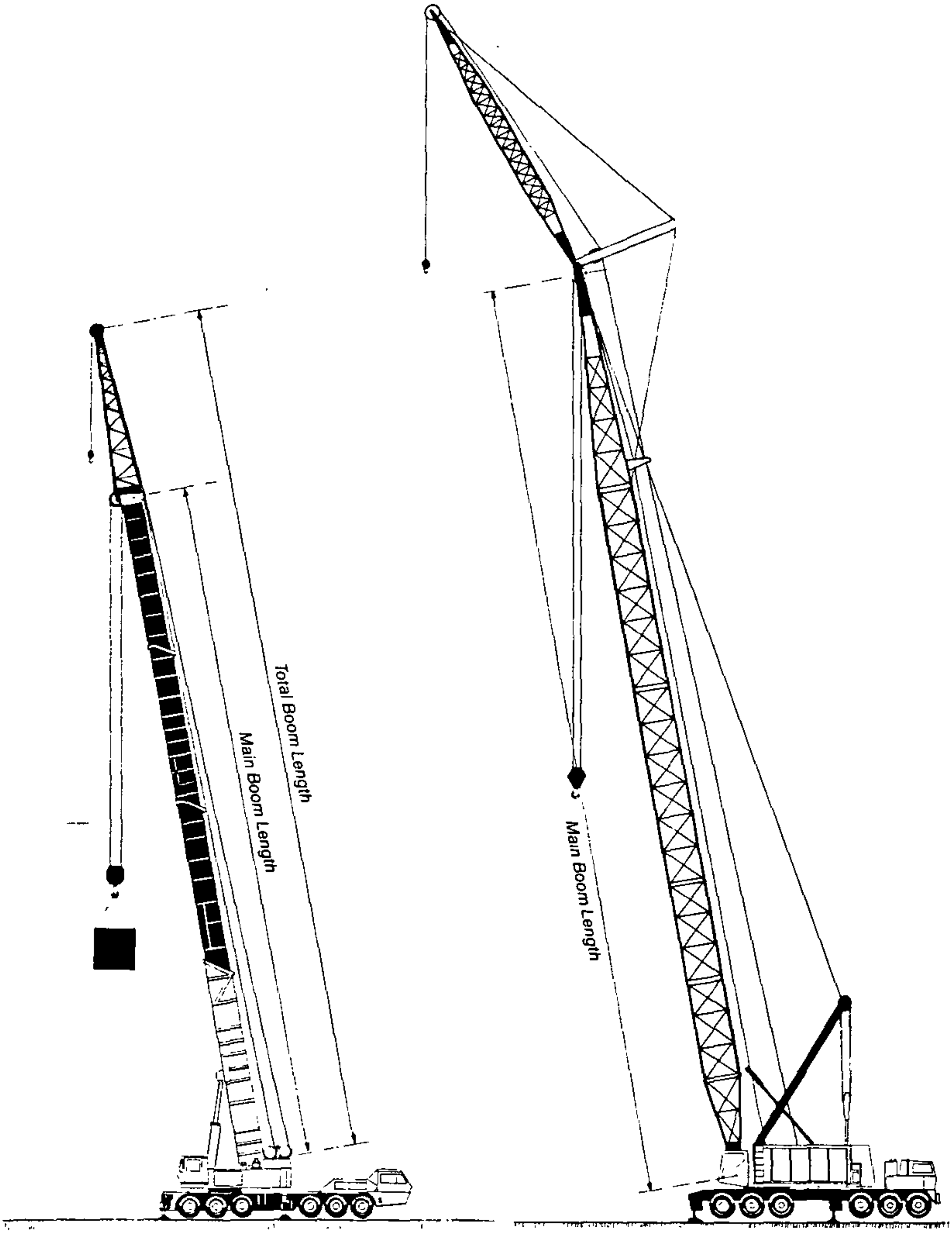


If a jib is mounted on a boom extension the term "boom length" in the load chart does not usually include the length of the jib.



on other makes of telescopic boom cranes the term "boom length" in the load chart includes the length of the boom extension.

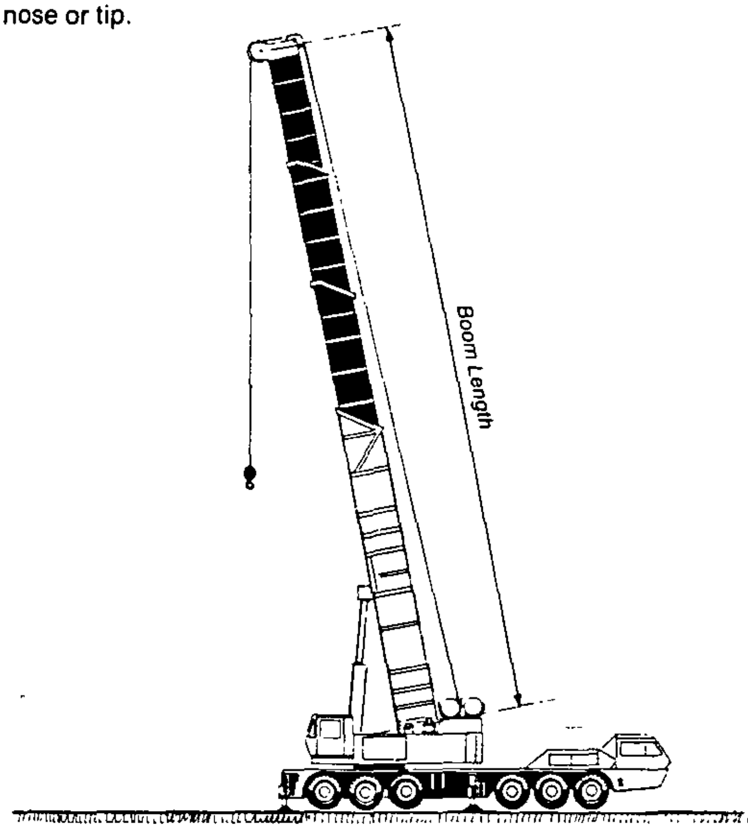
If a jib or boom extension is mounted on the crane but is not being used, the load chart capacity depends on the main boom length, not the total boom length.

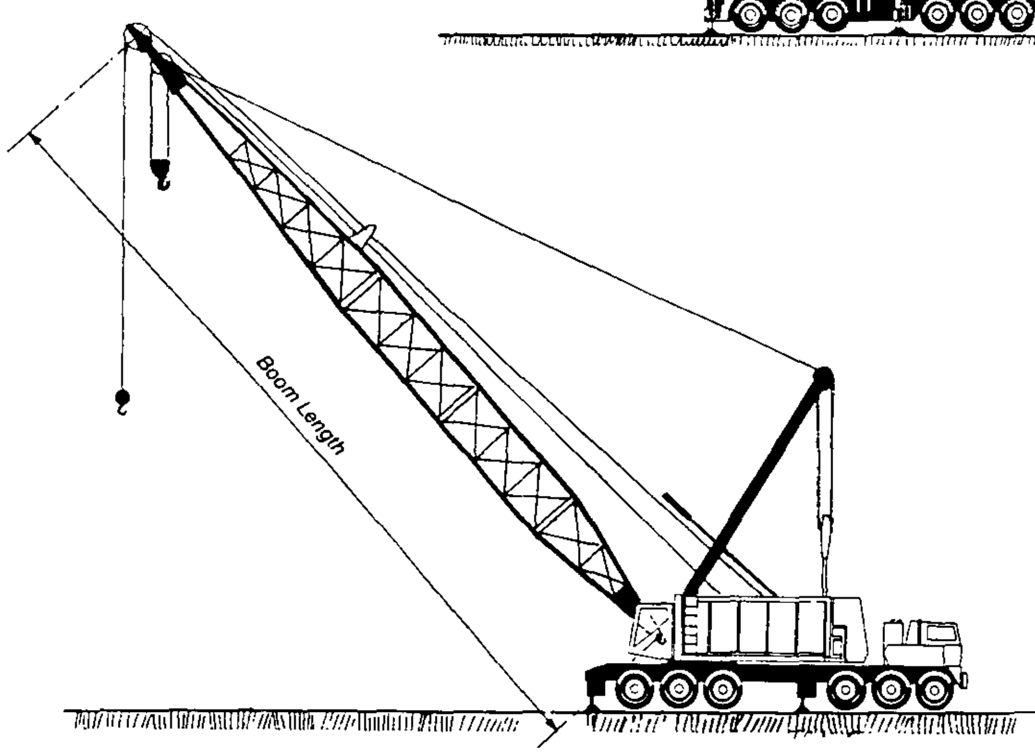


4.5 BOOM LENGTH

The capacities listed in the load chart depend on and vary with the crane's boom length.

The boom length is the distance measured from the boom foot pins or hinge pins to the center of the sheave on the boom nose or tip.

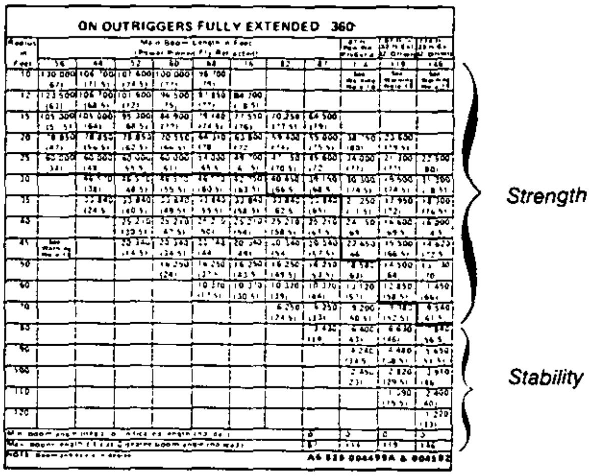




MAIN BOOM LOAD CHARTS

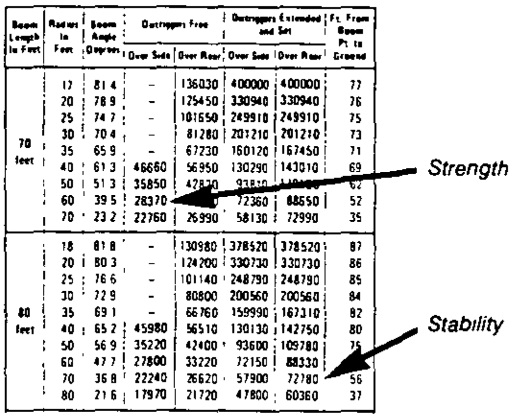
All mobile crane *main boom load charts* indicate where strength factors apply and where stability factors apply. This is generally done by dividing the chart with a heavy line, or by using asterisks or shaded areas.

SOLID LINE



Courtesy Grove Manufacturing Co.

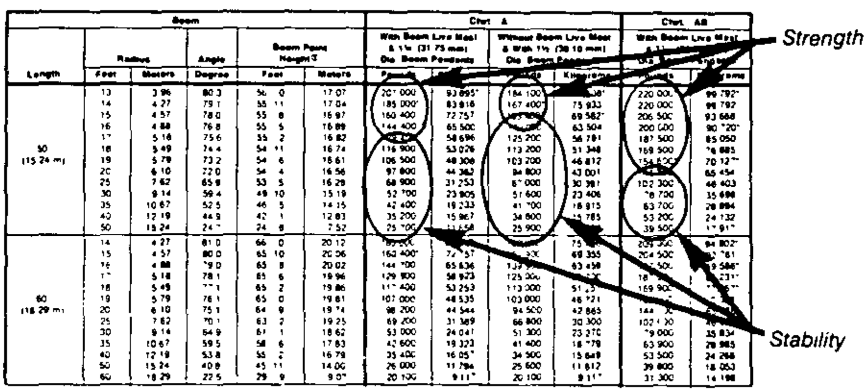
SHADED AREAS



Boom Length In Feet	Radius In Feet	Boom Angle Degrees	Outriggers Free: Over Side	Outriggers Free: Over Rear	Outriggers Extended and Set: Over Side	Outriggers Extended and Set: Over Rear	Ft. From Boom Pt. to Ground
70 feet	17	81 4	–	136030	400000	400000	77
	20	78 9	–	125450	330940	330940	76
	25	74 7	–	101650	249910	249910	75
	30	70 4	–	81280	207210	201270	73
	35	65 9	–	67230	160120	167450	71
	40	61 3	46660	56950	130290	143010	69
	50	51 3	35850	42870	93810	[illegible]	62
	60	39 5	28370	[illegible]	72360	88550	52
	70	23 2	22760	26990	58130	72990	35
80 feet	18	81 8	–	130980	378520	378520	87
	20	80 3	–	124200	330730	330730	86
	25	76 6	–	101140	248790	248790	85
	30	72 9	–	80800	200560	200560	84
	35	69 1	–	66760	159990	167310	82
	40	65 2	45980	56510	130130	142750	80
	50	56 9	35220	42400	93600	109780	75
	60	47 7	27800	33220	72150	88330	[illegible]
	70	36 8	22240	26620	57900	72780	56
	80	21 6	17970	21720	47800	60360	37

Courtesy American Hoist & Derrick Co.

ASTERISKS



Courtesy FMC Corp.

INTEGRATED JIB LOAD CHARTS

Some jib load charts (like the main boom load charts) show by means of a heavy line, asterisks or shaded areas where the jib capacities are limited by strength considerations and where they are limited by stability.

Jib load charts containing both strength and stability ratings are called "integrated" charts.

SOLID LINE

BOOM ANGLE	JIB OFFSET 0°	15°	30°
70°	9,000	6 000	4,400
65°	7 750	5,600	4 150
60°	6,900	5 200	3 950
55°	6,300	4 950	3 800
50°	5 800	4,700	3 700
45°	4,820	4,310	3 600
40°	4,080	3,700	3,520
35°	3,520	3,240	3,130
30°	3,090	2,890	
25°	2,760	2,620	

Ratings Above Line Based on Strength

Ratings Below Line Based on Stability

A6-829-001851

Courtesy Grove Manufacturing Co

SHADED AREAS

Boom Length	Boom Radius in Feet	40 Ft Jib 5° Offset	40 Ft Jib 15° Offset	40 Ft Jib 25° Offset	60 Ft Jib 5° Offset	60 Ft Jib 15° Offset	60 Ft Jib 25° Offset
160	34	32 000					
	35	32 000					
	38				29 710		
	40	32 000	32 000		29 530		
	50	32 000	32 000	32 000	28 550	25 770	
	60	32 000	32 000	32 000	27 880	25 110	23 080
	70	32 000	32 000	30 180	26 810	24 510	22 230
	80	32 000	32 000	28 120	25 920	23 880	20 730
	90	32 000	32 000	26 280	25 250	23 300	19 330
	100	32 000	30 590	24 750	24 500	22 720	[illegible]
	110	28 130	28 130	23 450	23 880	21 280	[illegible]
	120	24 670	24 670	22 290	23 230	19 910	16 300
	130	21 800	21 800	21 320	22 030	18 830	15 500
	140	19 360	19 360	19 370	19 900	17 800	14 830
	150	17 270	17 280	17 290	17 810	16 950	14 240
	160	15 460	15 470	15 480	16 010	16 010	13 730
180	36	32 000					
	40	32 000			29 280		
	50	32 000	32 000	32 000	28 510	25 560	
	60	32 000	32 000	32 000	27 700	25 030	22 860
	70	32 000	32 000	31 080	26 950	24 500	22 560
	80	32 000	32 000	29 000	26 140	23 980	21 220
	90	32 000	32 000	27 320	25 450	23 440	19 920
	100	31 480	31 490	25 800	24 790	22 960	18 780
	110	27 270	27 280	24 450	24 170	22 300	17 780
	120	23 810	23 820	23 270	23 620	20 950	16 890
	130	20 930	20 930	20 940	21 470	19 810	16 130
	140	18 490	18 500	18 510	19 020	18 710	15 440
	150	16 400	16 410	16 420	16 930	16 930	14 800
	160	14 590	14 590	14 600	15 120	15 120	14 260
	170	13 000	13 010	13 020	13 540	13 550	[illegible]
	180	11 620	11 620	11 640	12 150	12 150	[illegible]

Capacities in Shaded Areas Limited by Strength

Capacities in Non-Shaded Areas Limited by Stability

Courtesy American Hoist & Derrick Co.

ASTERISKS

62 6'–90' (19.05-27.43 m) Boom Plus 80' (24.38 m) Jib

Boom Length	Load Radius Feet	Load Radius Meters	Loaded Boom Angle Degree	Jib Point Height Feet	Jib Point Height Meters	On Outriggers No Jib Offset Over Side & Rear Pounds	On Outriggers No Jib Offset Over Side & Rear Kilograms	Loaded Boom Angle Degree	Jib Point Height Feet	Jib Point Height Meters	On Outriggers 7 5° Jib Offset Over Side & Rear Pounds	On Outriggers 7 5° Jib Offset Over Side & Rear Kilograms
76 (23 16 m)	50	15 24	73 5	154 6	47 09	6 800*	3 084	—	—	—	—	—
	55	16 76	71 6	152 8	46 54	6 100*	2 767*	—	—	—	—	—
	60	18 29	69 7	150 7	45 90	5 500*	2 495	73 2	150 1"	45 75	4 500	2 041*
	65	19 81	67 7	148 4	45 20	4 900	2 223	71 2	147 11"	45 08	[illegible]	1 860*
	70	21 34	65 7	145 11	44 47	4 500	2 041	[illegible]	[illegible]	[illegible]	3 800*	1 724*
	75	22 86	63 7	143 1	43 62	4 100	[illegible]	[illegible]	142 7	43 46	3 500*	[illegible]
	80	24 38	61 6	140 2	42 73	3 800*	1,724*	65 1	139 8"	42 58	3 200*	[illegible]
	85	25 91	59 5	137 0	41 76	3 500*	1 588*	63 0	136 5"	41 57	3 000*	1 361*
	90	27 43	57 3	133 6	40 69	3 200*	1 452*	60 8	132 11"	40 51	2 800*	1 270*
	95	28 96	55 0	129 7	39 50	3 000*	1 361*	58 5	129 1"	39 35	2 600*	1 179*
	100	30 48	52 7	125 6	38 25	2 800*	1 270	56 2	124 11"	38 07	2 400*	1 089*
	105	32 00	50 2	121 0	36 88	2 400	1 089	53 8	120 5"	36 70	2 300*	1 043
	110	33 53	47 7	116 1	35 39	2 100	953	51 3	115 5"	35 17	2 100	953*
	115	35 05	45 0	110 8	33 74	1 700	771	48 6	110 0	33 53	[illegible]	907*
	120	36 58	42 1	104 8	31 91	1 400	[illegible]	[illegible]	104 [illegible]	[illegible]	1 600	[illegible]
	125	38 10	39 1	98 1	29 90	1 100	499	42 7	97 4"	29 66	1 300	[illegible]
	130	—	—	—	—	—	—	39 4	89 10"	27 37	1,000	454

Capacities Limited by Strength Marked with an Asterisk

Un-marked Capacities Limited by Stability

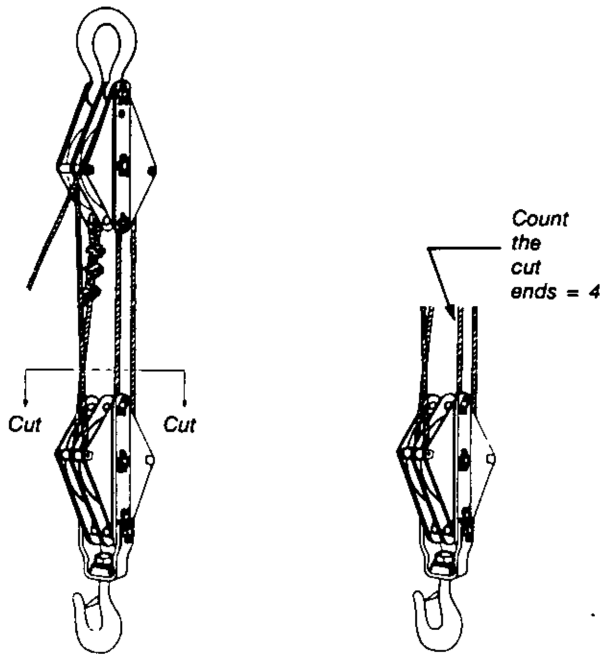
Courtesy FMC Corp

4.14 DETERMINING PARTS OF LINE, WEIGHT OF LINE AND SIZING THE HOOK BLOCK

Because hoist rope strength is not usually built into the load chart the operator must ensure before any lift is made, not only that the crane has sufficient net capacity to lift the load but also that it is rigged with enough "parts of line" to lift the load without breaking the hoist rope.

The term "parts of line" can be defined as follows:

- Imagine cutting all ropes above the hook block.
- Count the cut ends.
- This number is the "parts of line".



To calculate parts of line required:

Add: *Load Weight*
+ *Weight of Hook Block*
+ *Weight of Slings and Rigging Hardware*
= *Suspended Weight*

Divide: $\dfrac{\textit{Suspended Weight}}{\textit{Safe Working Load of Hoist Rope}}$

Answer: = *Parts of Line*

The figure calculated indicates how many parts of line are required to support the lift. This number also determines the size and weight of the hook block that must be used (because of the number of sheaves required).

Because of friction in the hook block and boom tip sheaves, extra parts of line will have to be added to ensure that each part of line is not overloaded. The following table can be used to approximate the number of parts of line to use.

PARTS OF LINE*	
CALCULATED VALUE	USE
1 or less	1 Part of Line
1 to 1.96	2 Parts of Line
1 97 to 2 88	3 " " "
2 89 to 3.77	4 " " "
3 78 to 4.63	5 " " "
4.64 to 5.43	6 " " "
5 44 to 6.22	7 " " "
6.23 to 6.97	8 " " "
6.98 to 7.68	9 " " "
7.69 to 8.37	10 " " "
8.38 to 9 02	11 " " "
9.03 to 9.65	12 " " "
9.66 to 10.25	13 " " "
10.26 to 10 83	14 " " "
10.84 to 11.37	15 " " "
11.38 to 11 90	16 " " "

*Based on 2% Sheave Friction for 180° Rope Bends. The values in this table are conservative Well maintained sheaves and blocks may have lower friction losses. It is good policy, however, to inspect and maintain both the boom tip and block sheaves regularly and allow extra parts of line to account for friction

When the size and weight of the hook block have not been determined, use the following method:

(1) Add load weight plus weight of slings and rigging hardware.

(2) Divide by maximum rated load of hoist rope. Answer is parts of line required. (See above table to account for sheave friction.)

(3) From parts of line required choose the hook block that provides sufficient sheaves. Read the weight of the hook block from note on load chart.

(4) Recheck parts of line as follows:

Add: *Load Weight*
+ Weight of Slings and Rigging Hardware
+ Weight of Hook Block
= Suspended Weight

Divide: $\dfrac{\textit{Suspended Weight}}{\textit{Rated Capacity of Hoist Rope}}$

Answer: *= Parts of Line*

Read. *Table above to account for sheave friction*

To calculate the maximum load that can be applied to the hoist line use the "Parts of Line" table, and for the number of parts of line in the right hand column use the higher number listed beside it in the left hand column. Multiply this number by the SWL (safe working load) of a single part of the line.

Example:

Crane's hook is reeved with six parts of line each having a SWL = 21,500 lbs.

Maximum Load = 5.43 x 21,500
= 116,745 lbs.

WEIGHT OF ROPE

The weight of the crane's hoist rope may have to be deducted from the gross capacity in some cases.

Typical weights of wire rope are as follows:

Rope Diameter Inches	Approx Weight Pounds Per Foot (IWRC)
1/4	.12
5/16	.18
3/8	.26
7/16	.35
1/2	.47
9/16	.60
5/8	.73
3/4	1.06
7/8	1.44
1	1.88
1-1/8	2.34
1-1/4	2.89
1-3/8	3.50
1-1/2	4.16
1-5/8	4.88
1-3/4	5.67
1-7/8	6.50
2	7.39
2-1/8	8.35
2-1/4	9.36
2-3/8	10.40
2-1/2	11.60
2-5/8	12.80
2-3/4	14.00

To calculate the weight of hoist rope:

Weight = Number of parts of line x Length of hoist line x Pounds per ft. of each part of line.

Example: If the hook block of a crane is hanging 50 ft. below the boom tip and rigged with six parts of 1 in. diameter rope, the total hoist rope weight is:

Weight = 6 x 50 x 1.88 = 564 lbs.

5.2 MAIN BOOM CAPACITY – LATTICE BOOMS

When the load is being lifted from the main boom tip, the crane's net capacity is determined as follows:

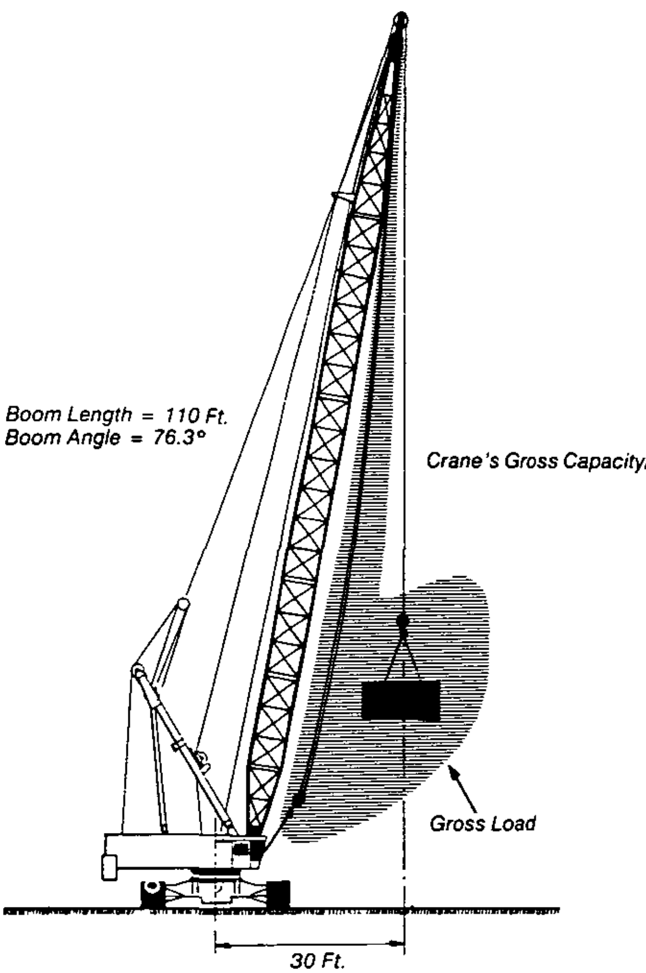
Procedure

(1) Determine weight of load to be lifted.
(2) Determine weight of slings and rigging hardware.
(3) Determine parts of line required and hook block weight.
(4) Determine load radius, boom length and boom angle as required.
(5) Select correct main boom load chart for actual crane configuration, base configuration, and quadrant(s) of operation.
(6) For correct boom length and load radius (or boom angle) read the *gross capacity* from the main boom load chart. Whenever possible, use load radius rather than boom angle as it will provide better accuracy.
(7) Determine capacity deductions.
(8) Calculate net capacity.

$$\text{Net Capacity} = \text{Gross Capacity} - \text{Capacity Deductions}$$

(9) Compare the net capacity to the load weight. If the net capacity is equal to or greater than the load, the lift can be made.

For examples of this procedure see the Bucyrus-Erie 110-T, American 9310, American 7250, Link-Belt LS-418A, Manitowoc 4100W, and American 9530 exercises in the Appendix.



Boom Lgth.: Feet	Oper. Rad.: Feet	Bm. Ang.: Deg.	Boom Point: Elev.	Capacity, Crawlers Retracted	Capacity Crawlers Extended
110	22	80.6	115.5	224,400B	268,900
	24	79.5	115.2	195,900B	232,200
	26	78.5	114.8	173,700B	204,100
	28	77.4	[illegible]	155,700	181,800
	30	76.3	113.9	141,000	163,800
	32	75.3	113.4	128,700	148,800
	34	74.2	112.8	118,300	136,300
	36	73.1	112.2	109,300	125,600
	38	72.0	111.6	101,500	116,300
	40	70.9	110.9	94,700	108,300
	45	68.1	109.1	80,800	92,000
	50	65.3	106.9	70,200	79,700
	55	62.4	104.5	61,800	70,100
	60	59.4	101.7	55,100	62,400
	65	56.3	98.5	49,500	56,000
	70	53.1	95.0	44,800	50,700
	75	49.8	91.0	40,800	46,100
	80	46.3	86.5	37,300	42,300
	85	42.6	81.4	34,300	38,900
	90	38.6	75.6	31,700	35,900
	95	34.2	68.8	29,300	33,300
	100	29.2	60.7	27,300	31,000
	105	23.3	50.6	25,400	28,900
	110	15.5	36.4	23,700	27,000

Courtesy The Manitowoc Co.

5.3 MAIN BOOM CAPACITY – FULL POWER TELESCOPIC BOOMS

The net capacity is determined as follows:

Procedure–All Boom Lengths

(1) Determine weight of load to be lifted.
(2) Determine weight of slings and rigging hardware to be used.
(3) Determine parts of line required and hook block weight.
(4) Determine load radius, boom length and boom angle.
(5) Select correct main boom load chart based on the crane's actual configuration and quadrant(s) of operation.
(6) For correct load radius, boom length and boom angle read the *gross capacity* from the main boom load chart.
(7) Determine capacity deductions.
(8) Calculate net capacity.

$$\text{Net Capacity} = \text{Gross Capacity} - \text{Capacity Deductions}$$

(9) Compare net capacity to the weight of the load to be lifted.

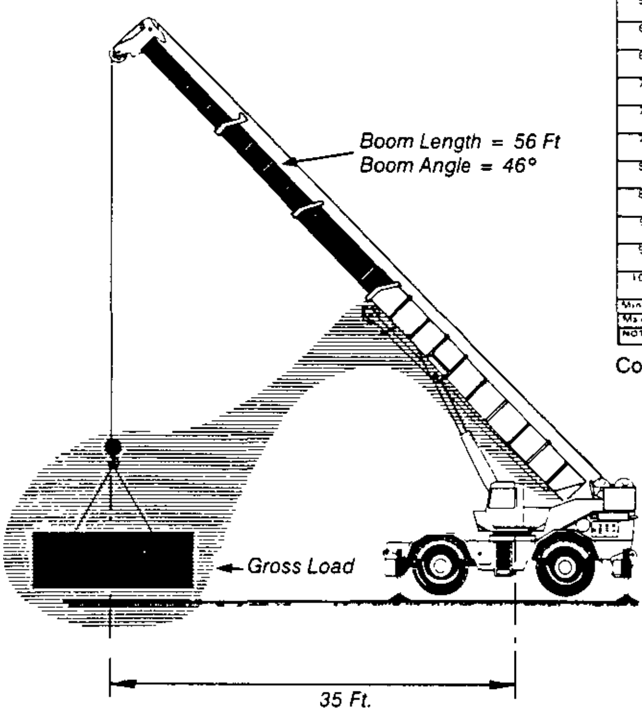
For examples of this procedure see the Link-Belt HTC-50, Grove RT522, P & H T-750 and Grove RT630 exercises in the Appendix.

Crane's Gross Capacity

ON OUTRIGGERS FULLY EXTENDED - 360°

Radius in Feet	Main Boom Length in Feet									[illegible] + 80 ft (2 Offset)
	32	38	44	50	56	[illegible]	[illegible]	[illegible]	80	106
10	60 000 (64 5)	52 000 (68 5)	51 000 (71 5)							
12	52 500 (60 5)	52 000 (65 0)	51 000 (69 0)	44 900 ([illegible])	40 500 (74 0)					
15	42 000 (54 0)	42 000 (60 0)	42 000 (64 5)	40 200 (68 0)	36 320 (70 5)	31 300 ([illegible])	26 900 ([illegible])			
20	30 000 (41 5)	30 000 (50 5)	30 000 (57 0)	30 000 (61 5)	30 300 (65 3)	24 500 (68 0)	23 000 (70 0)	[illegible]	[illegible]	
25	22 700 ([illegible])	22 700 (39 5)	22 700 (48 5)	22 700 (54 5)	22 700 ([illegible])	20 300 (62 5)	[illegible] 750 (65 5)	[illegible]	[illegible]	11 000 (75 0)
30		[illegible] (25 0)	17 000 ([illegible])	[illegible]	[illegible]	[illegible] (57 5)	16 650 ([illegible])	[illegible]	[illegible]	9 550 (73 0)
35			12 540 (26 5)	[illegible] (33 5)	12 540 (46 0)	12 540 (51 5)	[illegible] (55 5)	[illegible]	[illegible] (52 0)	8 360 (70 0)
40				9 900 (27 5)	[illegible] (38 0)	[illegible] (45 0)	[illegible] (50 5)	[illegible]	[illegible]	[illegible] 410 (67 0)
45					8 100 (25 5)	8 100 (38 0)	8 100 (44 5)	[illegible] (43 5)	[illegible]	6 630 (64 0)
50					[illegible] (13 5)	6 700 (29 5)	6 700 (36 0)	[illegible] (44 0)	[illegible] (43 5)	5 980 (61 0)
55						5 600 ([illegible])	5 600 (30 0)	5 600 ([illegible])	5 600 (43 0)	5 430 (57 5)
60							4 700 (19 5)	4 700 (20 5)	4 700 ([illegible])	4 960 (54 0)
65								3 950 (21 0)	3 950 (30 5)	4 550 (50 5)
70									3 250 (22 0)	4 090 (47 0)
74									[illegible] (11 0)	3 500 (44 0)
75										3 360 (43 0)
80										2 780 (38 5)
85										2 290 (34 0)
90										1 890 (28 5)
95										1 510 (21 5)
100										1 160 (9 5)
Min. boom angle (deg.) for indicated length (no load)									0	0
Max. boom length (ft.) at 0 degree boom angle (no load)									80 0	105 6
NOTE: Boom angles are in degrees										A6 829 004842 A 003343C

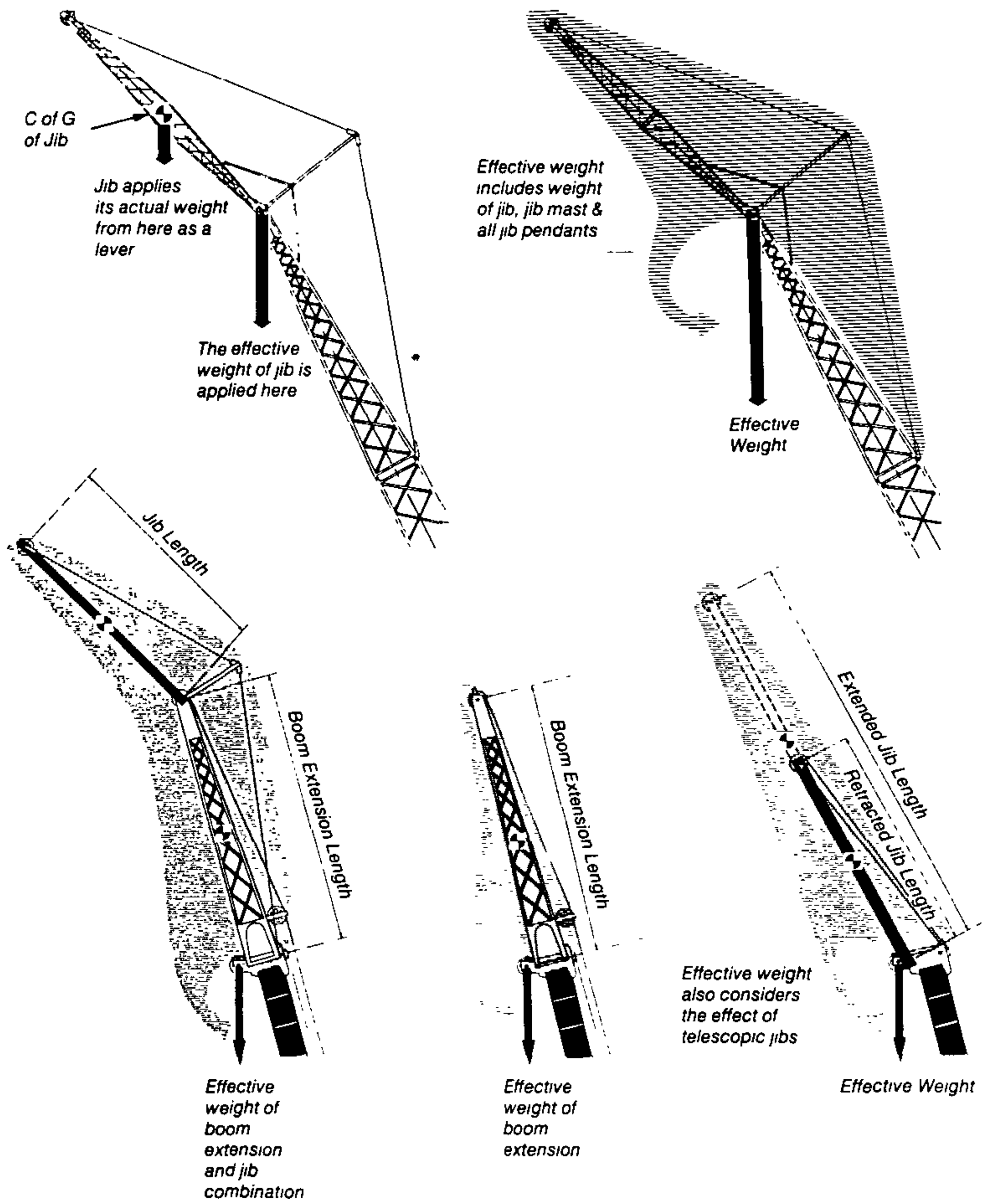
Courtesy Grove Manufacturing Co



6.1 CONTINUED

Effective weight is *not* the actual weight of the boom extension or jib, nor is it the weight deducted when the boom extension or jib is stowed. It is calculated by the manufacturer as the weight which when applied at the *boom tip* will have the same effect on the crane as the boom extension and/or jib.

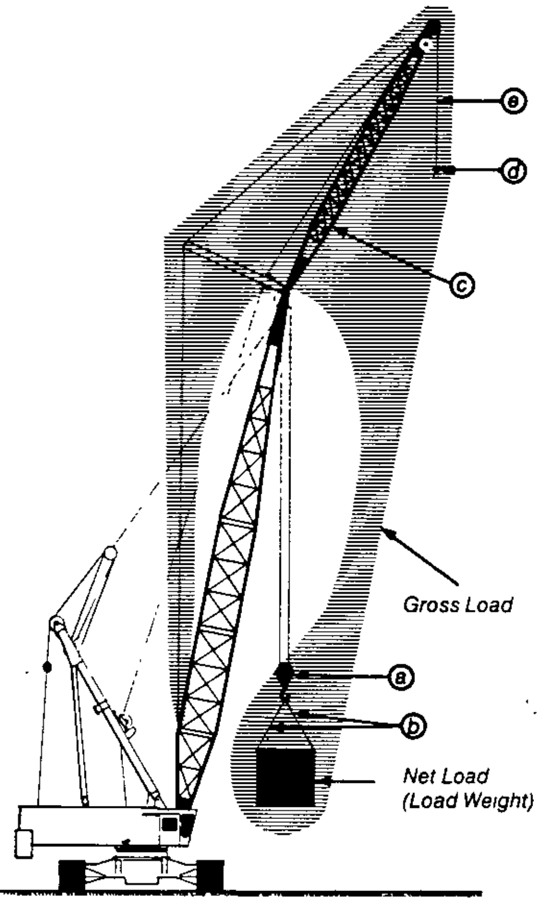
Note: The effective weight of the jib is not deducted when the hoisting is done from the jib itself (see Chapter 7).



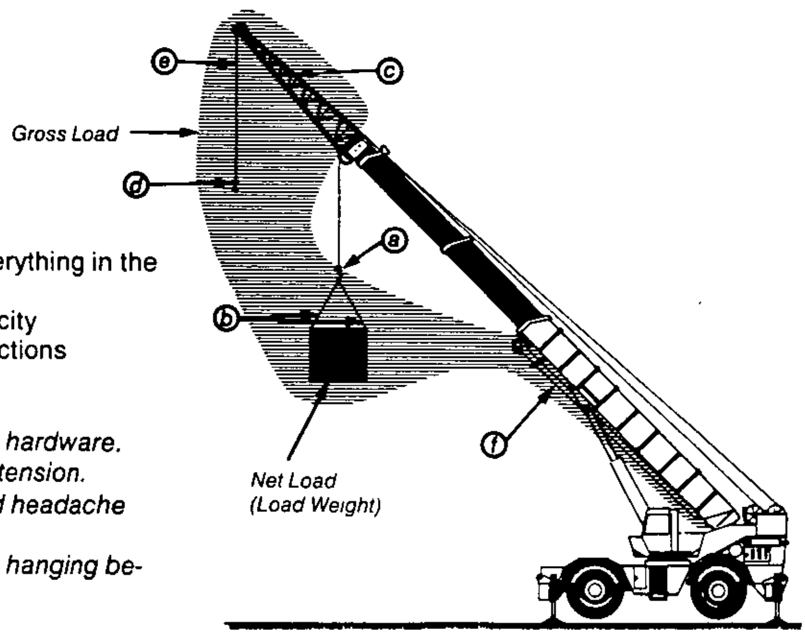
6.2 CAPACITY DEDUCTIONS

When a jib is mounted but not used, the capacity deductions that *must be subtracted* from the load chart ratings (gross capacity) to calculate the crane's net capacity differ from the case where the jib is not mounted. The most common load deductions are as follows:

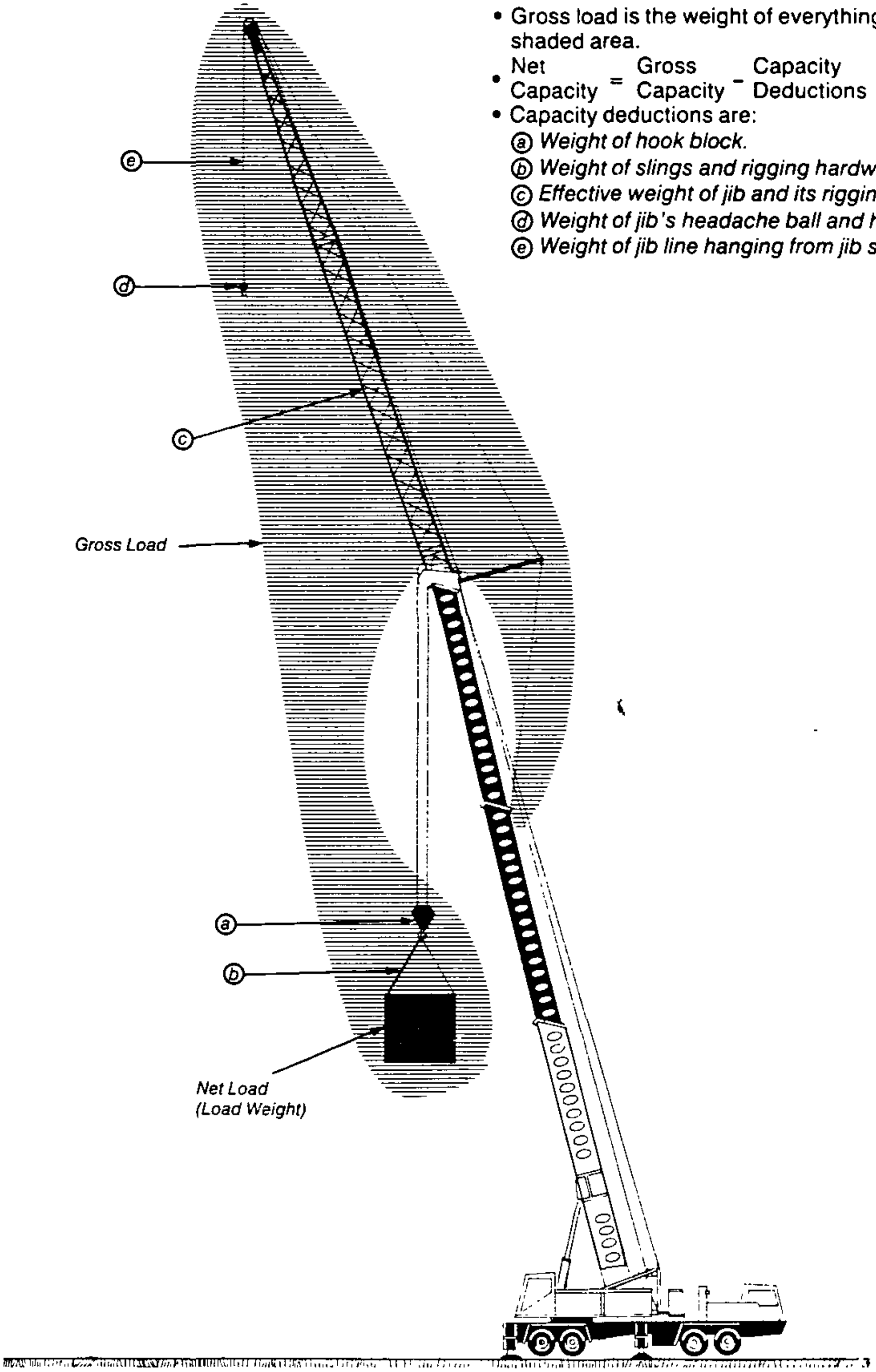
- Gross load is the weight of everything in the shaded area.
- Net Capacity = Gross Capacity − Capacity Deductions
- Capacity deductions are:
  - ⓐ *Weight of hook block.*
  - ⓑ *Weight of slings and rigging hardware.*
  - ⓒ *Effective weight of jib.*
  - ⓓ *Weight of headache ball and hook on jib.*
  - ⓔ *Weight of jib line (whip line) hanging from jib sheave.*



> *Caution:* Some manufacturers require that twice the weight of jib line and hook be deducted, e.g., American 7250.



- Gross load is the weight of everything in the shaded area.
- Net Capacity = Gross Capacity − Capacity Deductions
- Capacity deductions are:
  - ⓐ *Weight of hook block.*
  - ⓑ *Weight of slings and rigging hardware.*
  - ⓒ *Effective weight of boom extension.*
  - ⓓ *Weight of auxiliary hook and headache ball.*
  - ⓔ *Weight of auxiliary hoist line hanging below boom extension tip.*
  - ⓕ *Stowed weight of jib.*



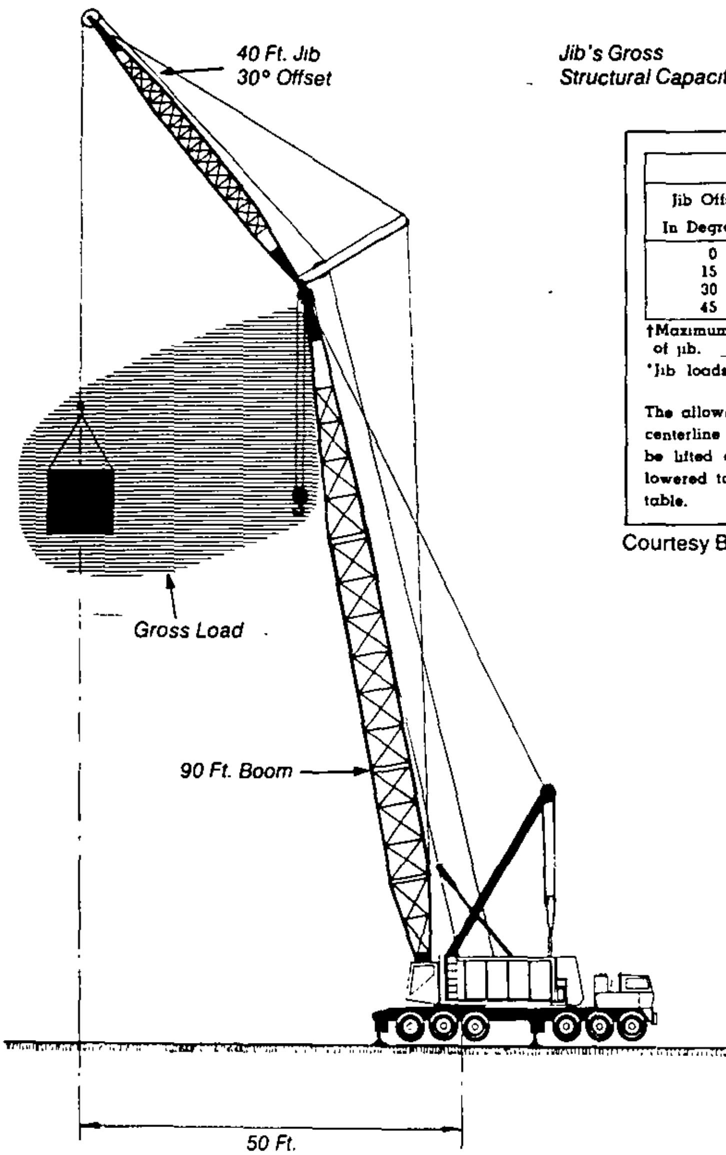
- Gross load is the weight of everything in the shaded area.
- Net Capacity = Gross Capacity − Capacity Deductions
- Capacity deductions are:
  - a *Weight of hook block.*
  - b *Weight of slings and rigging hardware.*
  - c *Effective weight of jib and its rigging.*
  - d *Weight of jib's headache ball and hook.*
  - e *Weight of jib line hanging from jib sheave.*

7.5.2 JIB CAPACITY – LATTICE BOOMS – METHOD 2

This method relies on two load charts to determine jib capacity. It uses a jib load chart based on the strength limits of the jib and the main boom chart for the crane's stability limits.

Procedure

(1) Determine the weight of the load to be lifted.
(2) Determine the weight of slings and rigging hardware.
(3) Determine the number of parts of line required to make the lift.
(4) Determine the size and weight of hook block required.
(5) Determine the load radius, main boom length, jib length, jib offset, or jib angle to horizontal.
(6) Select the jib load chart for the particular configuration of the crane and quadrant of operation (quadrants may not be a factor).
(7) For the correct jib length, jib offset and jib angle to horizontal read the *gross structural capacity* of the jib.



Jib's Gross Structural Capacity

JIB LOADS IN POUNDS				
Jib Offset In Degrees†	Jib Length			
	20 Ft.	30 Ft.	40 Ft	50 Ft
0	20,000*	17,000*	13,000	10,000
15	20,000*	17,000*	12,000	10,000
30	15,000	12,000	9,000	6,000
45	10,000	7,000	[illegible]	—

†Maximum offset (angular) from centerline of boom to centerline of jib.

*Jib loads over 16,500 pounds require 2-part jib hoist line.

The allowable load over the jib sheave, at any radius from the centerline of rotation of the machine, is the same load that may be lifted over the boom point sheave (without jib) with boom lowered to that radius, but not to exceed loads in the jib load table.

Courtesy Bucyrus-Erie Co.

7.5.2 CONTINUED

(8) Select the main boom load chart for the particular configuration of the crane and quadrant of operation.

(9) For the actual *main boom length* and *actual load radius* read the *gross capacity* listed in the chart. (See illustration and load chart below.)

(10) The *lower* of the two capacities determined in (7) and (9) becomes the *gross capacity* of the crane.

(11) Determine all capacity deductions.

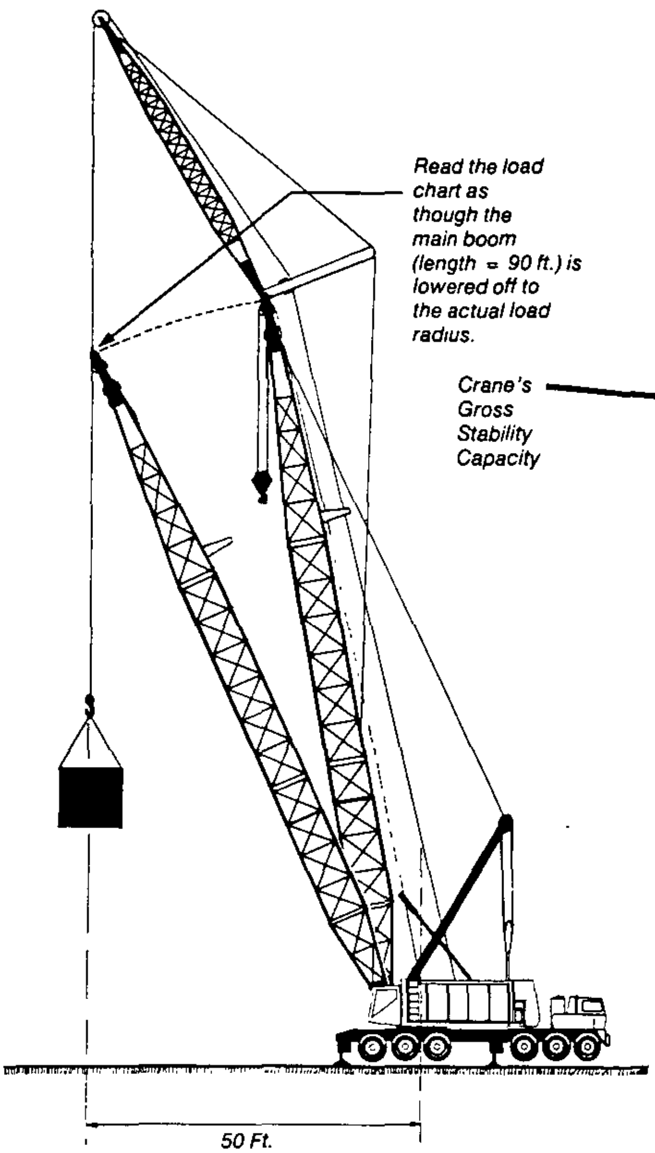
Caution: Read the fine print on the load chart because some manufacturers require that you reduce the main boom capacity determined in (9) by the effective weight of the jib. See the American 7250 exercise in the Appendix for an example of this procedure.

(12) Calculate net capacity

Net Capacity = Gross Capacity − Capacity Deductions

(13) Compare net capacity to the load weight.

For examples of this procedure see the Bucyrus-Erie 110-T and American 7250 exercises in the Appendix.



MAXIMUM ALLOWABLE LOADS IN POUNDS

BOOM LENGTH IN FEET	RADIUS IN FEET	BOOM ANGLE IN DEG.	BOOM POINT PIN HEIGHT FT. IN.	OUTRIGGERS SET † OVER SIDE OR REAR	WITHOUT OUTRIGGERS OVER SIDE	WITHOUT OUTRIGGERS OVER REAR
40	12	78	46' 6"	*220,000	100 400	139,[illegible]
	15	74	45' 9"	*185,000	71 800	[illegible] 400
	20	66	44' 0"	*149,000	48 100	54 600
	25	58	41' 3"	118,900	35,800	47,900
	30	49	37' 6"	86 700	28,300	37,800
	35	39	32' 6"	67,900	23,200	31 100
60	15	79	66' 3"	*180,000	71 000	97,300
	20	74	65' 0"	*147,000	47,300	63 800
	25	69	63' 6"	118,500	35 000	47 200
	30	64	61' 3"	86,200	27,500	37,100
	35	59	58' 6"	67 400	22,400	30,300
	40	53	55' 3"	54 700	18,800	25,500
	50	40	45' 9"	39 400	13,900	19 100
80	20	78	85' 9"	*145 000	46,700	63,300
	25	75	84' 6"	110,200	34,400	46 600
	30	71	83' 0"	85,900	26 900	36 500
	35	67	81' 0"	67,000	21 800	29 [illegible]
	40	63	78' 9"	54,300	18 200	24,900
	50	55	72' 9"	38 900	13,300	18,500
	60	45	64' 3"	29 900	10 100	14,300
	70	34	52' 3"	24 000	8 000	11,500
90	20	80	95' 9"	*143 000	46,400	63,100
	25	76	94' 9"	*117 200	34,100	46 300
	30	73	93' 6"	85,[illegible]	26,600	[illegible] 200
	40	[illegible]	[illegible]	54,100	17,900	24 600
	50	59	8[illegible]	38,600	13,000	18 200
	60	51	77' 9"	[illegible]	9,400	14,[illegible]
	70	43	68' 3"	23,800	7 700	11,200
	80	32	55' 3"	19 600	6,100	9,150
100	25	78	105' 0"	*116,000	33 900	46 100
	30	75	103' 9"	85,500	26,400	36 200
	40	69	100' 6"	53,900	17 600	24,400
	50	63	96' 3"	38 400	12,700	17,900
	60	56	90' 0"	29 400	9 600	13 930
	70	49	82' 3"	23 500	7 450	11,000
	80	41	72' 3"	19 400	5,900	8 900
	90	31	58' 3"	16,300	4,600	7 330
110	25	80	115' 3"	*115,000	33 500	45,700
	30	76	114' 3"	85 300	26,000	35 600
	40	71	111' 3"	53,600	17 300	24 000
	50	65	107' 3"	38 200	12,400	17 620
	60	59	102' 0"	29,100	9 250	13,500
	70	53	95' 3"	23 200	7 100	10 600
	80	46	86' 9"	19 100	5,450	8 550
	90	39	75' 9"	16 000	4,250	6 950
	100	29	60' 9"	13 600	3,250	[illegible] 650
120	25	80	125' 6"	*114 000	33,300	45,500
	30	77	124' 6"	85,100	25 800	35,400
	40	73	121' 9"	53 400	17 000	23,800
	50	67	118' 9"	38 000	12 100	17 330
	60	62	113' 3"	28 900	9 000	13 230
	70	57	107' 6"	23,000	6 900	10,300
	80	51	100' 0"	18,800	5,200	8 300
	90	44	90' 9"	15,700	4,000	6 950
	100	37	79' 3"	13,300	3 000	5 400
	110	28	63' 3"	11 400	2 200	4,400
*130	30	78	134' 6"	84,900	25,500	35 100
	40	74	132' 3"	53,200	16,700	23 500
	50	69	128' 9"	37 700	11,800	17 300
	60	64	124' 6"	28,700	8 700	12 900
	70	59	119' 3"	22,800	6 500	10 300
	80	54	112' 6"	18,600	4 900	8,000
	90	49	104' 6"	15,500	3 700	6,400
	100	42	94' 9"	13,000	2 700	5,100
	110	35	82' 6"	11 100	1 900	4,100
	120	27	65' 9"	9,600	1 250	3,250
140	30	79	144' 9"	84 700	25 200	34 [illegible]
	40	75	142' 6"	53 000	16,500	[illegible]
	50	71	139' 6"	37 500	11 600	16 800
	60	66	135' 6"	28,500	8 450	[illegible]
	70	62	130' 9"	22,500	6 250	9,600
	80	57	124' 9"	18 300	4 650	[illegible] 100
	90	52	117' 9"	15 200	3 400	6 100
	100	47	109' 0"	12,800	2,450	4 950
	110	41	98' 9"	10 900	1 650	3,850
	120	34	85' 6"	9 350	1,000	3,000
	130	26	68' 3"	8,050	—	2 250

Courtesy Bucyrus-Erie Co.

7.6.1 BOOM EXTENSION CAPACITIES – FULL POWER BOOMS

Procedure – Full Boom Extension

(1) Determine weight of load to be lifted.
(2) Determine weight of slings and rigging hardware.
(3) Determine number of parts of line required to make lift.
(4) Determine size and weight of hook block required.
(5) Determine load radius, main boom angle and boom length (including length of extension).
(6) Select the correct load chart based on the crane's actual configuration, base configuration and quadrant of operation.
(7) For the *actual full boom length* (including the length of the boom extension), *load radius* and *main boom angle* read the *gross capacity* from the load chart.
(8) Determine capacity deductions.
(9) Calculate net capacity

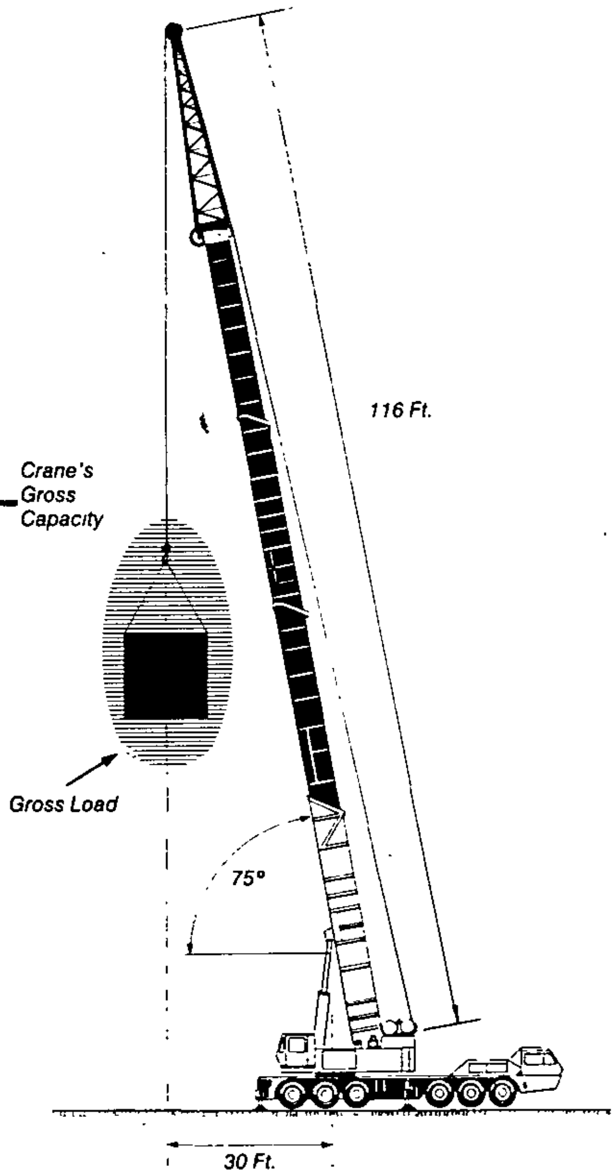
$$\text{Net Capacity} = \text{Gross Capacity} - \text{Capacity Deductions}$$

(10) Compare net capacity to the load weight.

ON OUTRIGGERS FULLY EXTENDED - OVER REAR

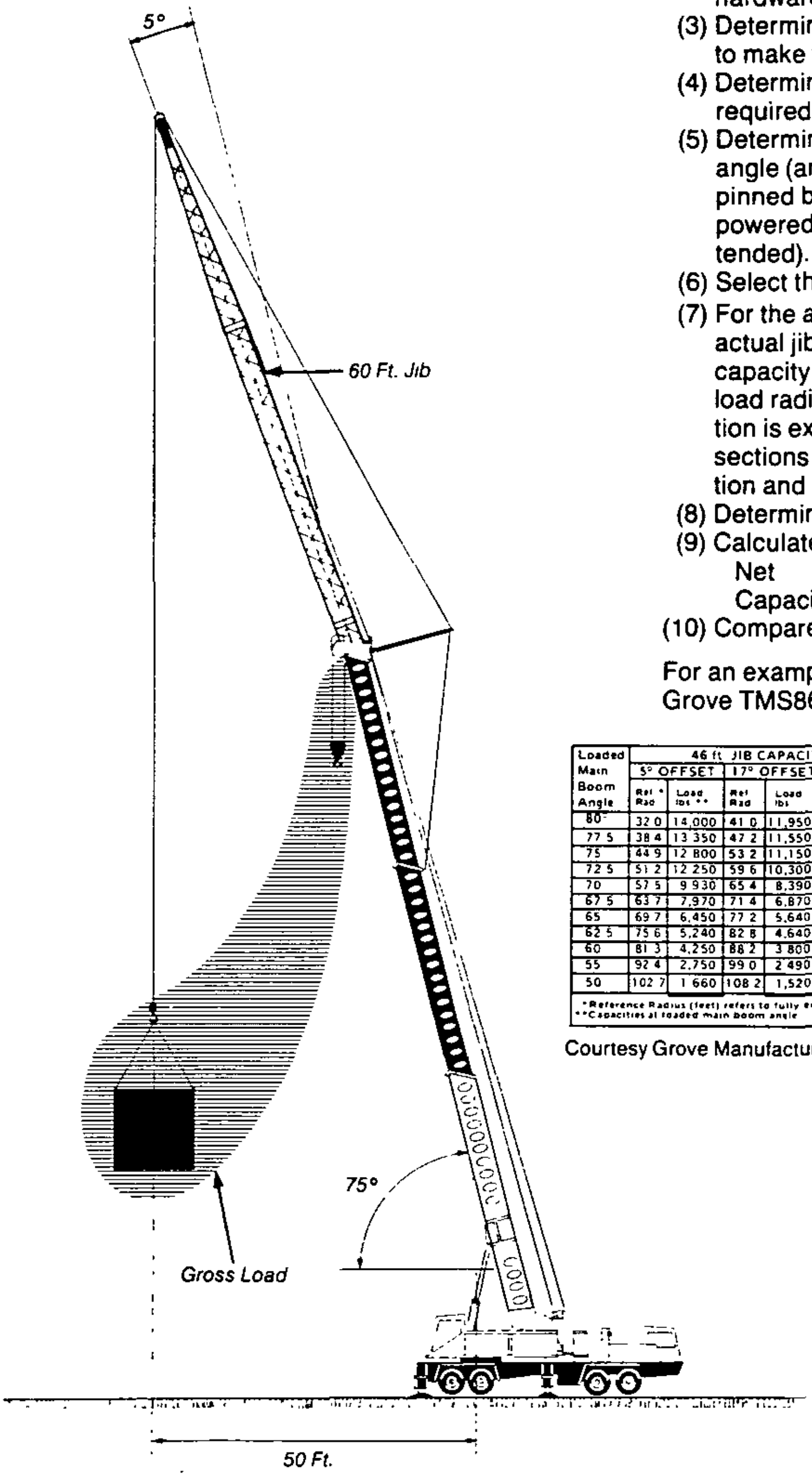
Radius in Feet	Boom Length in Feet							84 ft + 32 ft Ext
	*34	40	44	54	64	74	84	**116
10	100 000 (70)	74 000 (73)	72,000 (76)					
12	90 000 (66 5)	70,000 (70)	67 500 (73 5)	64 000 (76 5)				
15	72,000 (61)	63 700 (65 5)	61 000 (69)	55 000 (73)	44,700 (76)			
20	53 000 (50 5)	52 200 (57 5)	49 800 (62)	44,000 (67 5)	37 900 (71)	35 000 (74)	31 000 (76 5)	
25	41 000 (38 5)	41 000 (48)	41,000 (54)	36,300 (61 5)	31 900 (66)	29,200 (70)	27,500 (73 5)	17 500 (75 5)
30	29,690 (21 5)	29 690 (37 5)	29,690 (45)	29,690 (55 5)	27,000 (60 5)	25,000 (65 5)	23,900 (69 5)	16 600 (75)
35		22,650 (23)	22 650 (34 5)	22 650 (48 5)	22 650 (55)	21 800 (61)	20,500 (66)	14 500 (72 5)
40			18 090 (19)	18,090 (41)	18,090 (49)	18,090 (56 5)	17,900 (62)	12,800 (70)
45				14 840 (31 5)	14 840 (42)	14 840 (51 5)	14 840 (58)	11,400 (67)
50				12,330 (17 5)	12,330 (35)	12 330 (46)	12,330 (53 5)	10,200 (64 5)
55					10 440 (26)	10,440 (40 5)	10,440 (49)	9,190 (61 5)
60					9,100 (12 5)	9 100 (34)	9,100 (44)	8,440 (59)
65						7 990 (25 5)	7,990 (38 5)	7,760 (56)
70						6,880 (14)	6,880 (32 5)	7,100 (53)
75							5,770 (25)	6 630 (49 5)
80							4,660 (13 5)	6,130 (46)
85								5,360 (42 5)
90								4,630 (39)
95								3,980 (34 5)
100								3 420 (29 5)
105								2,940 (24)
110								2,560 (16)
Note Boom angles are in degrees						A6 629 002740 & 002760		

Courtesy Grove Manufacturing Co.



7.7.2 JIB CAPACITIES – USING ONE LOAD CHART – PINNED BOOMS

Increasingly, the most common method of determining jib capacity uses only one load chart containing both the jib's structural strength limits and the crane's stability limits (an integrated chart).



Procedure – Pinned Boom Section Extended or Retracted – All Boom Lengths

(1) Determine weight of load to be lifted.
(2) Determine weight of slings and rigging hardware.
(3) Determine number of parts of line required to make the lift.
(4) Determine size and weight of hook block required.
(5) Determine jib length, jib offset, main boom angle (and load radius but only if the pinned boom section is extended and the powered boom sections are fully extended).
(6) Select the correct load chart.
(7) For the *actual main boom angle* and the actual jib length and offset read the gross capacity from the jib load chart. Do *not* use load radius unless the pinned boom section is extended *and* the powered boom sections are fully extended. (See illustration and load chart.)
(8) Determine capacity deductions.
(9) Calculate net capacity

$$\text{Net Capacity} = \text{Gross Capacity} - \text{Capacity Deductions}$$

(10) Compare net capacity to load weight.

For an example of this procedure see the Grove TMS865 exercise in the Appendix.

Loaded Main Boom Angle	46 ft JIB CAPACITIES						60 ft JIB CAPACITIES					
	5° OFFSET		17° OFFSET		30° OFFSET		5° OFFSET		17° OFFSET		30° OFFSET	
	Ref.* Rad	Load lbs **	Ref Rad	Load lbs	Ref Rad	Load lbs	Ref. Rad	Load lbs	Ref Rad	Load lbs	Ref Rad	Load lbs
80°	32 0	14,000	41 0	11,950	49 0	8,480	36 3	10,600	48 2	8,160	58 0	5 680
77 5	38 4	13 350	47 2	11,550	55 2	8 080	43.3	9,970	54 5	7 790	64 7	5 320
75	44 9	12 800	53 2	11,150	61 2	7 690	50 3	9,490	60 9	7 450	71 3	5,020
72 5	51 2	12 250	59 6	10,300	67 2	7 350	57 2	8,[illegible]30	67 5	7,130	77 8	4,760
70	57 5	9 930	65 4	8,390	72 9	7,020	63 9	[illegible]380	74 1	6,830	84 2	4,540
67 5	63 7	7,970	71 4	6,870	78 6	6 100	70 6	[illegible]710	80 6	5 610	90 3	4 340
65	69 7	6,450	77 2	5,640	84 1	5,080	77 1	5,380	86 8	4,580	96 3	4 030
62 5	75 6	5,240	82 8	4,640	89 5	4,220	83 4	4,320	92 7	3 730	102 1	3 310
60	81 3	4,250	88 2	3 800	94 7	3,490	89 [illegible]	3 460	98 6	3,010	107 8	2,700
55	92 4	2,750	99 0	2 490	104 4	2,320	10[illegible] 4	2 130	110 0	1 870	118 3	1,700
50	102 7	1 660	108 2	1,520	113 4	1,420	1[illegible]2 5	1 160	121 9	1 010	128 1	920

*Reference Radius (feet) refers to fully extended boom and appropriate jib length
**Capacities at loaded main boom angle

Courtesy Grove Manufacturing Co

Crane's Gross Capacity

7.7.4 BOOM EXTENSION AND JIB COMBINATION CAPACITIES – USING ONE LOAD CHART – PINNED BOOMS

This method uses a single load chart containing both strength and stability limits.

PINNED BOOM SECTION RETRACTED OR EXTENDED

Procedure – All Boom Lengths

(1) Determine weight of load to be lifted.
(2) Determine weight of slings and rigging hardware.
(3) Determine number of parts of line required to make the lift. (Multiple-part reeving might not be permitted.)
(4) Determine size and weight of hook block required.
(5) Determine boom extension and jib type and length, jib offset, main boom angle (and load radius, but only if the pinned boom section is extended and powered boom sections are fully extended).
(6) Select the correct load chart.
(7) For the *actual main boom angle* and *jib offset* read the gross capacity from the combination boom extension and jib load chart. Do not use load radius unless the boom is fully extended with the pinned boom section extended.
(8) Determine capacity deductions.
(9) Calculate net capacity

$$\text{Net Capacity} = \text{Gross Capacity} - \text{Capacity Deductions}$$

(10) Compare net capacity to load weight.

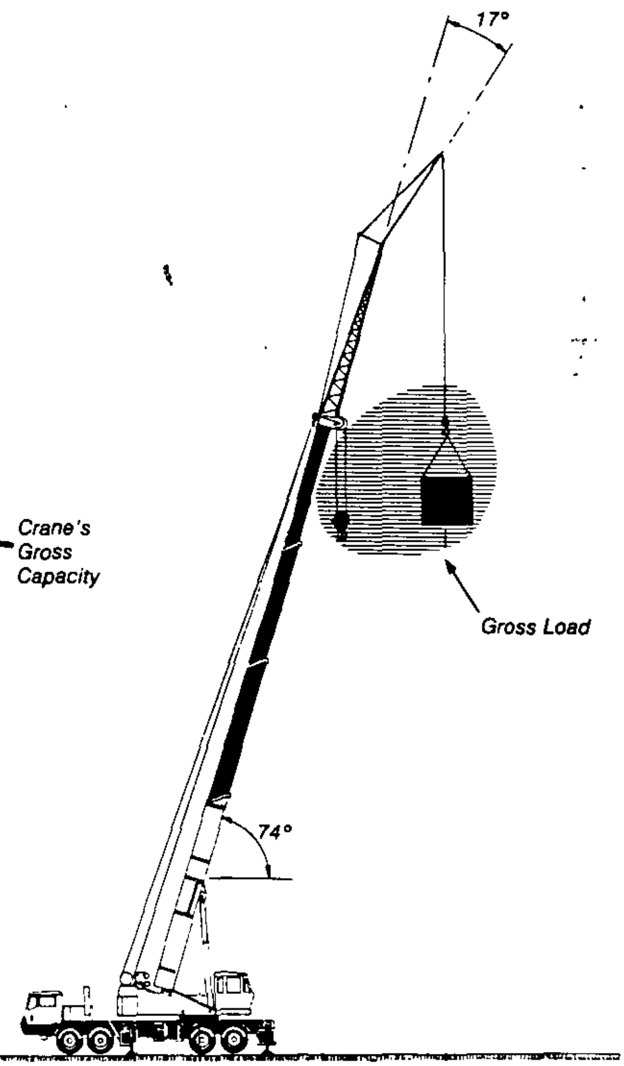
For examples of this procedure see the Grove RT865 and P & H Omega 40 exercises in the Appendix.

MAXIMUM LOAD RATINGS IN POUNDS			
Minimum Boom Angle	JIB ANGLE OFFSET 5°	17°	30°
78	5000	4200	3700
75	4600	4000	3500
70	4100	3700	3300
65	3500	3200	2900
60	3100	2900	2700
55	2600	2500	2400
50	2200	2100	2000

JIB CAPACITY NOTES

1 MAXIMUM JIB LOAD RATINGS ARE BASED ON STRUCTURAL COMPETENCE AND DO NOT EXCEED 85% OF TIPPING LOAD WITH FULLY EXTENDED OUTRIGGERS USE OF OUTRIGGERS IS REQUIRED WHEN BOOM IS EQUIPPED WITH JIB
2 FOR BUCKET RATINGS ON JIB DEDUCT 20% FROM MAXIMUM JIB LOAD RATINGS
3 **WARNING.** DO NOT LIFT WITH JIB AT BOOM ANGLES BELOW 50° LOSS OF STABILITY OCCURS RAPIDLY
4 **WARNING** DO NOT EXCEED 130 FOOT OPERATING RADIUS WITH ERECTED JIB OR A TIPPING CONDITION WILL OCCUR

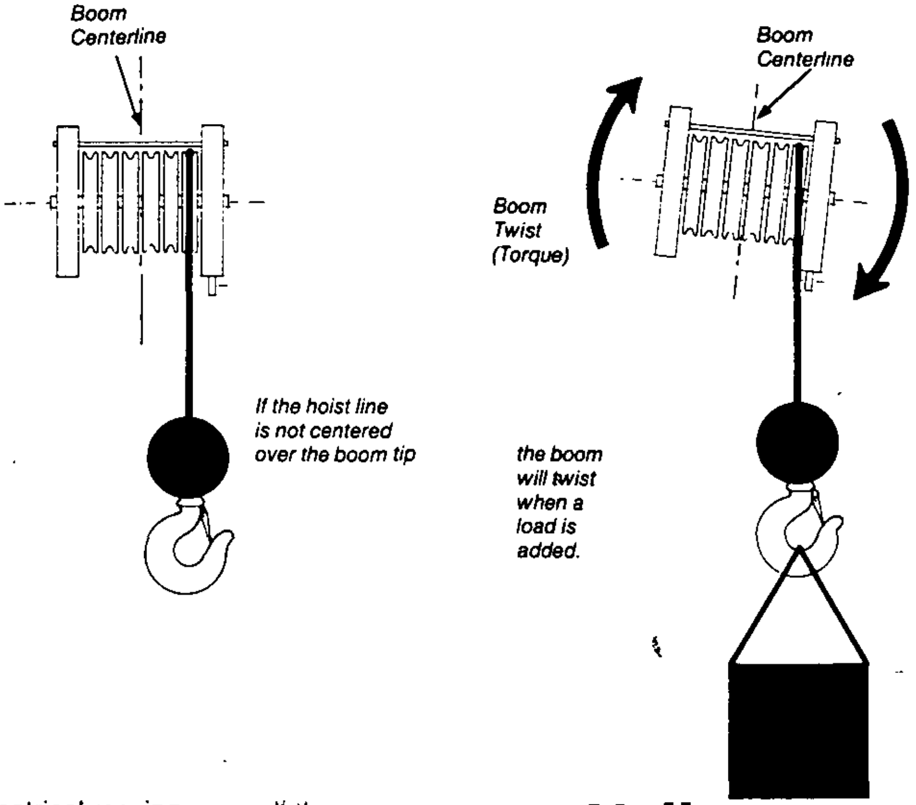
Courtesy Harnischfeger Corp.



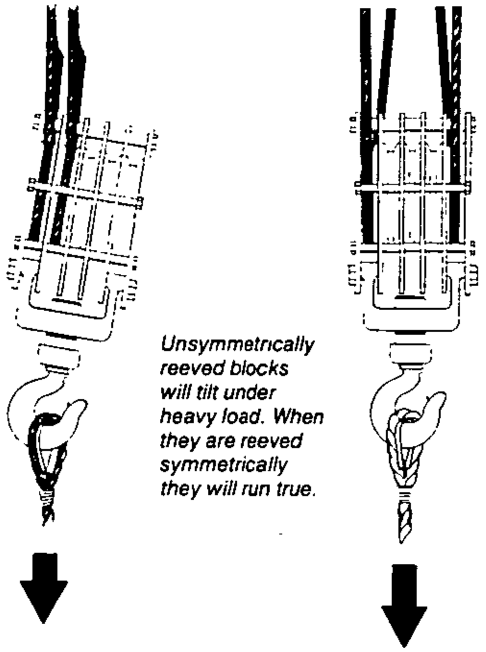
8.3 ECCENTRIC REEVING

Eccentric (Unbalanced) Reeving

Eccentric or unbalanced reeving of the boom tip will cause torsion (twisting) in the boom for which there is no allowance in the load chart. Full chart ratings apply only when the tip is symmetrically rigged.

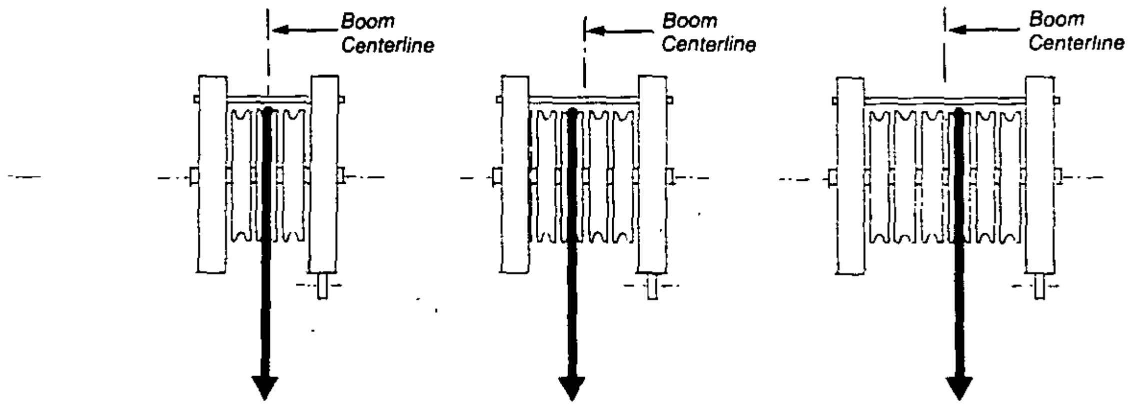


Unsymmetrical reeving of the load block will cause it to tilt and creates rapid sheave wear.

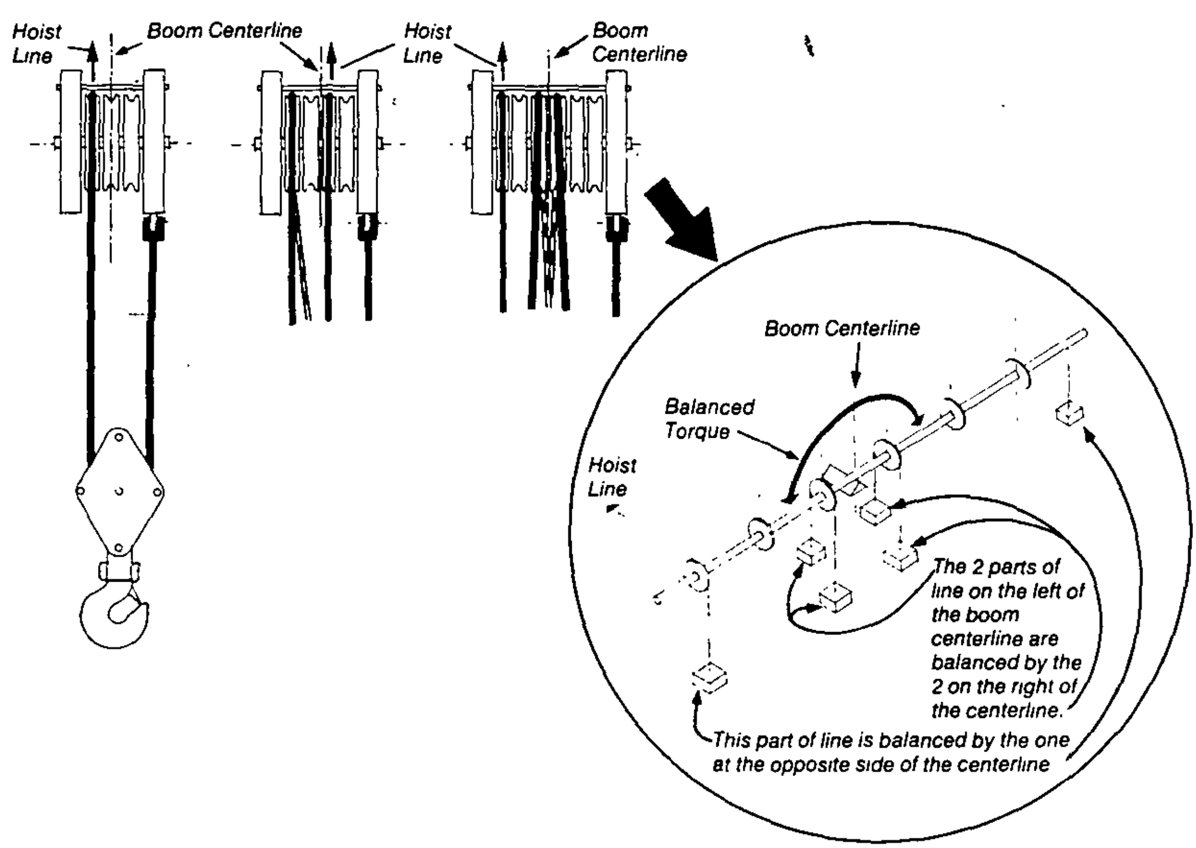


Symmetrical (Balanced) Reeving

On single line reeving, when the hoist line runs on the center sheave or on the sheave beside the centerline of the boom, boom torque is eliminated, or minimized.



On multi-part reeved systems, if the parts of line are evenly distributed on either side of the boom centerline, boom torque will be eliminated or minimized as much as possible.



9.3 WIRE ROPE INSPECTION

One of the most important pre-operational checks to be made on the crane is the rope and rigging inspection. Assurance of safety and economy of the equipment requires a program of periodic inspections of all wire rope and fittings. Factors such as abrasion, wear, fatigue, corrosion and kinking are often of greater significance in determining the usable life of wire rope than are strength factors based on new rope conditions.

All wire rope should be observed during normal operation and visually inspected on a weekly basis. A complete and thorough inspection of all ropes in use must be made at least once a month.

Where the rope is in constant use, a thorough inspection should be made regularly once a week or more often if required.

It is also good practice, when the equipment is in constant use, to give the rope a certain length of service, several hundred hours, several weeks or months and then replace the rope regardless of its condition. This method eliminates the risk of fatigue causing rope failure.

All rope which has been idle for a period of a month or more should be given a thorough inspection before it is put back into service.

All inspections are the responsibility of the operator.

A record of each rope should be kept (include date of fitting, size, construction, length, defects found during inspections and length of service).

Any deterioration, resulting in a suspected loss of original rope strength, should be carefully examined and a determination made as to whether further use of the rope would constitute a safety hazard.

The time to remove a rope from service is related to the conditions of the particular installation. These conditions include the size, nature and frequency of the lifts, when the next inspection will be and what the operating and maintenance practices are.

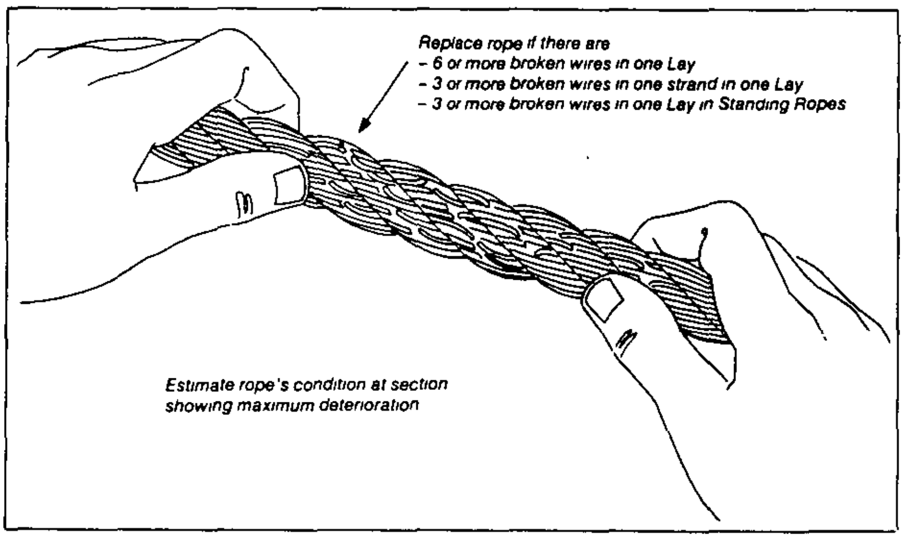
Only by inspection can it be determined whether or not the rope should be replaced. The operator must decide:

- If the rope's condition presents any possibility of failure; and
- If the rate of deterioration of the rope is such that it will remain in safe condition until the next scheduled inspection.

When inspecting the rope give every inch of its length equal care as serious deterioration frequently occurs in localized positions. The estimate of the rope's condition must be made at the section showing the maximum deterioration.

Conditions such as the following are sufficient to either seriously *question the rope safety or remove the rope from service.*

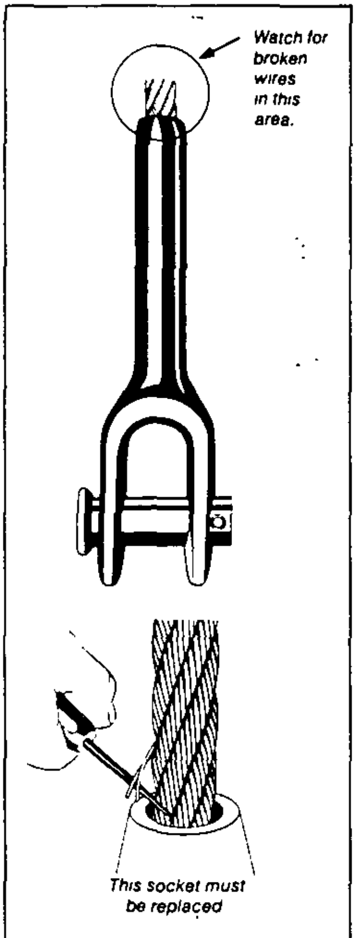
(1) Broken Wires: Occasional premature wire failures may be found early in the life of almost any rope and in most cases they should not constitute a basis for rope removal provided they are at well spaced

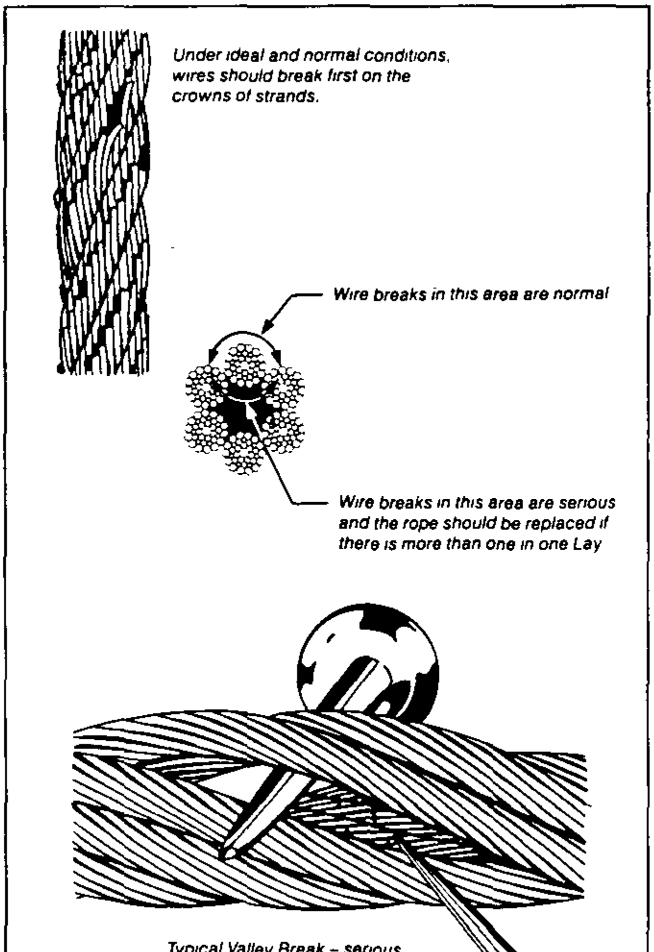


intervals. Note the area and watch carefully for any further wire breaks.

The rope must be replaced if:

- In *running ropes,* there are *six* or more randomly distributed broken wires *in one rope lay, or three* or more broken wires *in one strand* in one rope lay. (A rope lay is the length along the rope in which one strand makes a complete revolution around the rope.)
- In *pendants* or standing ropes, there are *three* or more broken wires *in one rope lay.*
- In any rope there is more than one broken wire near an attached fitting. Breaks that occur near attached fittings, such as sockets, are usually the result of fatigue stresses concentrated in these localized sections. Wire breaks of this type should be cause for replacement of the rope or renewal of the attachment to eliminate the locally fatigued area. Six to eight feet should be cropped off the rope below the socket.
- In running ropes there is *any evidence* of wire breaks in the valleys between strands.





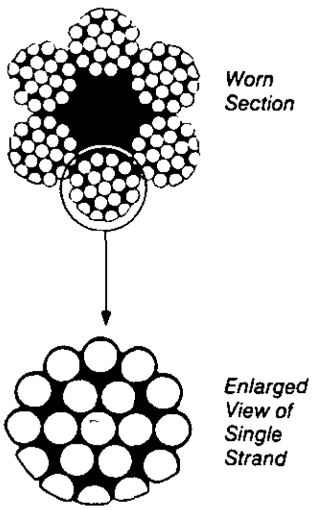
Typical Valley Break – serious

IMPORTANT. The most poorly maintained and least inspected ropes on most cranes are the pendants and the boom hoist ropes. They *must* be given frequent detailed inspections because they are the most important ropes on the machine.

Breaks occurring on crowns of outside wires indicate normal deterioration. Breaks in valleys between strands indicate an abnormal condition, possibly fatigue or breakage of other wires not readily visible. More than one of these valley breaks in one rope lay should be cause for replacement.

(2) Worn and Abraded Wires: Each individual wire in a rope, when new, is a complete circle in cross section. Wear eventually causes the outer wires to become flat on the outside.

This is normal service deterioration and in most installations where operating conditions are not particulary severe, relatively even abrasion will occur on the outer wires. The rope must be replaced, however, if this wear exceeds 1/3 of the diameter of the individual wires.



When the surface wires are worn by 1/3 or more of their diameter the rope must be replaced.

(3) Reduction in Rope Diameter: Any marked reduction in rope diameter is critical. It is often due to excessive abrasion of the outside wires, loss of core support, internal or external corrosion, inner wire failures or a loosening of the rope lay. All new ropes stretch slightly and decrease in diameter after being used. This is normal, but the rope must be replaced if the rope diameter is reduced by more than:

- 3/64 in. for rope diameters up to and including 3/4 in.
- 1/16 in. for rope diameters of 7/8 to 1 1/8 in.
- 3/32 in. for rope diameters of 1 1/4 to 1 1/2 in.

(4) Rope Stretch: Severe stretch or elongation of rope is also a deterioration factor. All wire ropes will stretch during initial use. This is known as "Constructional Stretch" and is caused by the tightening of the wires and strands into their respective cores. An approximate elongation of 6 inches per 100 feet of rope can be expected in a 6 strand rope and approximately 9 to 10 inches in an 8 strand rope. Excessive stretch beyond this should be cause for replacement. Watch for a lengthening of rope lay or a reduction in rope diameter. These are the signs of severe stretch generally caused by overloading.

(5) Corrosion: This can be infinitely more dangerous than wear because more wires will be affected.

Corrosion frequently develops inside the rope before any surface evidence is visible. Therefore, if surface corrosion is detected by the characteristic discoloration of the wires or, in particular, if pitting is observed then consideration must be given to replacing the rope. Noticeable rusting in the vicinity of fittings is also cause for replacement.

(6) Insufficient Lubrication: Examine the grooves between the strands. Where these are filled with hard packed grease or dirt the rope needs to be relubricated.

(7) Crushed, Flattened or Jammed Strands: Replace the rope. These are dangerous conditions because of severe wire deformation. They often occur when there are multiple layers of rope wound on drums. They can also occur if the hoist rope becomes slack and cross-coiled on the drum or trapped in the machinery.

(8) High Stranding and Unlaying: Replace the rope. In cases such as this, excessive wear and crushing take place and the other strands become overloaded.

(9) Continued next page.

(9) Bird Caging: Replace the rope.

Multi-strand rope "bird cages" due to torsional unbalance. Typical of build-up seen at anchorage end of multi-fall crane application.

A "bird cage" caused by sudden release of tension and resultant rebound of rope from overloaded condition. These strands and wires will not return to their original positions.

(10) Kinks: Replace the rope. They are usually caused by faulty handling or reeving. The strands become dog-legged and where running on sheaves are subject to excessive wear at the kink.

(11) Bulges in Rope: Replace the rope, particularly if it is of a non-rotating construction. This is indicative of core slippage or "turns" being put into or taken out of the rope.

(12) Gaps or Excessive Clearance Between Strands: Replace the rope.

(13) Core Protrusion: Replace the rope.

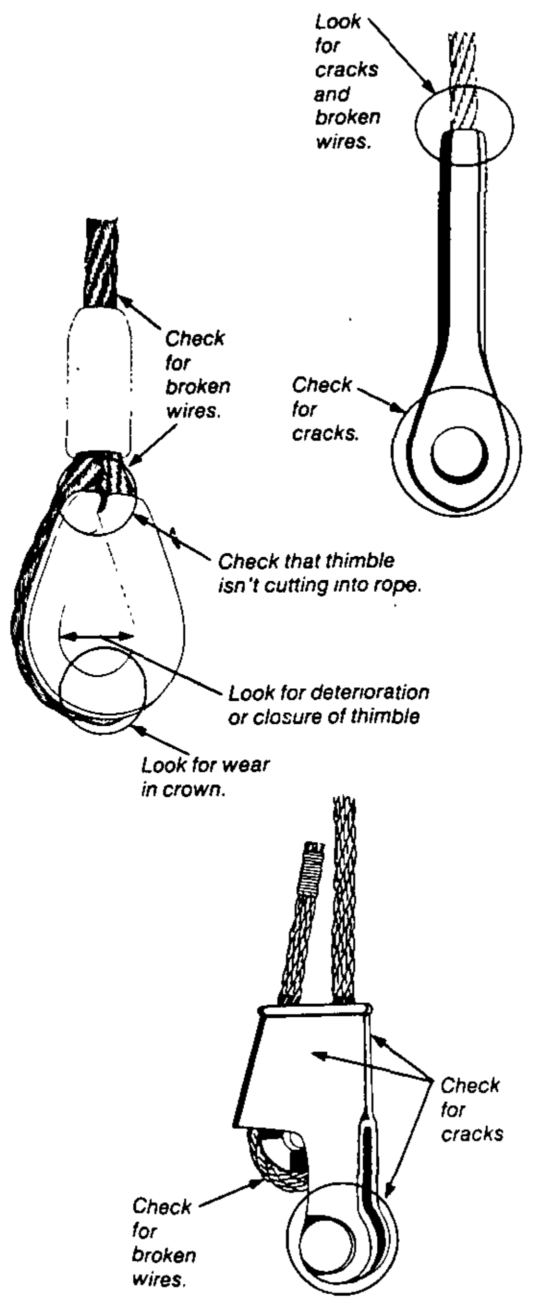
Core protrusion as a result of torsional unbalance created by shock loading.

Protrusion of IWRC resulting from shock loading.

(14) Unbalanced Severely Worn Areas: Replace the rope.

(15) Heat Damage, Torch Burns, Electric Arc Strikes: Replace the rope.

(16) Corroded, Cracked, Bent, Worn and Improperly Applied End Connections: If any of these conditions exist, replace the fitting.



When inspecting a rope remember that its operating speed has a bearing on its life. The life expectancy of a high speed rope (due to increased impact at sheaves and drums, friction and abrasion) is less than a slow speed rope. Due consideration must be given to this aspect and inspections made accordingly.

Where multi-layer drums are used, examine not only that part of the rope which is in constant use, but also the rope which may remain spooled and inoperative on the drum.

When replacing a rope, make certain that the replacement rope is of the correct size, grade and construction.

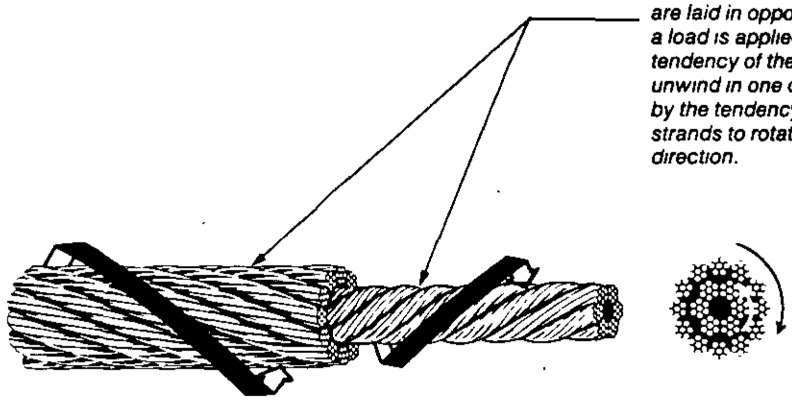
If the rope does not achieve its normal life expectancy, examine it closely and try to pinpoint the cause of the damage or rapid deterioration. The following table might be helpful.

FAULT	POSSIBLE CAUSE
Accelerated Wear	Severe abrasion from being dragged over the ground or obstructions. Rope wires too small for application or wrong construction or grade Poorly aligned sheaves. Large fleet angle Worn sheaves with improper groove size or shape. Sheaves, rollers and fairleads having rough wear surfaces. Stiff or seized sheave bearings High bearing and contact pressures
Rapid Appearance of Broken Wires	Rope is not flexible enough. Sheaves, rollers, drums too small in diameter. Overload and shock load Excessive rope vibration Rope speed too high Kinks that have formed and been straightened out. Crushing and flattening of the rope Reverse bends Sheave wobble
Rope Broken Off Square	Overload, shock load Kink. Broken or cracked sheave flange
Strand Break	Overload, shock load. Local wear Slack in 1 or more strands.
Corrosion	Inadequate lubricant. Improper type of lubricant. Improper storage Exposure to acids or alkalis
Kinks, Dog Legs, Distortions	Improper installation Improper handling
Excessive Wear in Spots	Kinks or bends in rope due to improper handling in service or during installation Vibration of rope on drums or sheaves
Crushing and Flattening	Overload, shock load. Uneven spooling Cross winding. Too much rope on drum Loose bearing on drum. Faulty clutches. Rope dragged over obstacle.
Stretch	Overload Untwist of Lang Lay ropes
Broken Wires or Undue Wear on One Side of Rope	Improper alignment. Damaged sheaves and drums
Broken Wires Near Fittings	Rope vibration
Burns	Sheave groove too small Sheaves too heavy Sheave bearings seized Rope dragged over obstacle
Rope Core Charred	Excessive heat
Corrugation and Excessive Wear	Rollers too soft. Sheave and drum material too soft.
Distortion of Lay	Rope improperly cut. Core failure. Sheave grooves too big.
Pinching and Crushing	Sheave grooves too small
Rope Chatters	Rollers too small
Rope Unlays	Swivel fittings on Lang Lay ropes Rope dragging against stationary object.
Crushing and Nicking	Rope struck or hit during handling.
High Stranding	Fittings improperly attached Broken strand Kinks, dog legs Improper seizing.
Reduction in Diameter	Broken core. Overload. Corrosion Severe wear.
Bird Cage	Sudden release of load.
Strand Nicking	Core failure due to continued operation under high load
Core Protrusion	Shock loading. Disturbed rope lay. Rope unlays Load spins

9.4 INSPECTION OF ROTATION-RESISTANT WIRE ROPE

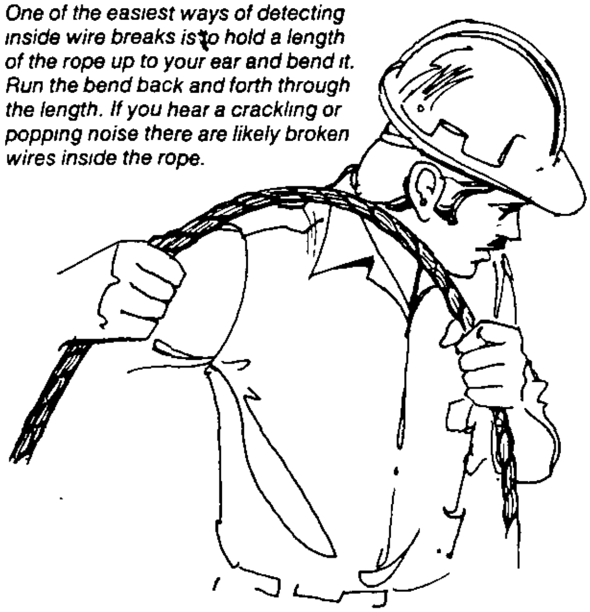
Inspect non-rotating (rotation-resistant, non-spin, spin-resistant) wire ropes very carefully because they tend to wear from the inside out. Before any damage or wear is evident on the outside of the rope they can be severely damaged inside. This is due to the construction of the rope.

The outer and inner layers of strands are laid in opposite directions. When a load is applied to the rope the tendency of the outer strands to unwind in one direction is countered by the tendency of the inner layer of strands to rotate in the opposite direction.



The ropes are not perfectly non-rotating – when a load is applied they will rotate to a degree (when lifted on a single part line). This rotation is due to the fact that there is more "torque" in the outer layer of strands than there is in the inner layer. As the outer strands unwind (and become longer), the inner strands wind up (and become shorter). The shorter of the two layers (the inner) will carry more load. This extra loading plus the very high stresses created at the points where the two layers of strands are in contact causes wire fatigue which in time results in broken wires in the inner layer of strands.

One of the easiest ways of detecting inside wire breaks is to hold a length of the rope up to your ear and bend it. Run the bend back and forth through the length. If you hear a crackling or popping noise there are likely broken wires inside the rope.



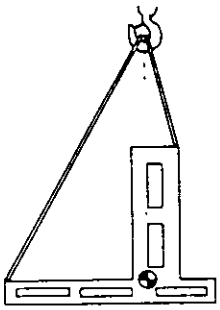
Note: In most manuals and safety regulations the guidelines for replacing wire rope are not applicable to non-rotating rope because they ignore the "inside wire break" problem most often found in the ropes.

10.6 CENTER OF GRAVITY OF THE LOAD

Two conditions are necessary for load stability:

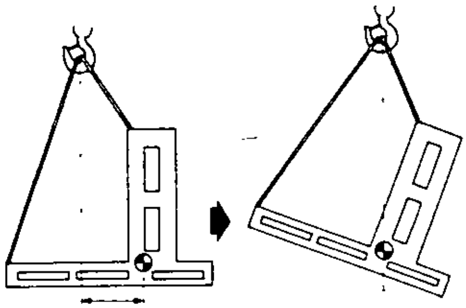
(1) The crane hook must be directly above the load's center of gravity. If not, the load will shift when airborne until the center of gravity lines up with the hook.

(2) The sling attachment points on the load must be above the load's center of gravity to prevent toppling.



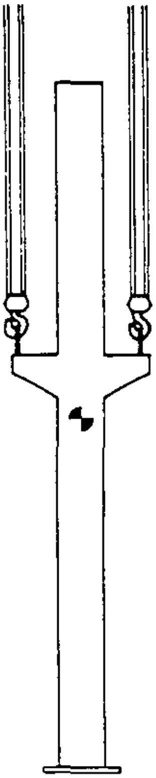
Stable

Load's C of G is under crane hook and lower than sling attachment points.



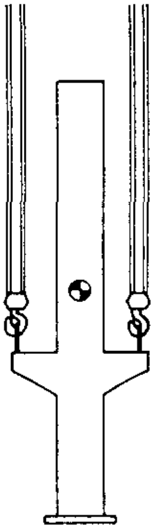
Unstable

Load's C of G is not under the crane hook. The load will shift when airborne.



Stable

Load's C of G is below sling attachment points.



Unstable

Load's C of G is above sling attachment points.

10.7 CRITICAL LIFTS

Critical lifts are those where the load weight is close to the rated capacity of the crane. When lifting load weights heavier than 75% of the rated capacity it is recommended that the following precautions be taken.

NOTE: Even though the actual load weight may be small compared to the base rating of the crane it can still be a critical lift. For example, a 1 ton load on a 50 ton capacity crane may seem insignificant but if that crane's rated capacity at the actual load radius is only 2,400 lbs. the lift becomes critical.

SUPPORTING SURFACE

The ground must be compact and stable.

BLOCKING

Unless crane sits on a concrete pad, outrigger blocking must be used and crawlers should be on pads or cribbing.

LEVEL

The machinery deck or boom foot pins must be absolutely level.

LOAD

The load weight must be determined exactly.

CENTER OF GRAVITY

The location of the load's C of G must be determined and the crane hook positioned above it.

LOAD RADIUS

The radius must be measured exactly.

BOOM LENGTH

The boom length must be determined exactly.

BOOM ANGLE

The boom angle, if necessary for determining the crane's capacity, must be determined exactly. Do not rely on the crane's boom angle indicator.

WIND

Wind effects must be considered and the lift delayed if the wind loads are significant. If the wind speeds are in excess of 30 mph do not make the lift. If the speeds are more than 20 mph consider postponing it. (See Sect. 8.12.)

REEVING

The reeving must be balanced.

LOAD RIGGING

Check for adequacy and security. The weight of rigging must be known exactly.

OPERATION

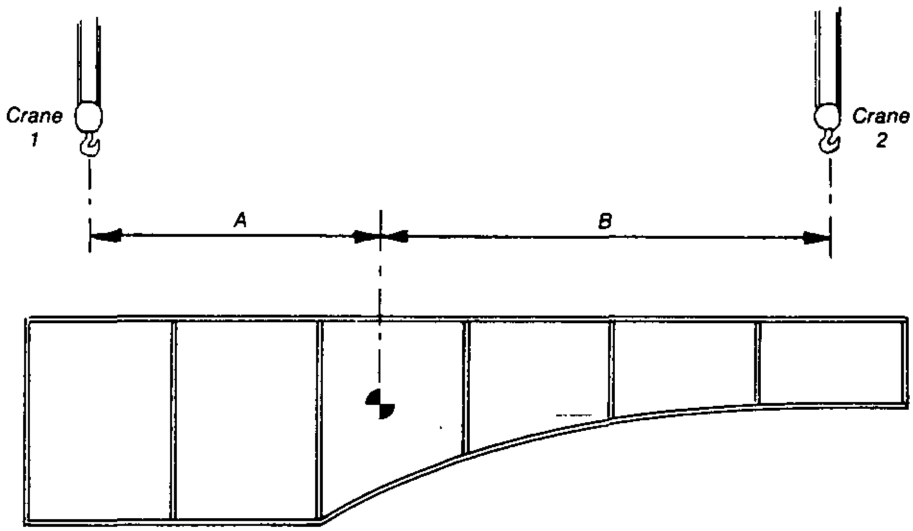
All control, machine and load movements must be made as slowly and smoothly as possible.

Caution: The determination of these parameters by untrained personnel or by using uncalibrated instruments is extremely hazardous.

10.21 SHARE OF LOAD ON DUAL LIFTS

Whenever two cranes are used to lift a load, the location of each crane's hook block above the load will have to be chosen carefully so as to properly share the load between the cranes.

The method for properly locating each crane's hook block above the load is as follows:



(1) Determine the *lowest* value of rated *net capacity* that crane 1 will have during the *whole operation* considering all factors such as configuration, quadrant of operation, boom length, boom angle, load radius, weight of rigging and all capacity deductions.

(2) Repeat step (1) for crane 2.

(3) Check to ensure that the sum of the lowest rated net capacities of the cranes exceeds the load weight.

(4) Locate the position of the center of gravity of the load.

(5) The load blocks of each crane must be positioned relative to the load's center of gravity according to the following formula:

$$\begin{pmatrix}\text{Net Capacity}\\ \text{Crane 1}\end{pmatrix} \times A = \begin{pmatrix}\text{Net Capacity}\\ \text{Crane 2}\end{pmatrix} \times B$$

(6) In most cases the shape of the load will give an indication of where one of the hook blocks should be located. The crane with the greatest net capacity must always be closest to the load's center of gravity. Measure the distance of this point from the center of gravity. If you have measured distance A then:

$$B = \frac{\text{Net Capacity of Crane 1}}{\text{Net Capacity of Crane 2}} \times A$$

If you measured distance B then:

$$A = \frac{\text{Net Capacity of Crane 2}}{\text{Net Capacity of Crane 1}} \times B$$

If the calculated value of either A or B is unsuitable (e.g. if it is so long that it is beyond the end of the load) then choose another value and recalculate.

(7) Determine the share of the load weight that will be carried by each crane.

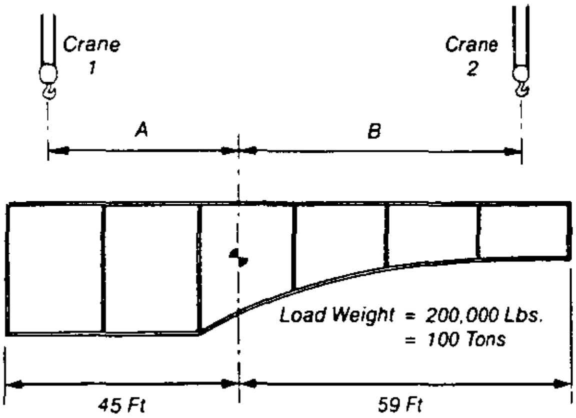
$$\text{Load on Crane 1} = \frac{B}{A+B} \times \text{Load Weight}$$

$$\text{Load on Crane 2} = \frac{A}{A+B} \times \text{Load Weight}$$

(8) Check to ensure that neither crane is loaded beyond its lowest rated net capacity. Ideally neither crane should be loaded in excess of 75% of its lowest rated net capacity.

An example of this procedure is provided on the following page.

EXAMPLE



(1) Determine lowest rated net capacity of crane 1 = 140,500 lbs.

(2) Determine lowest rated net capacity of crane 2 = 100,200 lbs.

(3) Check that total crane capacity is greater than load weight:

Crane 1 =	140,500 lbs.
Crane 2 =	100,200 lbs.
Total	240,700 lbs.

which exceeds the load weight. Ideally 75% of the total crane capacity should exceed the load weight.

(4) The load's center of gravity is located as shown on the illustration.

(5) Use the formula

$$\begin{pmatrix}\text{Net Capacity}\\ \text{Crane 1}\end{pmatrix} \times A = \begin{pmatrix}\text{Net Capacity}\\ \text{Crane 2}\end{pmatrix} \times B$$

to set the location of the hook blocks.

(6) Choose dimension A = 43 ft.

$$\text{Calculate } B = \frac{\text{Net Capacity Crane 1}}{\text{Net Capacity Crane 2}} \times A$$

$$= \frac{140{,}500 \text{ lbs.}}{100{,}200 \text{ lbs.}} \times 43 \text{ ft.}$$

$$= 60.3 \text{ ft.}$$

This won't work because the hook block would be beyond the end of the load. Choose another value for A = 40 ft.

$$\text{Recalculate } B = \frac{\text{Net Capacity Crane 1}}{\text{Net Capacity Crane 2}} \times A$$

$$= \frac{140{,}500 \text{ lbs.}}{100{,}200 \text{ lbs.}} \times 40 \text{ ft.}$$

$$= 56.1 \text{ ft.}$$

This dimension fits the load.

(7) Determine the actual load that each crane will be carrying:

$$\text{Load on Crane 1} = \frac{B}{A+B} \times \text{Load Weight}$$

$$= \frac{56.1}{40+56.1} \times 200{,}000 \text{ lbs.}$$

$$= \frac{56.1}{96.1} \times 200{,}000 \text{ lbs.}$$

$$= 116{,}753 \text{ lbs.}$$

This load is less than crane 1's rated net capacity of 140,500 lbs. but is more than 75% of the capacity (75% of 140,500 lbs. = 105,375 lbs.).

$$\text{Load on Crane 2} = \frac{A}{A+B} \times \text{Load Weight}$$

$$= \frac{40}{40+56.1} \times 200{,}000 \text{ lbs.}$$

$$= \frac{40}{96.1} \times 200{,}000 \text{ lbs.}$$

$$= 83{,}247 \text{ lbs.}$$

We could also have calculated this load as follows:

Load on Crane 2 = Load Weight – Load on Crane 1
= 200,000 lbs. – 116,753 lbs.
= 83,247 lbs.

This load is less than crane 2's rated net capacity of 100,200 lbs. but is more than 75% of the capacity (75% of 100,200 lbs = 75,150 lbs.).

(8) Since neither crane is overloaded the lift can proceed with A = 40 ft. and B = 56.1 ft. but it must be recognized that both cranes are loaded beyond the guideline of 75% of capacity. If one or both of the machines could be replaced with higher capacity units the target could be met. If other cranes are unavailable the lift could proceed but only with extreme caution.

SHARE OF LOAD CAN CHANGE DURING LIFT

The share of the load weight between two cranes calculated previously depends on *both* cranes carrying the load throughout the lift.

When the load is first lifted off the ground and when it is finally placed it is possible for only one of the cranes to be loaded. This changes the load sharing completely and can grossly overload one of the cranes.

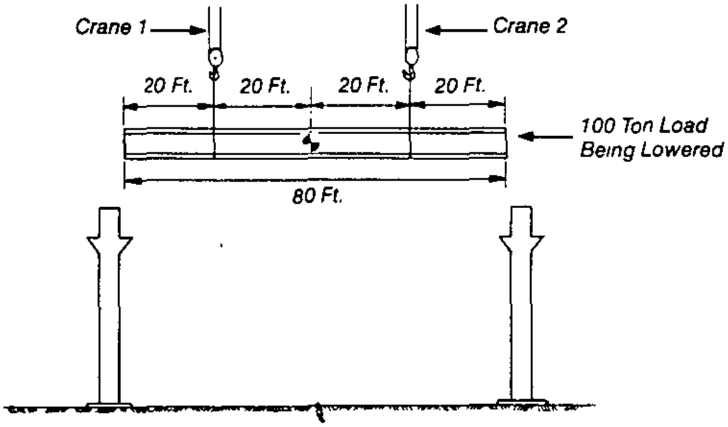
Starting the Lift: If one end of the load is lifted even slightly off the ground before the other end the *leading crane can be overloaded.* Both cranes must take the load at the same time if the load sharing is to be maintained.

Placing the Load: If one end of the load is placed even slightly ahead of the other, the lagging crane (the one placing its end last) can be overloaded. Both cranes must unload at the same instant if the load sharing is to be maintained.

EXAMPLE OF CHANGE OF LOAD SHARING

DURING LIFT

Load on Crane 1 = 50 Tons
Load on Crane 2 = 50 Tons

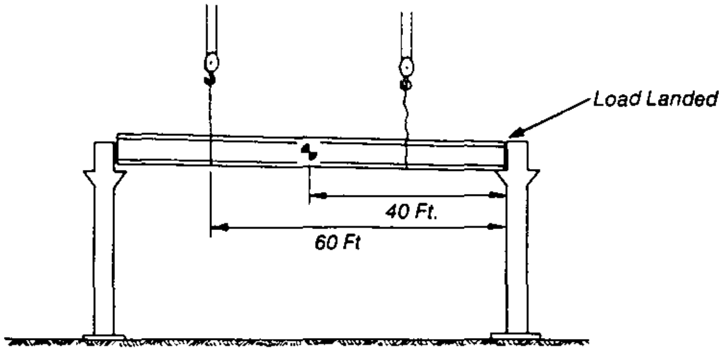


AS LOAD IS PLACED

Load on Crane 2 = Nil

Load on Crane 1 $= \dfrac{40\ ft.}{60\ ft.} \times 100\ Tons$

$= 66.66\ Tons$



CHANGE IN SHARE OF LOAD

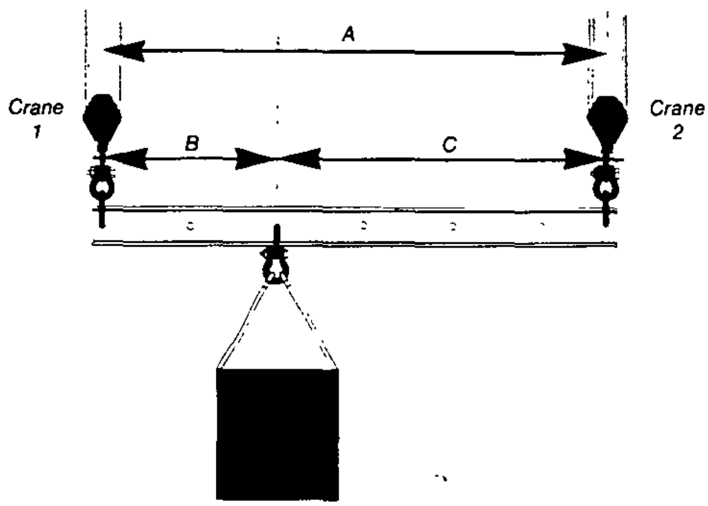
The load on crane 1 has increased from 50 tons to 66.66 tons. This 16.66 ton increase is applied as a shock load and also represents a 33⅓% increase in loading. Conditions such as this are very likely to cause a failure.

> On dual lifts use extreme caution to ensure that both cranes are loaded and unloaded simultaneously.

10.22 USE OF EQUALIZER BEAMS ON DUAL LIFTS

Equalizer beams distribute the weight of a load to two cranes. By adjusting the crane and/or load attachment points on the beam it is possible to share the load weight between the cranes in any proportion desired.

The method for properly establishing the point on the equalizer where the load is to be picked up so that each crane shares the load in proportion to its capacity is as follows:



(1) Determine the *lowest* value of rated net capacity that crane 1 will have during the *whole operation* considering all factors such as configuration, quadrant of operation, boom length, boom angle, load radius and all capacity deductions. Remember that the load radius of each crane is measured to its pick-up point on the equalizer, not to the center of gravity of the load.

(2) Repeat step (1) for crane 2.

(3) Check to ensure that the sum of the lowest rated net capacities of the cranes exceeds the load weight plus the equalizer weight. Ideally, 75% of the sum of their capacities should exceed the weight.

(4) Measure the distance between the crane pick-up points on the equalizer beam = A.

(5) Establish the attachment point of the load on the equalizer beam according to the following formulas:

$$B = \frac{\text{Net Capacity of Crane 2}}{\text{Total Net Capacity of Cranes 1 \& 2}} \times A$$

$$C = \frac{\text{Net Capacity of Crane 1}}{\text{Total Net Capacity of Cranes 1 \& 2}} \times A$$

(6) When the equalizer pick-up points have been established determine the share of the load weight including the weight of the equalizer that will be carried by each crane:

$$\text{Load on Crane 1} = \frac{C}{A} \times \text{(Load Weight)}$$

$$\text{Load on Crane 2} = \frac{B}{A} \times \text{(Load Weight)}$$

(7) Check to ensure that neither crane is loaded beyond its lowest rated net capacity. Ideally, neither crane should be loaded in excess of 75% of its lowest rated net capacity.

EXAMPLE

For the illustration shown above:

- *Load weight = 60 tons (120,000 lbs.)*
- *Equalizer weight = 2,400 lbs.*
- *Distance A = 20 ft.*
- *Lowest rated net capacity crane 1 = 100,000 lbs.*
- *Lowest rated net capacity crane 2 = 64,000 lbs.*

(1) Check total capacity of cranes.

Crane 1 = 100,000 lbs.
Crane 2 = 64,000 lbs.
Total 164,000 lbs.

This is greater than the total load weight of 122,400 lbs. (load + equalizer) and the 75% of capacity guidelines will be met. (75% of 164,000 = 123,000 lbs.)

(2) Establish load's pick-up point.

$$B = \frac{\text{Net Capacity of Crane 2}}{\text{Total Net Capacity of Cranes 1 \& 2}} \times A$$

$$= \frac{64{,}000 \text{ lbs.}}{64{,}000 \text{ lbs.} + 100{,}000 \text{ lbs.}} \times 20 \text{ ft.}$$

$$= \frac{64{,}000}{164{,}000} \times 20 \text{ ft.}$$

$$= 7.8 \text{ ft.}$$

We could also have calculated C from the other formula but it is also = A – B = 20 – 7.8 = 12.2 ft.

(3) If the equalizer beam does not have a pick-up point 7.8 feet away from crane 1, the procedure is then to choose the pick-up point that is closest to the 7.8 ft. mark. Suppose in this case that it is at the 8 ft. mark. Therefore, B = 8 ft. and C = 12 ft.

(4) Determine the load that each crane will actually be carrying.

$$\text{Load on Crane 1} = \frac{C}{A} \times (\text{Total Load Weight})$$

$$= \frac{12}{20} \times 122{,}400$$

$$= 73{,}440 \text{ lbs.}$$

which is less than its lowest rated net capacity and also meets the 75% of net load guideline.
(75% of 100,000 lbs. = 75,000 lbs.)

$$\text{Load on Crane 2} = \frac{B}{A} \times (\text{Total Load Weight})$$

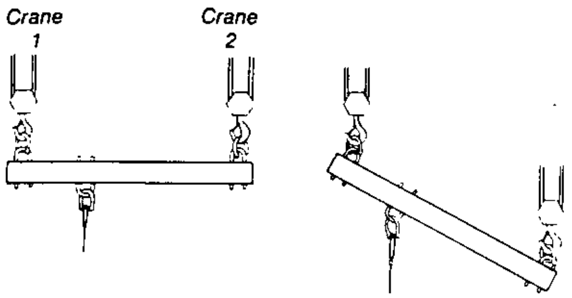
$$= \frac{8}{20} \times 122{,}400$$

$$= 48{,}960 \text{ lbs.}$$

which is less than its lowest rated net capacity and just slightly more than the 75% of net load guideline.
(75% of 64,000 lbs. = 48,000 lbs.)

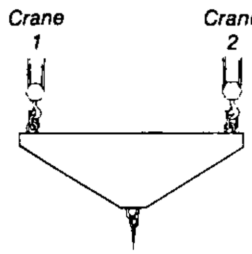
SHARE OF LOAD CAN CHANGE

The shape of the equalizer beam can influence load distribution when beam angle changes.



If the crane and load pick-up points on the equalizer beam are perfectly in-line the loading on cranes 1 and 2 does not change when the beam angle changes. If they are slightly out-of-line as shown then the loads on the cranes will change only slightly as the beam angle changes.

Because of the possibility of load redistribution between cranes when the beam angle changes, it is recommended that the beam be kept level throughout the lift regardless of shape.



When the crane and load pick-up points on the equalizer beam are considerably out-of-line as shown then the load distribution between the cranes will change significantly as the beam angle changes.

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

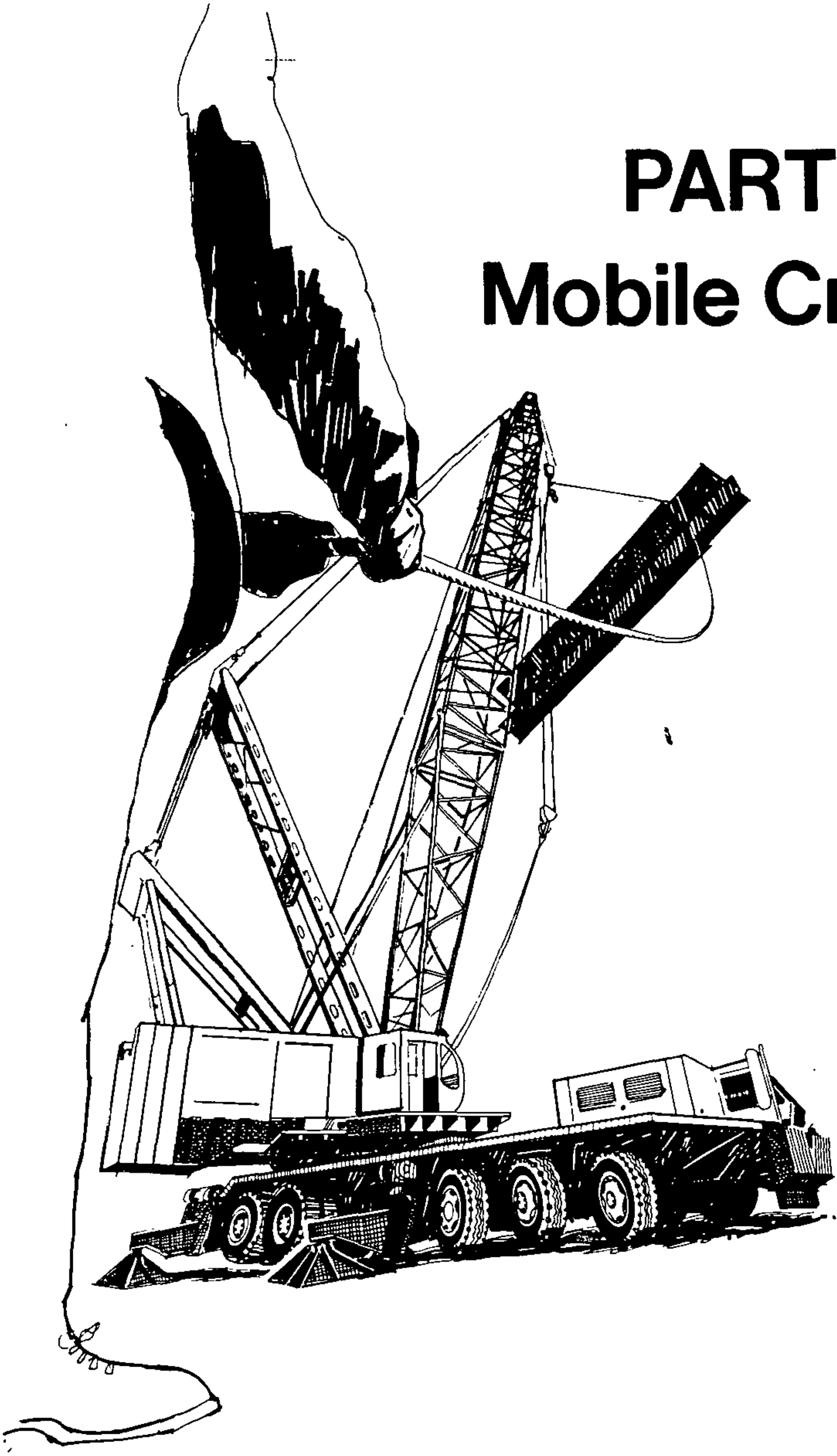
DESCRIPCIÓN Y FUNDAMENTOS

a).- GRÚAS MÓVILES

b).- GRÚAS TORRE

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

PART 1
Mobile Cranes



CHAPTER 1

MACHINE SELECTION AND EQUIPMENT REQUIREMENTS

MACHINE SELECTION

One of the basic requirements of any crane safety program involves selecting the machine to suit the requirements of the job. If the crane's basic characteristics do not match the job's requirements then unsafe conditions are created before any work is done. Job personnel are forced to "make do" and improvise in a rushed atmosphere, a combination that leads to accidents.

No machine should be selected to do any lifting on a specific job until its size and characteristics are considered against:

— The weights, dimensions and lift radii of the heaviest and largest loads. (Fig. 1.1)

— The maximum lift height and the maximum lift radius and the weight of the loads that have to be handled at them.

— The number of lifts that have to be made and with what frequency.

— The type of lifting to be done, i.e. is precision placement of loads important.

— The type of carrier required. This will depend on the ground conditions and the capacity of the machine in its operating quadrants. The maximum capacity is normally over the rear of the carrier and decreases as the boom is swung over the side. Most cranes are not designed to lift over the front and should be avoided unless capacities are supplied by the manufacturer.

— Whether or not loads will have to be walked or carried.

— Will the load have to be held in the air for lengthy periods of time.

— The site conditions, including such factors as ground conditions where the machine will be located, access roads and ramps that the machine has to travel, available space for erection, operation and dismantling, obstacles that might impede operation.

— Service availability and unit cost.

— The cost of operations such as erection, dismantling, on and off site transport, altering boom lengths.

It is recommended that the selected machine:

— Be capable of making all its lifts in its standard configuration. The machine and its main boom should be of sufficient length and capacity to do all known tasks and the jib, extra counterweight and special reeving should be held in reserve for unanticipated problems. (Fig. 1.2)

— Have at least a 5% working margin with respect to the load capacity on every lift.

— Be highly mobile and capable of being routed with a minimum amount of tear down.

— Have sufficient clearance between the load and the boom and adequate head room between the load and whatever rigging is required to make the lift. (Fig. 1.3)

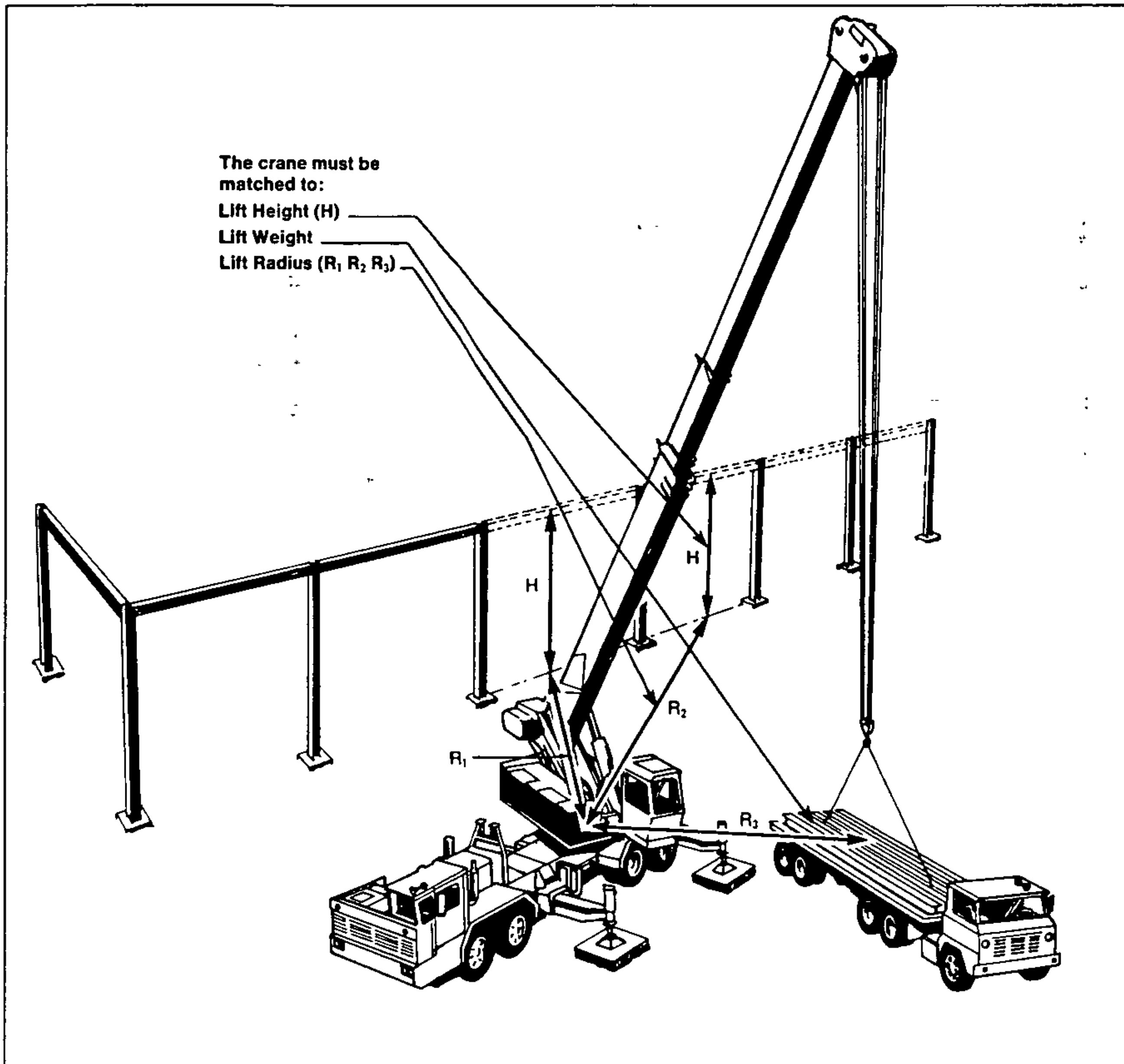
The selection of a crane or cranes for any job should be made only after a thorough examination of all the factors involved. When renting a crane be certain to let the rental agent know your requirements as their selection must be based on the data provided.

When making equipment selections, those responsible must **ensure that the unit is going to be safe and reliable for as long as it will be used** and under all anticipated conditions to which it will be exposed and operated. Nothing can take the place of experience in making these decisions, however, the guidelines set out in this section are intended to simplify the process by stressing those critical considerations that must not be overlooked.

The responsibility of equipment selection involves getting units that will not only get the job done as quickly and economically as possible, but also units that eliminate all possibility of hazard to personnel on the site, the public and the property.

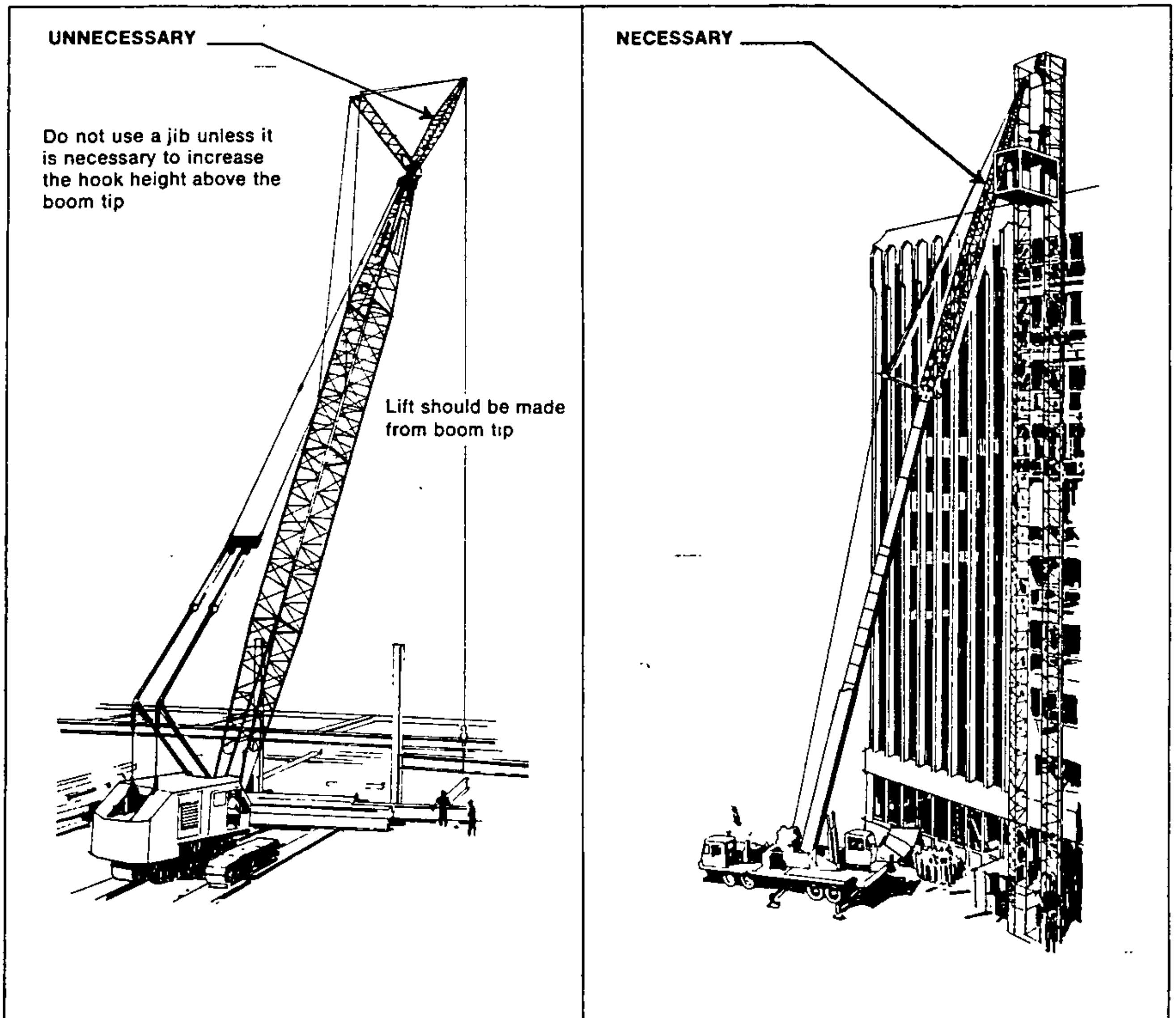
There are certain equipment considerations and requirements that apply to all cranes. These requirements can be specified in purchase orders when contracting to buy new equipment. They can be written into leasing agreements and they can be used as guidelines for equipment superintendents and equipment owners in keeping their units up to par.

Machines should be rented only from reputable firms or contractors and every effort must be made to ensure that they are in good working condition.



THE CRANE MUST BE PROPERLY MATCHED TO THE JOB.

Fig. 1.1



JIBS ARE NOT INTENDED TO INCREASE RADIUS.

JIBS ARE INTENDED TO INCREASE HEIGHT.

Fig. 1.2

With regard to rented cranes, it is important to remember that all cranes of the same model number may not have the same capacity rating. However, the correct rating can be ascertained from the manufacturer through the serial number. When the load to be lifted is close to the capacity of the crane, an enquiry should be directed to the manufacturer or his agent, giving the serial number, make and model of the crane. With this data, accurate information can be quickly obtained.

Changes in counterweight and in the type of boom inserts may have been carried out by an owner. Suspected changes of this nature should be carefully checked as they will alter the loading data given on the load chart.

EQUIPMENT REQUIREMENTS

One of the prime requirements of any crane safety program is ensuring that all necessary equipment is on the machine and that it is in good working order. If the machine has been designed, manufactured, inspected, tested and maintained in accordance with Canadian Standards Association Code Z150 — Mobile Cranes, then that is adequate assurance that all these recommendations and all Provincial and Federal Safety Regulations are met.

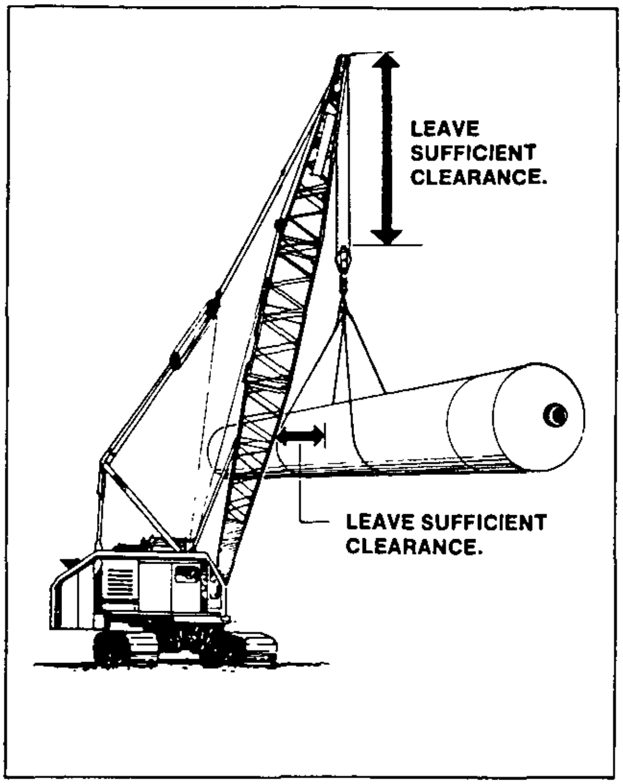


Fig. 1.3

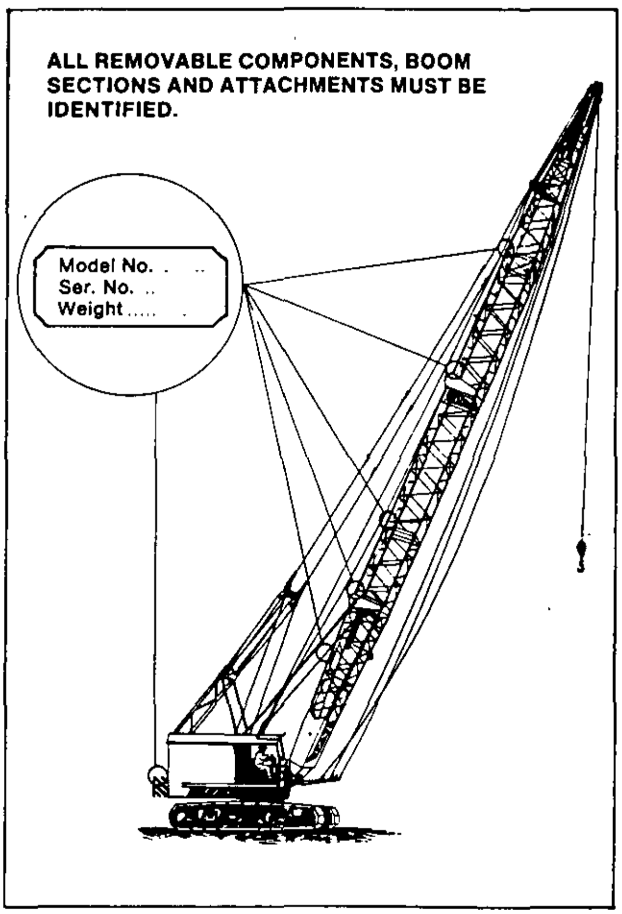


Fig. 1.4

If any of this equipment or information is missing then it is the equipment owner's responsibility to see that it is made available and put on the crane and its accessories.

Identification

Every mobile crane and carrier should have a permanent durable plate bearing the manufacturer's name, machine model number, serial number, year of original sale by the manufacturer and weight of the unit.

All basic, removable components and attachments of the machine such as outriggers, counterweights, jibs and boom sections should also be clearly identified to show that they belong with that machine. It is extremely important that these components be used only on that machine or identical models or on equipment for which they were specifically intended by the manufacturer. (Fig. 1.4)

Any components or boom sections designed and manufactured or altered by anyone other than the original equipment manufacturer or his agent must have the certificate of a qualified Professional Engineer attesting to their structural integrity to accommodate all the loads which the booms or components of the original equipment manufacturer can sustain. They must also be permanently identified in the same manner as the boom sections supplied by the original equipment manufacturer.

It is important to note that all boom sections must be capable of meeting the performance requirements of SAE J987 — Crane Structures, Method of Test and as such all other modified sections should meet the same requirements.

Load Rating Information

Every mobile crane **must** be equipped with a substantial and durable load chart with clearly legible letters and figures. It must be securely attached to the cab in a location easily visible to the operator while seated at his control station.

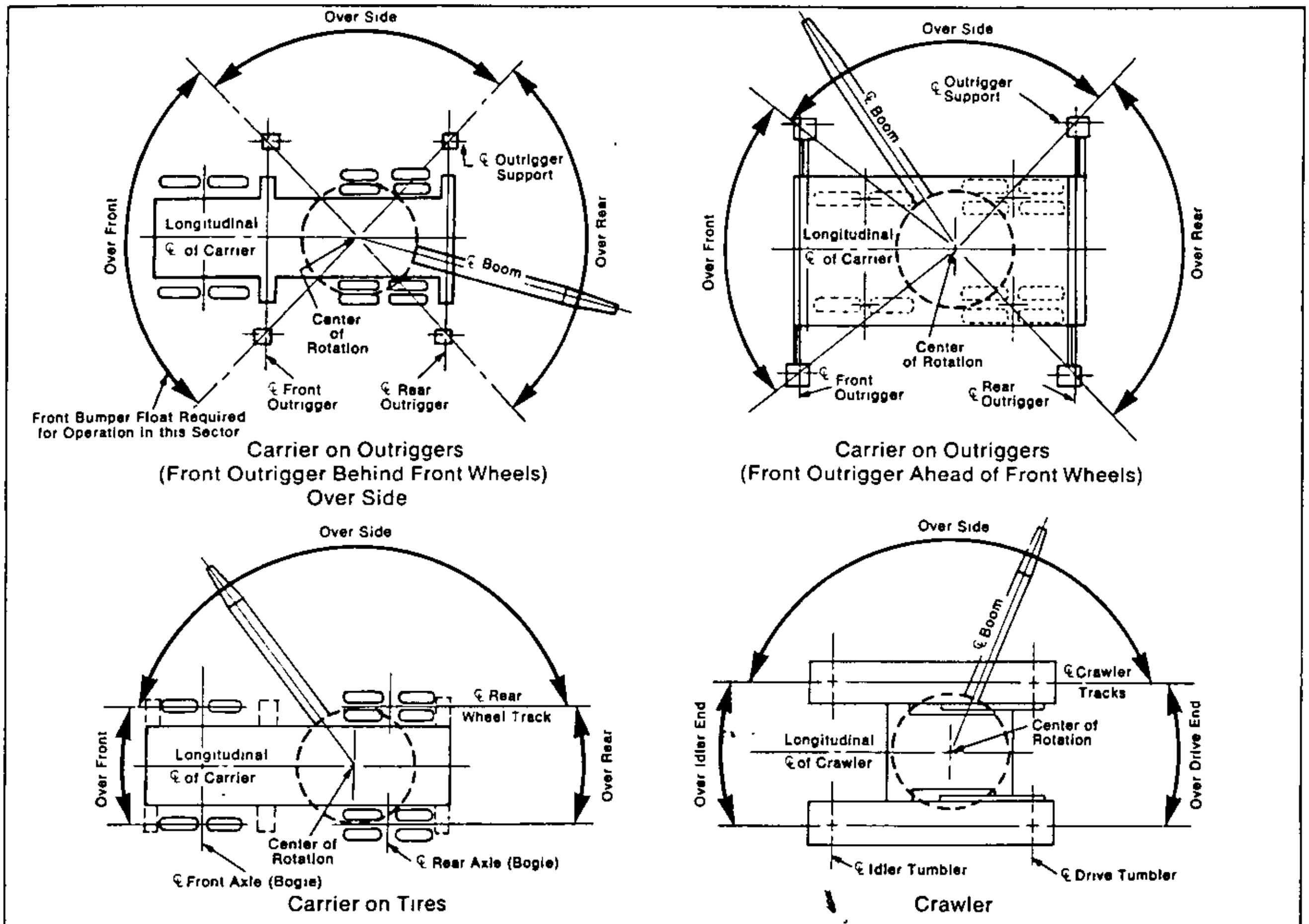
The following information must be available to the operator so that he can quickly and accurately determine the crane's capacity:

— Crane model number, serial number and date of manufacture.

— Load ratings for the main boom at all stated operating radii, boom angles, boom lengths and boom types.

— Method of determining boom-jib combination ratings.

— Jib ratings.



MOBILE CRANE QUADRANTS OF OPERATION.

Fig. 1.5

— Work areas, for which capacities are listed in the chart. The manufacturer may, at his option, list capacities for one or more of these working areas or may list capacities for any combination of working areas so long as such areas or combinations of areas are identified on the capacity chart. (Fig. 1.5)

— Alternate load ratings when using permissible, optional and variable geometry equipment on the crane, such as outriggers, gantries and extra counterweights which affect the ratings. It must be clearly indicated that outrigger ratings apply only when the outriggers are fully extended, and all wheels within the boundary of the outriggers are off the ground.

— Where the load ratings are limited by structural strength rather than stability, these ratings must be clearly identified and emphasized to separate ratings with structural limits from ratings with stability limits.

— If the specification for a crane with a non-symmetrical mounting includes additional ratings for directions other than the least stable, the directional range must be indicated.

— Adequate warning must be indicated that no allowance is made for such factors as effects of swinging loads, tackle weight, wind, degree of machine level, ground conditions, inflation of tires or operating speeds.

— Recommended parts of hoist reeving, size, and type of rope for various crane loads.

— Essential precautionary or warning notes relative to limitations on equipment and operating procedures.

— Drum data, permissible line pull, line speeds and rope spooling capacity.

— Tire pressures where applicable.

— Wind velocity operating limits.

— Low temperature operating limits.

On cranes equipped with hydraulic booms, a plate must be installed in the cab stating whether the hoist holding mechanism is automatically controlled, manually controlled, and whether or not free fall is available.

Telescopic boom information must also be included on the plate and in the operator's manual and should include:

— The maximum telescopic travel length of each boom section.
— Whether the sections are telescoped with power or manually.
— The sequence and procedure for extending and retracting the boom sections.
— The maximum loads permitted during boom telescoping operations and any limiting conditions or precautions.

Guards and Protective Structures

The owner of the crane must ensure that all exposed moving parts such as gears, pulleys, belts, chains, shafts, flywheels, etc. which might constitute a hazard under normal operating conditions are guarded or fenced. As a rule of thumb, each guard should allow for routine inspection and maintenance. Guards should be capable of supporting, without permanent distortion, the weight of a man unless located where it is impossible to step on them. Lubricating points should be accessible without the necessity of removing the guards, and all friction brakes and clutches should be protected from the weather as much as practical.

Check that all exhaust pipes are insulated in areas where the operator or maintenance personnel could accidentally be burned by coming in contact with them.

If the operator is exposed to overhead hazards from falling debris or material a substantial protective structure should be installed over the cab roof.

Cabs and Control Stations

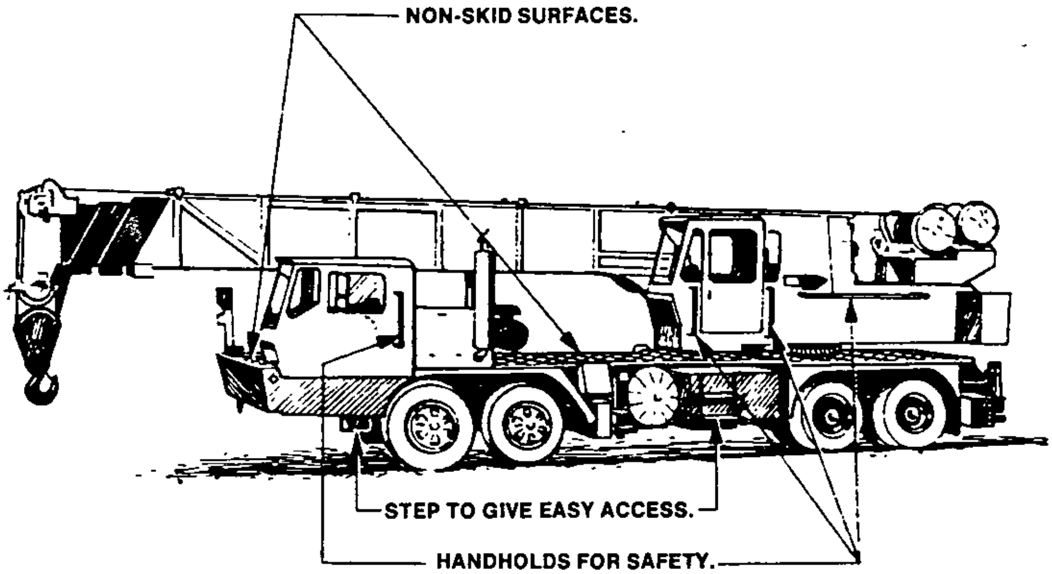
The operating cabs and control stations on all mobile cranes must, where applicable:

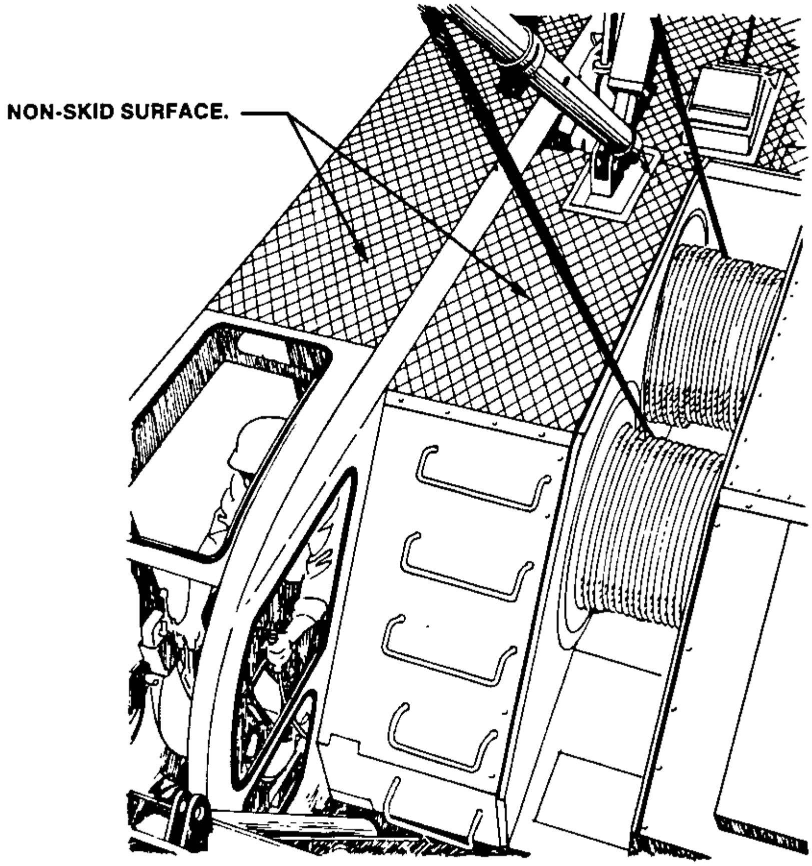
— Be designed and constructed to protect the superstructure machinery, brakes and operator's station from the weather.

Fig. 1.6

— Be designed and constructed to provide the operator a clear and unrestricted view of the load and boom point in all normal working positions, visibility to either side and as clear a view of the job site as possible. (Fig. 1.6)
— Have windows constructed of safety glass or equivalent and designed to provide ventilation as needed. The front window should have a section which can be removed or held open if desired. If the section is of the type held in the open position it must be well secured to prevent accidental closure.
— Be fitted with a lock to prevent unauthorized entry when the unit is left unattended, unless the control unit can be separately locked. The cab doors must be restrained from opening and closing accidentally whenever the machine is in use.
— Have a safe access route to and from the cab to ensure that there is no danger of the operator being trapped. This also necessitates providing a clear passageway from the operator's station to the exit door on the operator's side.
— Have a self generated noise level inside the cab, as measured at the operator's ear, not greater than 90 dBA.
— Have lighting in the cab adequate to enable the operator to see clearly enough to perform his work.
— Be provided with an operator's seat that is a fully adjustable five-way unit (fore-aft, up-down, and tilt) equipped with a suitable head rest.
— Have all walking surfaces of the anti-skid type. The most common accidents that happen to crane operators are slips and falls while walking, climbing or working on the machine. A spray-on anti-skid paint would prevent most of them if it were applied to all surfaces to which they have access. (Fig. 1.7)
— Have guardrails provided on all outside and access platforms. If they are too narrow for guardrails, hand holds should be provided at convenient points above the platform. (Fig. 1.7)
— Have hand holds and steps to facilitate entrance to and exit from the cab. (Fig. 1.7)
— Have, where necessary for rigging or service requirements, a ladder or steps installed to give access to the cab roof. (Fig. 1.7)
— Have the following accessories:

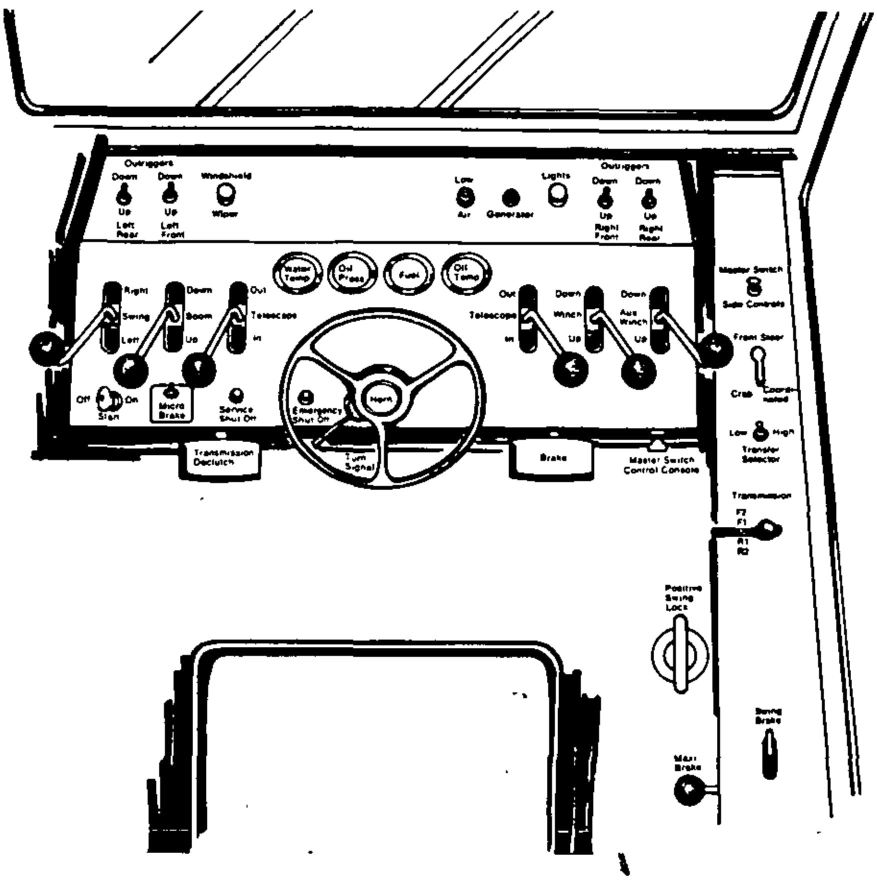
(a) Windshield wipers to cover the operator's normal viewing area, including the overhead window.
(b) A cab heater capable of maintaining the temperature in the cab at 50°F minimum when the outside temperature is —20°F.
(c) A windshield defroster.
(d) A CO_2, dry chemical or equivalent fire extinguisher. It is recommended that it be a 5:B:C type.





SLIPS AND FALLS WHILE WORKING OR CLIMBING ON THE CRANE ACCOUNT FOR MANY INJURIES.

Fig. 1.7



EVERY CONTROL MUST BE CLEARLY MARKED.

Fig. 1.8

Where applicable, engine exhaust gases must be piped to the outside of the cab and discharged in a direction away from the operator.

Operating Controls

All operating controls should meet the following requirements:

— All controls used during the normal crane operating cycle must be located within easy reach of the operator and allow him ample room for operation. The arrangement of the controls should be in accordance with SAE J983.

— The controls for load hoist, boom hoist, and swing clutches must be provided with means for holding in the neutral position without the use of positive latches.

— Each control must be clearly marked to indicate its function. (Fig. 1.8)

— All cranes must be provided with a clutch for disengaging power to the superstructure machinery. The clutch control should be within reach from the operator's station.

— All controls should be installed so as to move in the direction of the resultant load movement or machine movement. (Fig. 1.9)

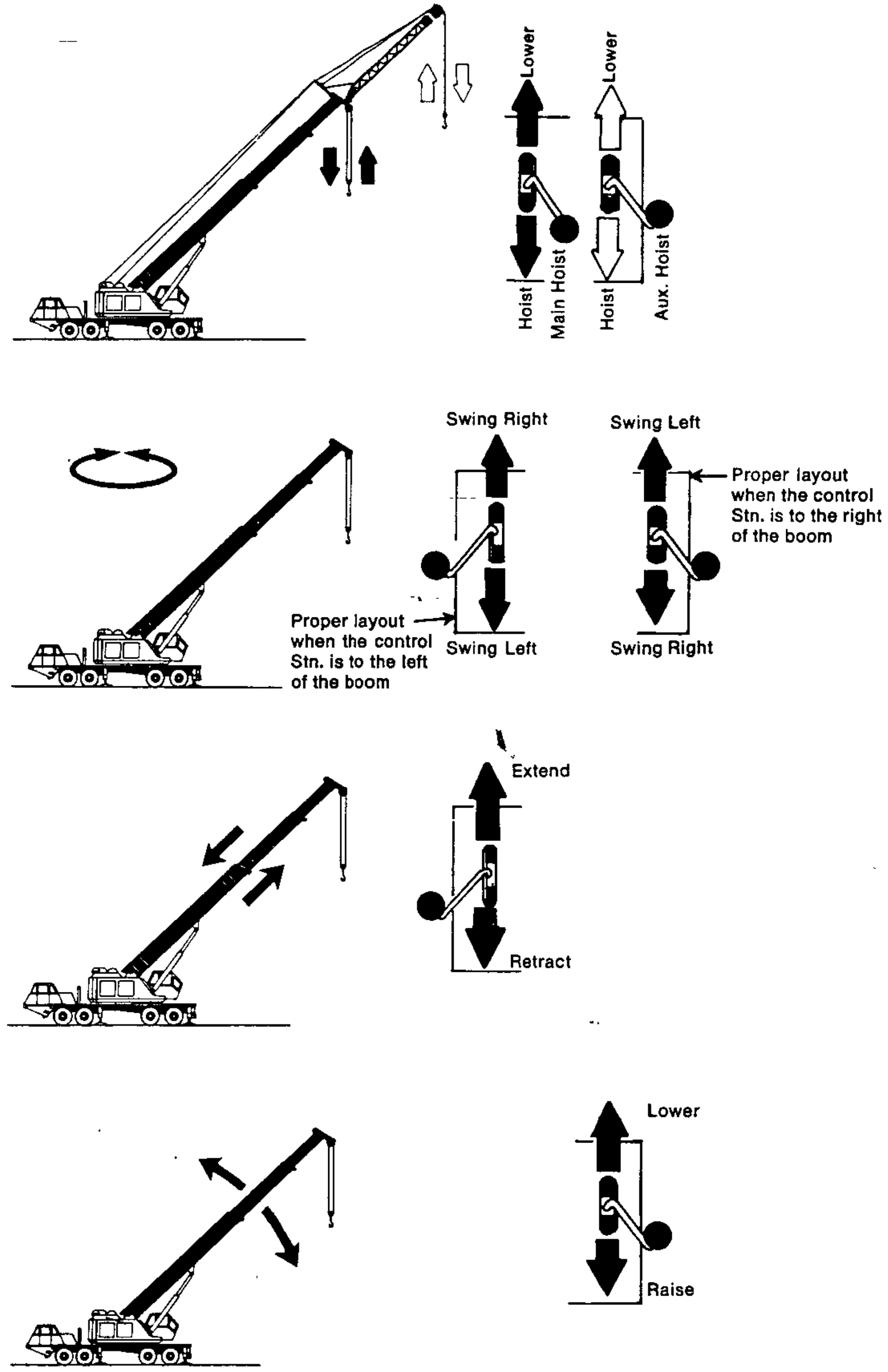
— Whenever possible, deadman controls should be provided.

— The controls must be adjusted so that the forces needed to actuate the hand controls are less than 35 lbs. and those needed to actuate the foot controls are less than 50 lbs. In addition, the travel distance on hand levers must not exceed 24 inches on 1-way levers and 14 inches on 2-way levers (from the neutral to the engaged positions). The travel distance on foot pedals must not exceed 10 inches.

Drum Assemblies

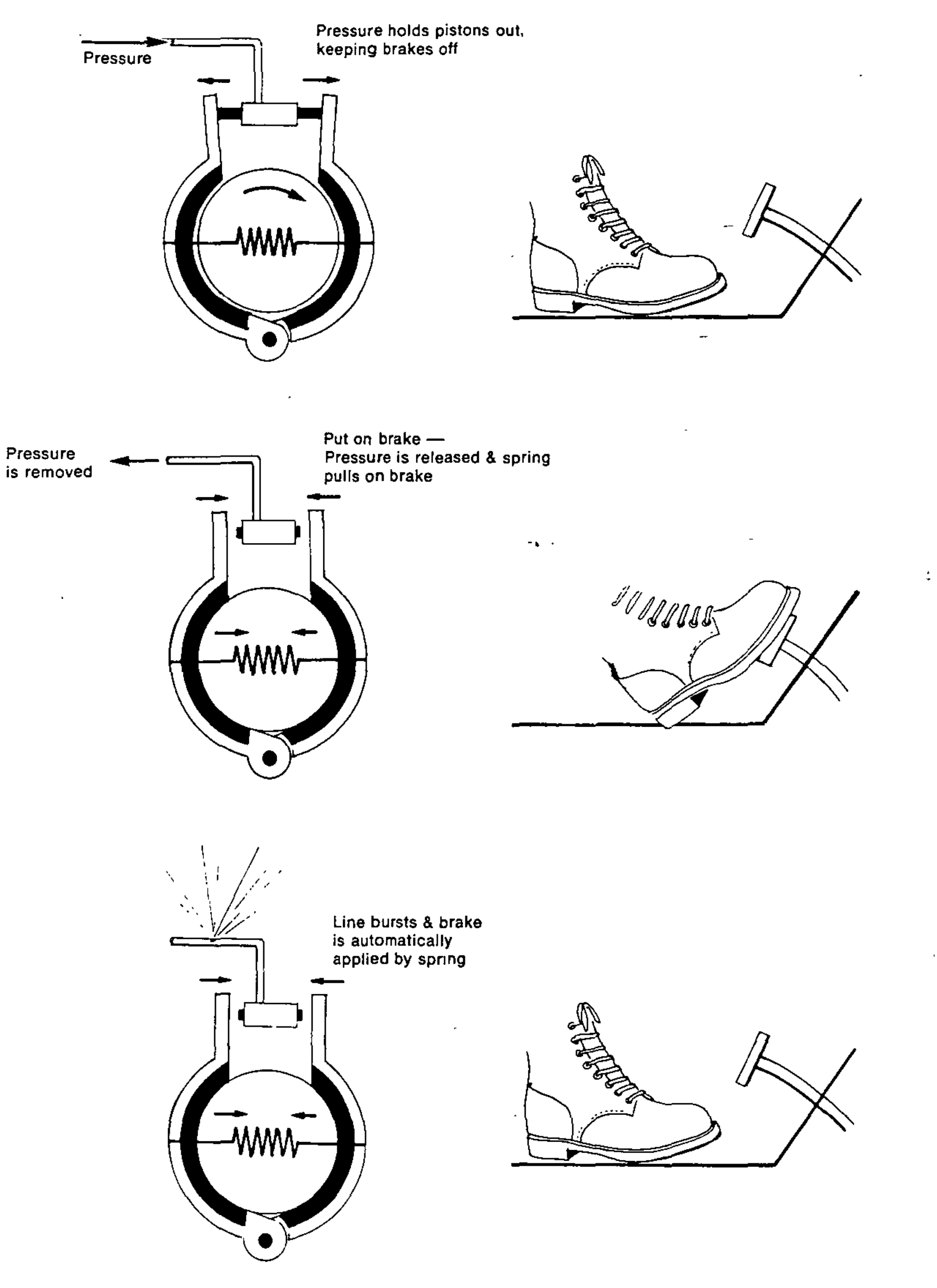
With regard to the drum assemblies, ensure that:

— They have adequate power and operational characteristics to perform all hoisting, holding and lowering functions when operated under all conditions and configurations as recommended and approved by the crane manufacturer.



ALL CONTROLS SHOULD MOVE IN THE DIRECTION OF THE LOAD AND MACHINE.

Fig. 1.9



FAIL-SAFE BRAKE OPERATION.

Fig. 1.10

— They are provided with suitable clutching or power engaging devices that facilitate immediate starting and stopping of the drum motion.

— They are provided with self-setting fail-safe brakes that are capable of supporting all rated loads with recommended reeving and operate automatically should power fail. (Fig. 1.10)

— Their brakes and clutches are provided with adjustments to compensate for wear and maintain adequate force in springs, where used.

— All boom hoisting mechanisms must also be provided with an auxiliary ratchet and pawl or other positive locking device. (Fig. 1.11)

— The drums have sufficient rope capacity with recommended rope size and reeving to perform all hoisting and lowering functions under recommended and actual service conditions. In addition, all hoist drums should be provided with adequate means to ensure even spooling of the rope on the drum.

— At least three full wraps of rope remain on the drum in all service conditions. (Fig. 1.12)

— The drum end of the rope is anchored by a clamp securely attached to the drum with an arrangement approved by the crane or rope manufacturer.

— The drums are provided with rims and flange guards of size sufficient to prevent the rope from jumping off the drum.

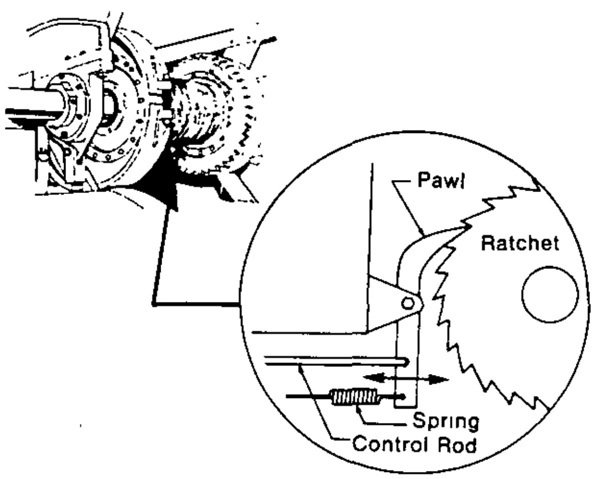
— The diameter of the load hoist drums are sufficient to provide a first layer rope pitch diameter of not less than 18 times the nominal diameter of the rope used and the diameter of the boom hoist drum must be sufficient to provide a first layer rope pitch diameter of not less than 15 times the nominal diameter of the rope used. (Fig. 1.13)

— Grooved drums have the correct minimum groove pitch for the diameter of the rope. The depth of the groove must also be correct for the diameter of the rope.

— The flanges on grooved drums must project, either 2 times the rope diameter or 2 inches beyond the last layer of rope, whichever of the two is the greater. (Fig. 1.14)

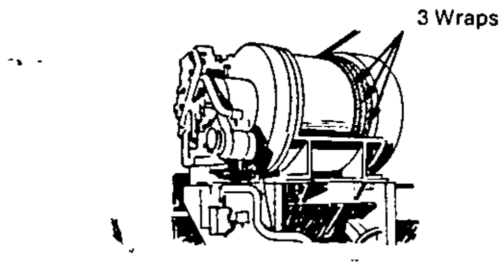
— The flanges on ungrooved drums must project either 2 times the rope diameter or 2½ inches beyond the last layer of rope, whichever of the two is the greater. (Fig. 1.14)

— The fleet angle for grooved drums should lie between ¼° and 1¼°, and for smooth drums it should lie between 1° and 2°. (Fig. 1.15, 1.16, 1.17)



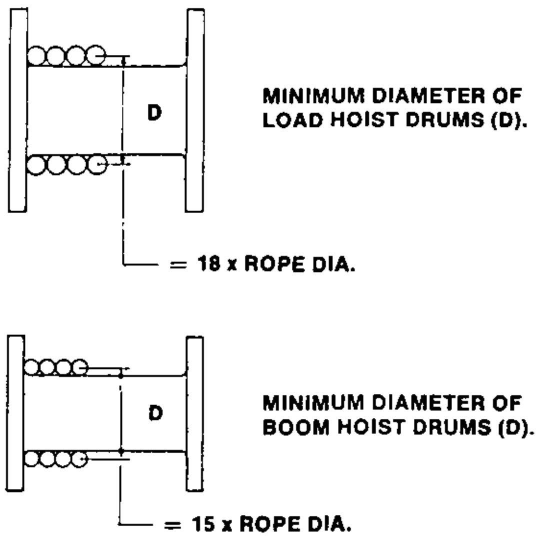
ALL BOOM HOISTS ON FRICTION MACHINES MUST BE EQUIPPED WITH POSITIVE LOCKS.

Fig. 1.11



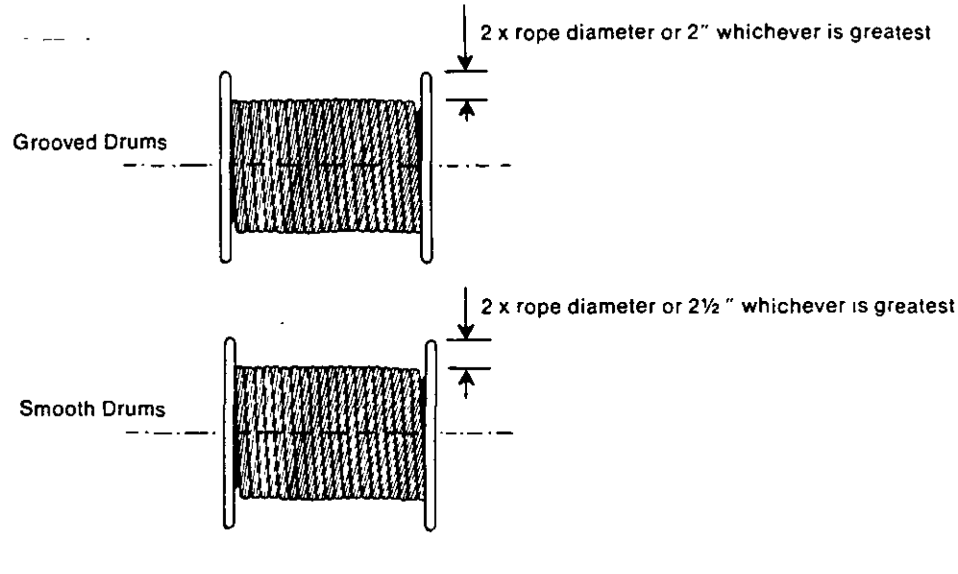
NEVER LESS THAN 3 FULL WRAPS UNDER ANY SERVICE CONDITION.

Fig. 1.12



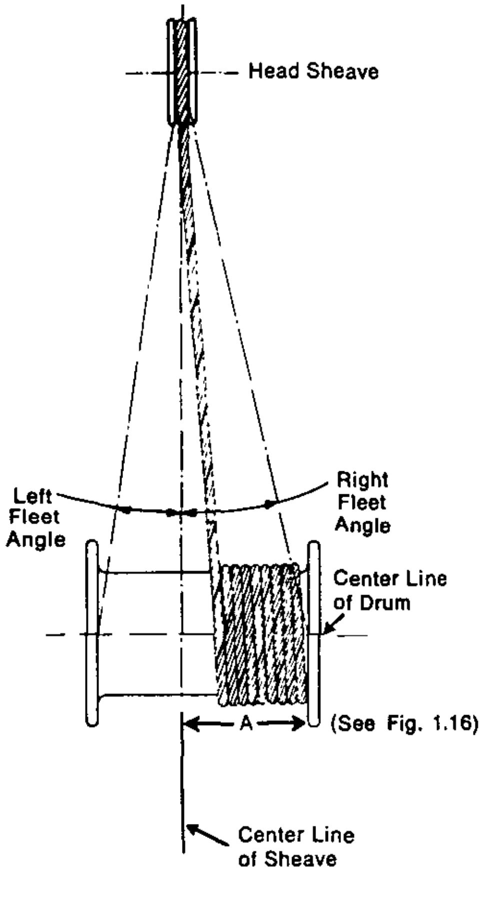
DRUM DIAMETER REQUIREMENTS.

Fig. 1.13



THE TOP LAYER OF ROPE MUST NOT BE CLOSER TO THE FLANGE TIPS THAN SHOWN.

Fig. 1.14



FLEET ANGLE RELATIONSHIPS.

Fig. 1.15

— Drum rotation indicators should be provided and located to afford easy sensing by the operator.

Ensure that the rope is properly installed on the drum. Information on this subject and other precautions relating to drums is provided in the Construction Safety Association of Ontario "Rigging Manual".

Brakes

— If the brakes are not mechanically linked to the pedals or control levers (pneumatic, hydraulic and electrical brakes) then the braking system must be designed so that the brakes will be automatically applied if there is a loss of power or pressure. These brakes must not release until the power has been restored to the machine and then only when deliberately released.

— Foot-operated brake pedals must be constructed so that the operator's feet will not easily slip off. They should also be equipped with a means for latching them in the applied position.

— All load hoisting drums should be equipped with fail-safe brakes capable of being released electrically, hydraulically or pneumatically. The application of this type of brake must have a **direct** effect on the hoisting drum and as such no belts or chains are permitted between the brake and the drum.

— All load holding brakes and clutches must have sufficient size and thermal capacity to control all rated hoist loads with minimum recommended reeving.

— The wearing surfaces of all brake drums must be smooth and free from all defects and the assemblies should have accessible devices to compensate for the wear of the linings.

— If the unit is equipped with travelling brakes they must, in addition to stopping the unit, have sufficient size to prevent the unit from moving during high winds.

- Swing brakes should consist of a fail-safe mechanical unit capable of not only stopping the swing with full load but also holding the full boom in winds of up to 30 mph. The brake must be capable of being set in the holding position and remaining that way without further attention from the operator.

— The crane must also be equipped with a positive swing lock or house lock that is designed to prevent accidental engagement or disengagement. This lock should be used when travelling the crane and when moving over rough terrain with heavy loads suspended from the boom.

— On mobile cranes the brakes must be capable of holding the machine stationary under normal working conditions, and on the maximum grade for travel recommended by the manufacturer. These brakes must also be fail-safe so as to be able to operate in the event of loss of operating pressure.

— On wheel mounted cranes, the brakes must be capable of bringing the machine to a stop on level ground within a distance of 32 feet from a speed of 15 mph and holding the machine stationary on the maximum grade for travel recommended by the manufacturer. Where these brakes are operated by air pressure, they must also have the ability to stop the vehicle if the operating pressure falls below the specified minimum level.

— Where long or steep grades are to be negotiated, a retarder or similar device such as a Jacobs brake should be provided.

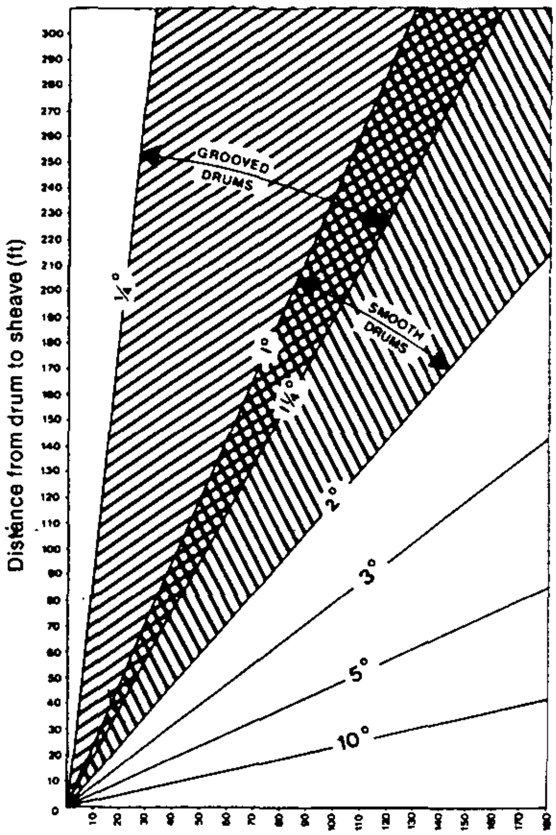
Ropes, Rigging, Reeving and Accessories

For complete information on these items please refer to the Construction Safety Association of Ontario "Rigging Manual".

Sheaves

The condition and contour of sheave grooves have a major influence on rope life. The grooves must be smooth and slightly larger than the rope to prevent it from being pinched or jammed in the groove. Since most ropes are made slightly larger than their nominal size, the sheave grooves for new rope should just accommodate the full oversize of the rope, as given in the following table:

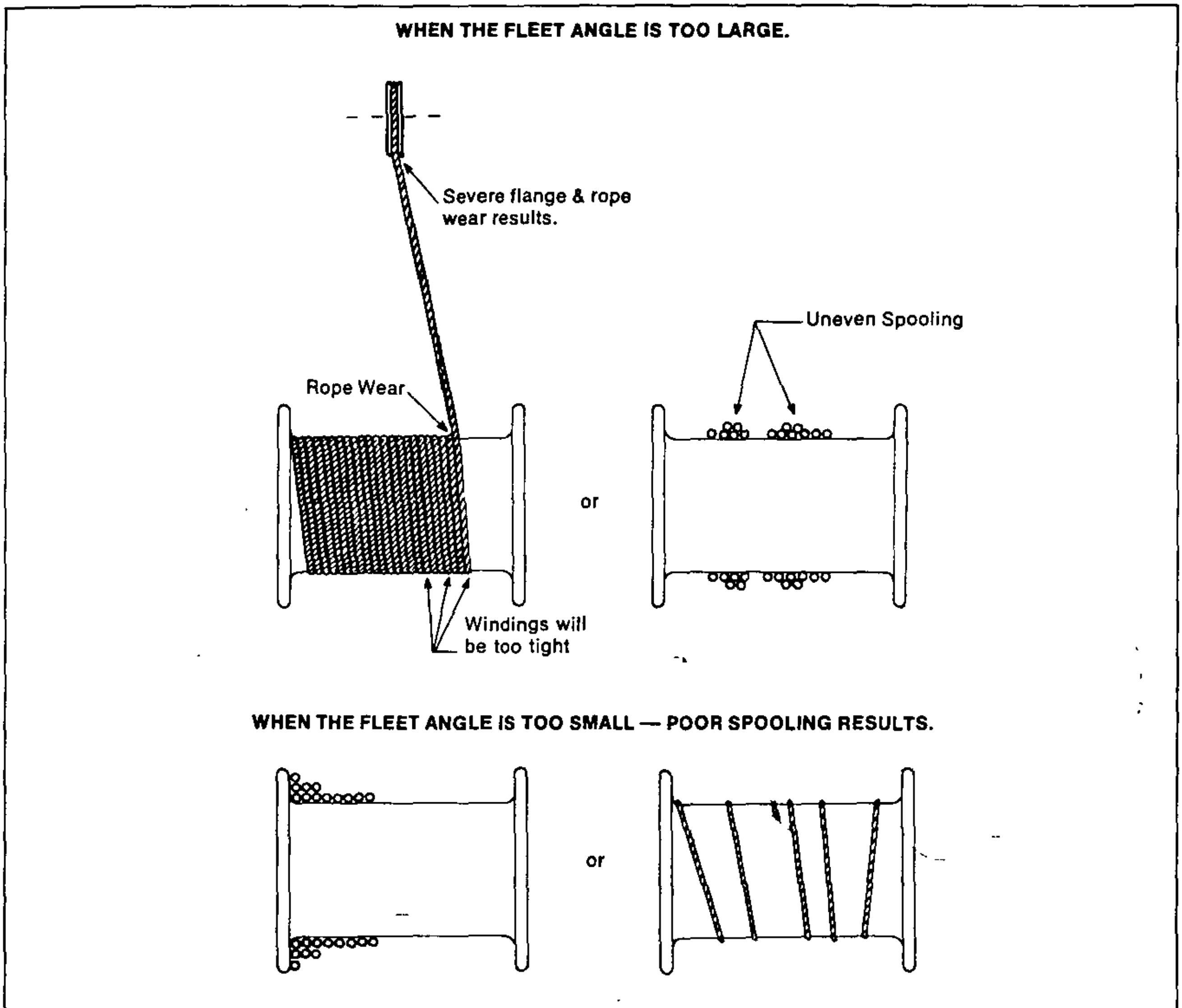
PROPER FLEET ANGLE RANGES FOR GROOVED AND SMOOTH DRUMS.



Drum width (inches) between flanges. If sheave is located on center line of drum or 2 x A (see Fig. 1.15) if sheave is offset from center line of drum.

Fig. 1.16

SHEAVE GROOVE TOLERANCES		
Nominal Rope Diameter (Inches)	Groove Oversize (Inches)	
	Min.	Max.
1/4 - 5/16	1/64	1/32
3/8 - 3/4	1/32	1/16
13/16 - 1-1/8	3/64	3/32
1-3/16 - 1-1/2	1/16	1/8
1-9/16 - 2-1/4	3/32	3/16
2-5/16 & up	1/8	1/4



EFFECT OF FLEET ANGLE ON SPOOLING.

Fig. 1.17

When the groove diameter is slightly larger than the nominal rope diameter it provides maximum support for the rope. If the groove is too large it cannot provide adequate support and the rope may become flattened. Alternatively, too small a groove will pinch and bind the rope, causing abrasion and burning. (Fig. 1.18)

The diameter of a rope diminishes in size after lengthy service due to abrasion and loss of core support. An undersized rope may wear an undersized groove in the sheave and if a new rope is installed in the worn groove it may become wedged between the flanges. (Fig. .19)

To ensure a long and efficient rope life the grooves should be smoothly contoured, free of surface defects, and possess rounded edges.

The bottom of the groove should have an arc of support of at least 120° to 150°, and the sides of the grooves should be tangent to the arc (Fig. 1.20). The more closely the contour of the groove approaches that of the wire rope the greater becomes the area of contact between the two. This minimizes rope distortion, bending fatigue and eases sheave rotation. If the groove diameter is too large, the rope will not be properly supported and will tend to flatten and become distorted. This accelerates bending fatigue in individual wires and can cause premature failures.

If the sheave groove is too narrow for the rope the operating tension will draw the rope deeply into the groove, causing it to be pinched and subjecting both the rope and sheave to severe abrasive wear. This condition can arise if new ropes are installed over old sheaves.

If the sheaves are not perfectly aligned both the rope and sheave flanges will be subjected to severe wear and rapid deterioration will occur. A ready indication of poor alignment is rapid wear of only one of the flanges on any given sheave. (Fig. 1.21)

One of the fastest ways to ruin a rope is to operate it over small sheaves. The excessive and repeated bending and straightening of the wires leads to premature failure from fatigue. Use the maximum possible diameter of sheave that the equipment will carry.

Boom hoisting sheaves must have pitch diameters of not less than 15 times the nominal diameter of the rope used. (Fig. 1.22)

Load hoisting sheaves must have pitch diameters not less than 18 times the nominal diameter of the rope used, and load block sheaves must have pitch diameters not less than 16 times the nominal diameter of the rope used. (Fig. 1.22)

The depth of the sheave grooves should be at least 1½ times the rope diameter and the tapered side walls of the grooves should not make an angle of more than 18° with respect to the centre line. The flange corners should be rounded and the rims should run true about the axis of rotation. (Fig. 1.23)

The bearings should either be permanently lubricated or be equipped with means for lubrication. Inadequate lubrication or a sheave that is too heavy for the load will cause the rope to

MATCHING OF ROPES AND SHEAVES.

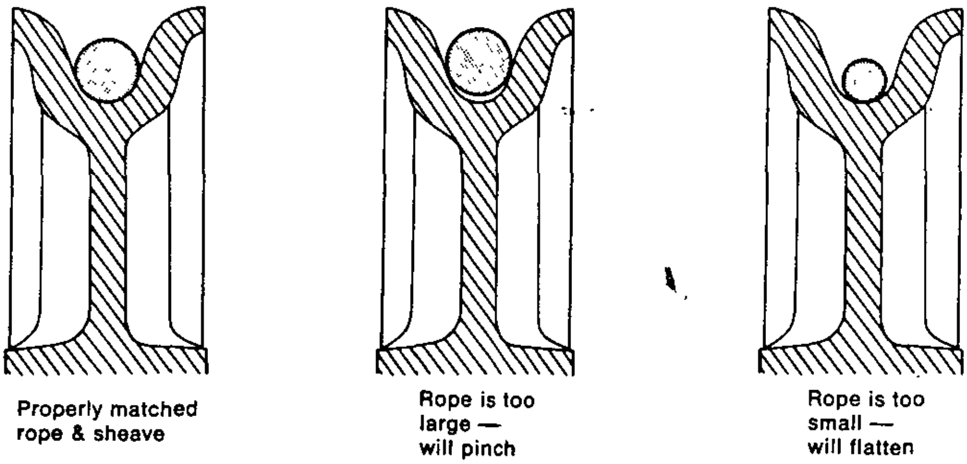
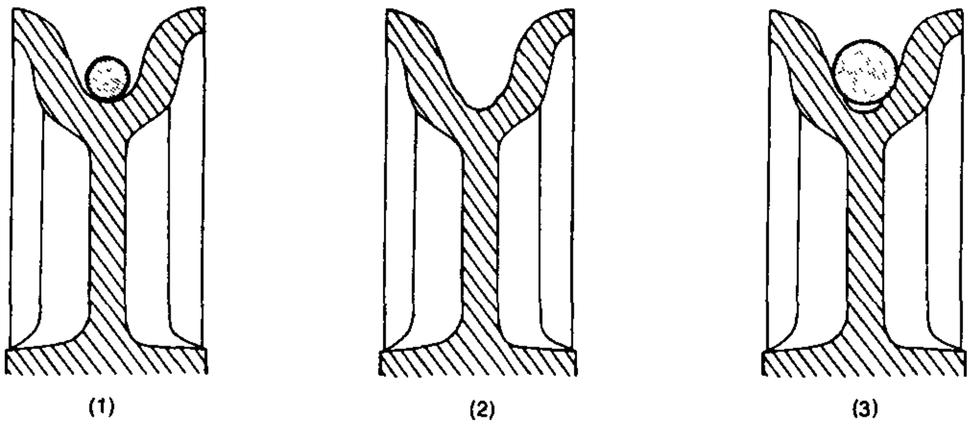


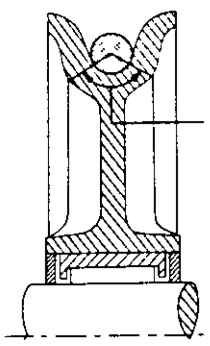
Fig. 1.18

EFFECT OF IMPROPER MATCH BETWEEN ROPE & SHEAVE.



If a rope is too small for a sheave (1) it will cut an undersized groove (2) and if another rope of the correct size is installed (3) it will be severely damaged by the undersized groove.

Fig. 1.19



PROPER ARC OF SUPPORT PROVIDED THE ROPE BY A SHEAVE.

Fig. 1.20

slip in the sheave whenever the rope velocity changes. The momentum of the heavy sheave will cause it to continue turning after the rope has stopped. This grinding wheel action causes severe rope abrasion and will wear flat spots in the sheave that further damages the rope. (Fig. 1.24)

If the sheaves are carrying ropes that can be momentarily unloaded, as in the case of a hoist line, then the sheave must be equipped with cablekeepers that prevent the unloaded rope from leaving the groove. The sheaves in lower load blocks should also be equipped with either cablekeepers or with close fitting guards that prevent the ropes from becoming fouled when the block is lying on the ground with the ropes loose. When these guards are fitted it is important that they be removed only for the purposes of maintenance, inspection or adjustment. Failure to observe this procedure may allow a rope to jump clear and become trapped. (Fig. 1.25)

Badly worn sheaves have an adverse effect on rope life and must be examined at regular intervals. When replacement becomes necessary only equipment supplied or approved by the crane manufacturer should be fitted. Some designs may allow for re-machining the grooves but this operation should only be undertaken in accordance with the manufacturer's instructions as there is a limit to the amount of metal which can be removed before the strength of the component will be affected.

Inspect the sheaves carefully for any sign of cracks in the flanges. If the flange breaks off it will allow the rope to jump free with disastrous results. If even a small portion of the flange is broken, the rope is liable to be cut completely through by the rough edge of the break.

It is also important to know that fibre ropes must never be used on pulleys or sheaves that have seen wire rope service and wire rope must never be used on equipment designed for fibre rope.

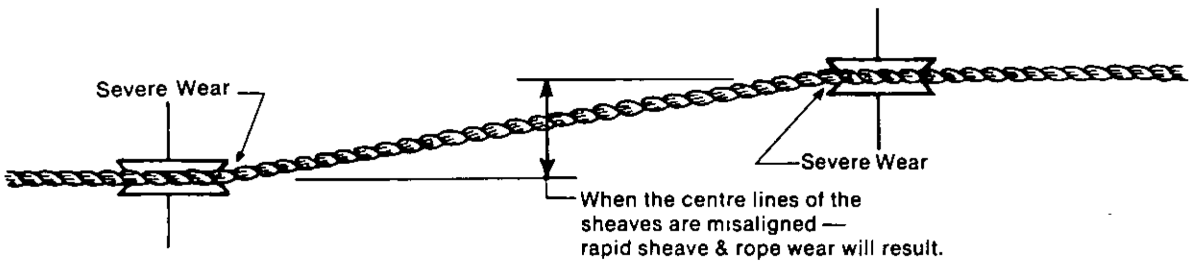
The groove surfaces on both sheaves and grooved drums and the complete surface on smooth drums should be perfectly smooth, those which have taken the imprint of the outer wires of previous ropes will exert a grinding action on new ropes (Fig. 1.26). This imprinting and scoring is caused by high contact pressures between the rope and drum surface. If this condition is evident then the drum must be re-surfaced and the contact pressure reduced by:

(a) Decreasing the load on the rope; or
(b) Increasing the drum diameter; or
(c) Replacing the drum with one having harder metal.

Outriggers

All outriggers must be capable of being securely held in the retracted position while travelling and in the extended position when blocked for hoisting. The power actuated jacks, where used, must never lose pressure or leak while under load.

Each outrigger should be visible from its actuating location unless the operator is assisted by a signalman.



EFFECT OF SHEAVE MISALIGNMENT.

Fig. 1.21

The outrigger floats must be capable of being securely fastened to the outriggers and must not be damaged.

The outrigger beams must be marked or painted in a manner to indicate the fully extended position. (Fig. 1.27)

Power Controlled Lowering

It is strongly advised that power controlled lowering be provided for the main hoist line and that it be capable of handling all rated loads and speeds specified by the manufacturer. These units are invaluable in providing precision lowering and reducing demand on the load brake.

Boom Stops

All mobile cranes should be provided with boom stops which effectively prevent the boom from toppling or being pulled backwards over the top of the cab (Fig. 1.28). Accidents of this type occur frequently due to:

— The hook block being pulled into the boom head. (Fig. 1.29)

— Travelling with the boom held at too high an angle. (Fig. 1.30)

— Carrying a high boom on sloping ground or operating on sloping ground and swinging from the low side to the high side. (Fig. 1.31)

— Dragging clutches that cause the boom drive

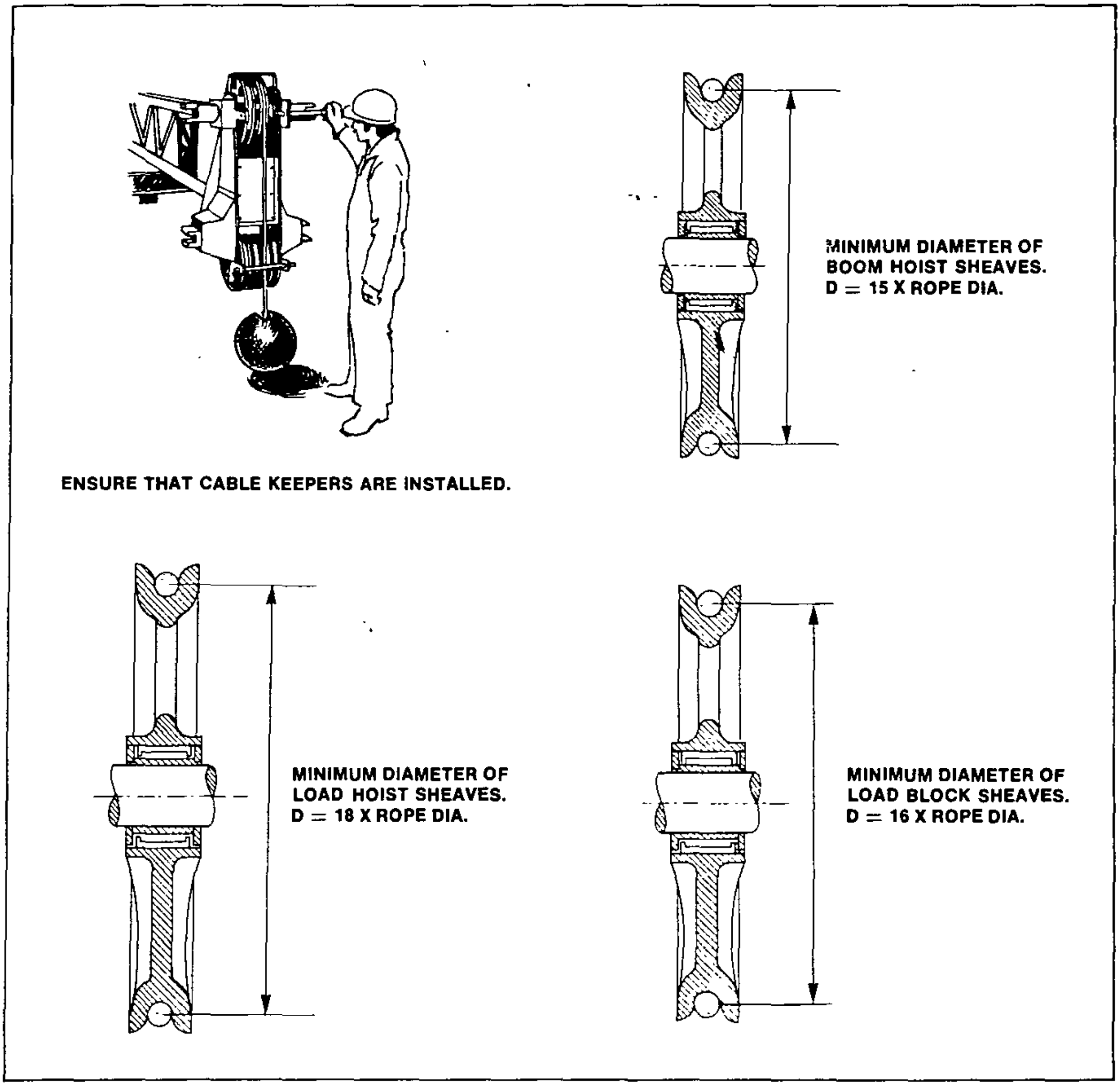
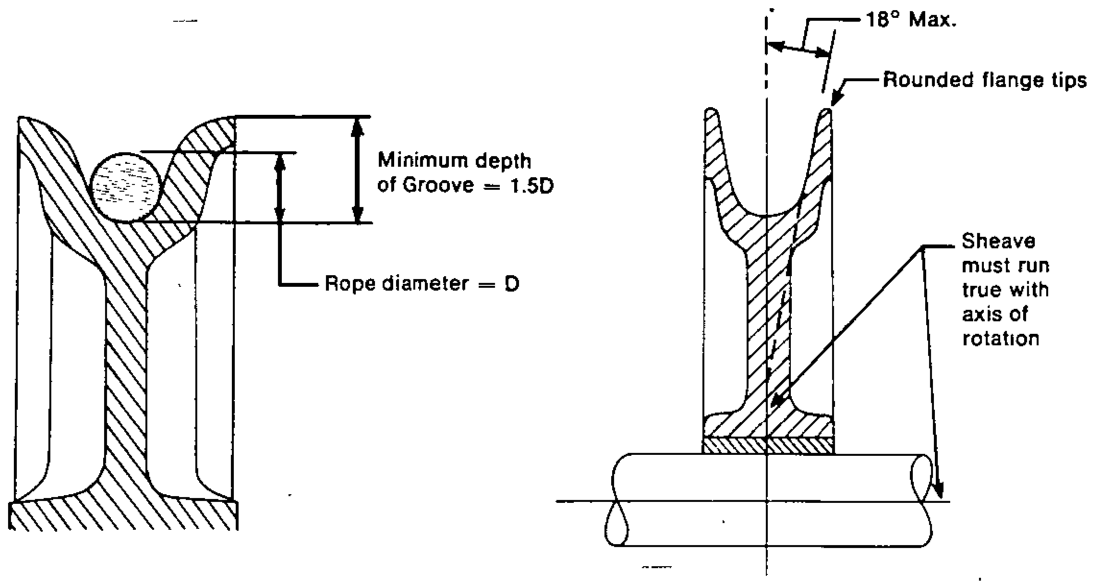
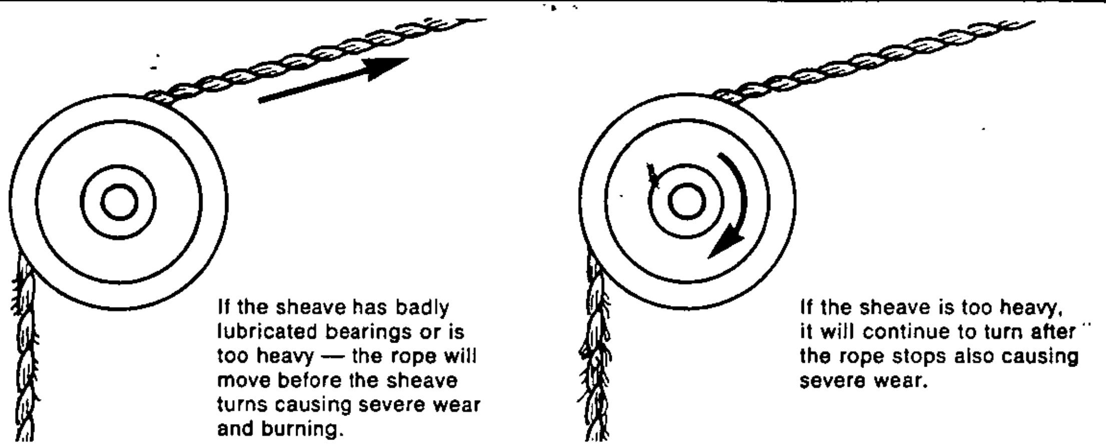


Fig. 1.22



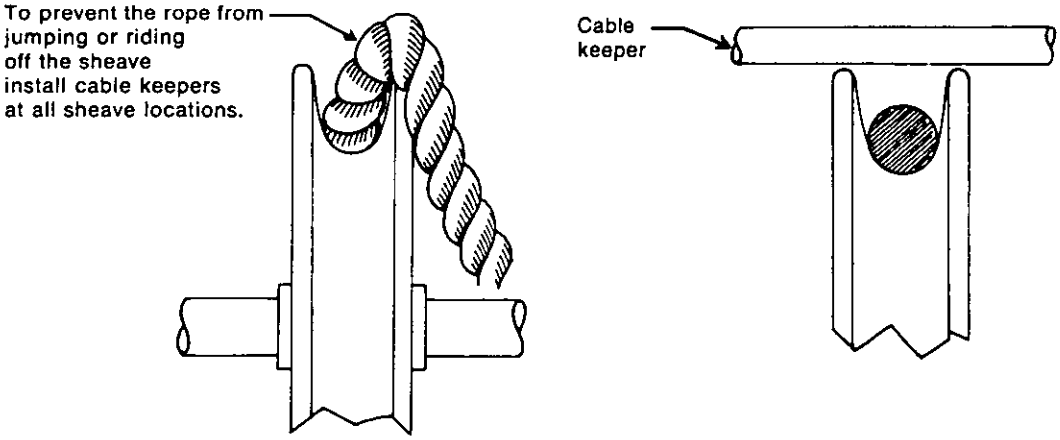
CHECK THAT ALL SHEAVES MEET THESE REQUIREMENTS.

Fig. 1.23



EFFECT OF IMPROPER SHEAVES OR POOR MAINTENANCE ON ROPE LIFE.

Fig. 1.24



SHEAVE CABLE KEEPERS.

Fig. 1.25

to be engaged even though the clutch is thought to be disengaged.

— The parting or slipping of heavily loaded slings when the boom is high. Rapid load lowering can also cause the boom to kick back when the pendant stretch and boom deflection come out suddenly. (Fig. 1.32)

— High winds acting on a high boom and forcing it past the vertical position. (Fig. 1.33)

The best type of boom stop is one that combines the functions of disengaging the master clutch and physically stopping the boom as it reaches a predetermined maximum angle.

This type of stop usually takes the form of a spring or pneumatically loaded piston running in a closed cylinder that is mounted on the A-frame or gantry to intercept the boom as high above the boom hinges as possible. When the boom reaches a predetermined high angle it trips a cut-out switch that causes the boom hoist drum to stop. (Fig. 1.34)

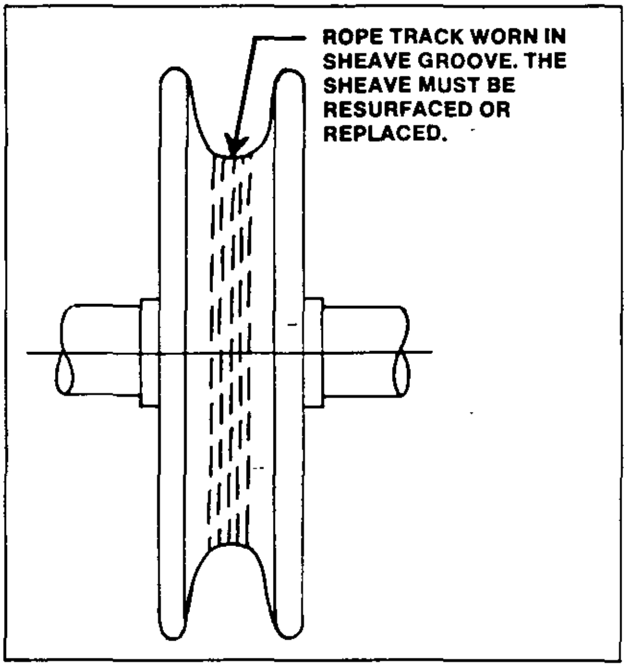
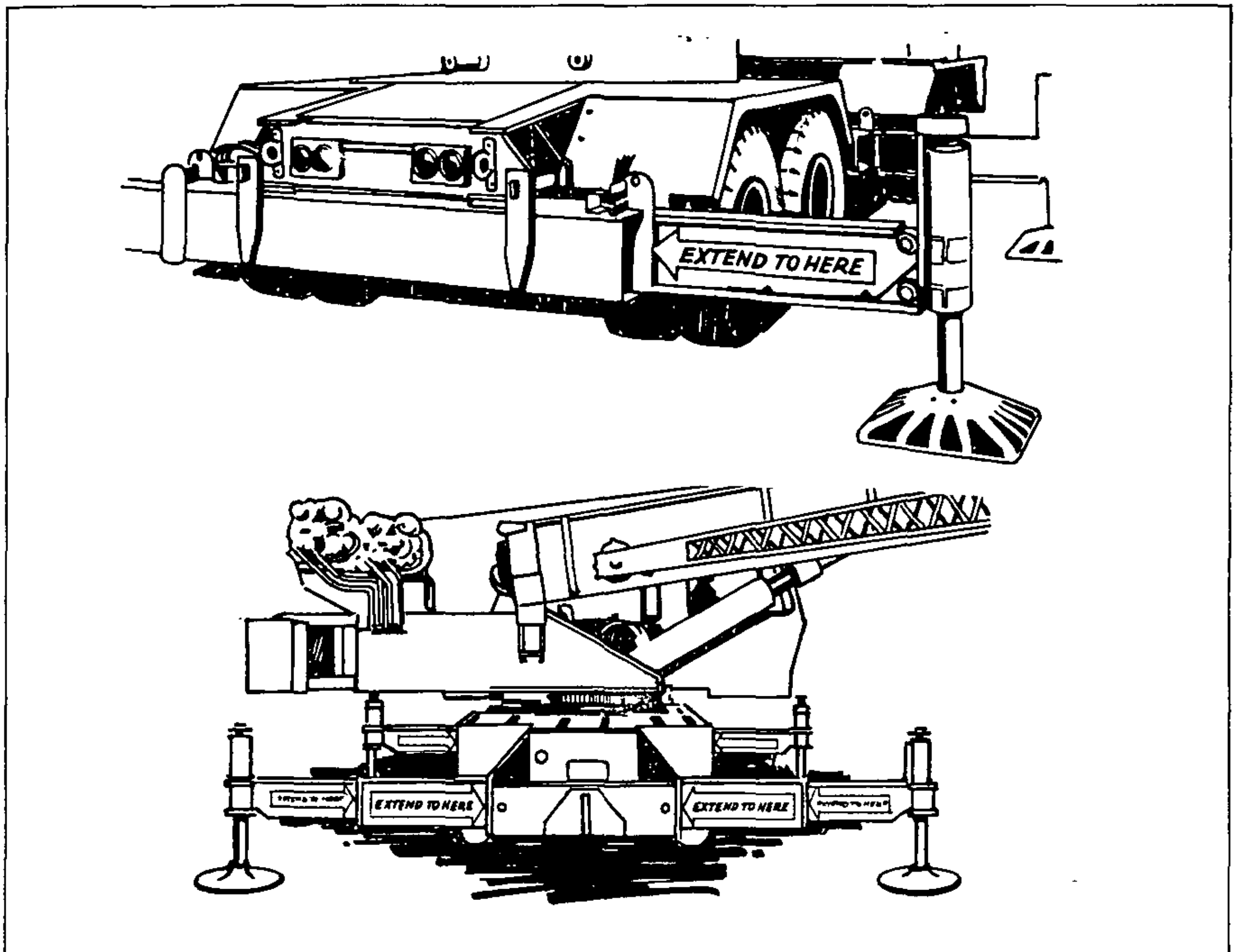
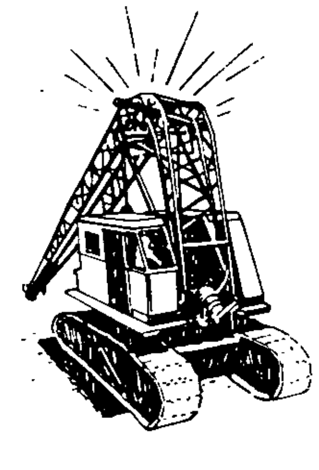


Fig. 1.26



OUTRIGGER BEAMS SHOULD BE MARKED TO SHOW FULLY EXTENDED POSITION.

Fig. 1.27

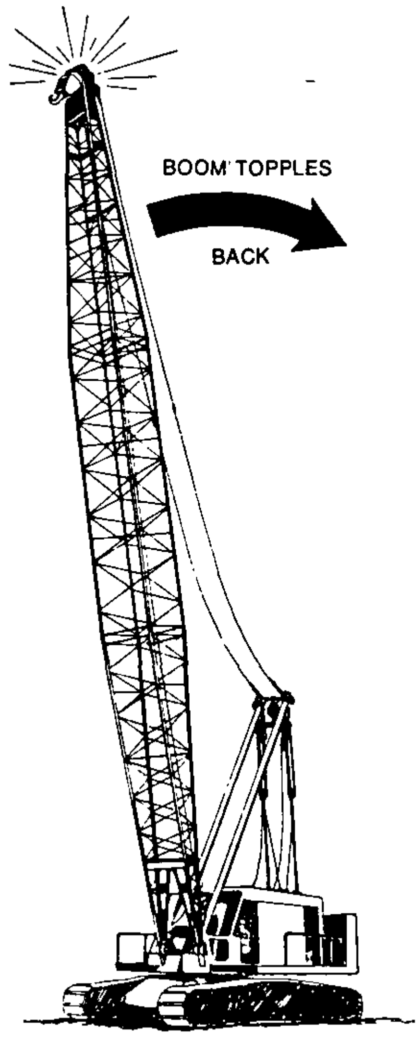


A COMMON CRANE ACCIDENT.

Fig. 1.28

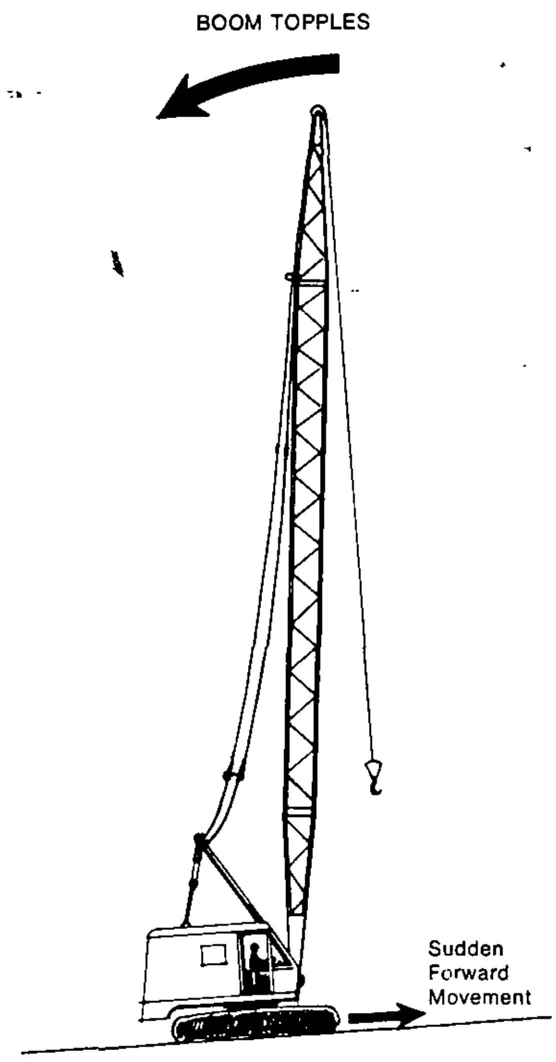
These telescopic pipes are generally installed in pairs and are commonly attached to the gantry head shaft at one end and extend to and are attached to each side of the boom at the first section joint. Some telescopic boom stops are equipped with spring or pneumatic bumpers to cushion the shock of the whip of the boom. This shock absorbing feature is highly desirable. (Fig. 1.34)

Some devices employing the principles of limit switches which disconnect the clutches or stop the engine when the boom reaches a predetermined angle are available but are not recommended for use because they do not stop the boom from being thrown backward by boom whip. They are ineffective in preventing common boom accidents. These devices are recommended for use in conjunction with the boom stopping devices however.



A COMMON CAUSE OF BOOM ACCIDENTS IS "TWO-BLOCKING" WHERE CONTINUING PULL ON THE HOIST LINE TIPS THE BOOM OVER WHEN THE LOAD REACHES THE BOOM.

Fig. 1.29



CRANE CAN "WALK" OUT FROM UNDER THE BOOM.

Fig. 1.30

Safety Features

It is strongly recommended that all mobile cranes be equipped with the following safety features and devices:

— Self-closing filler caps and flame arrestors on fuel tanks.

— A metal receptacle secured permanently to the machine for storing tools and lubricating equipment.

— Adequate lighting for night operation, including back-up lights for all mobile units.

— Wheel chocks on mobile units to block movement on slopes when the equipment is left unattended or is undergoing maintenance. (Fig. 1.35)

— Rear view mirrors on both sides of mobile equipment that are each at least 100 square inches in area.

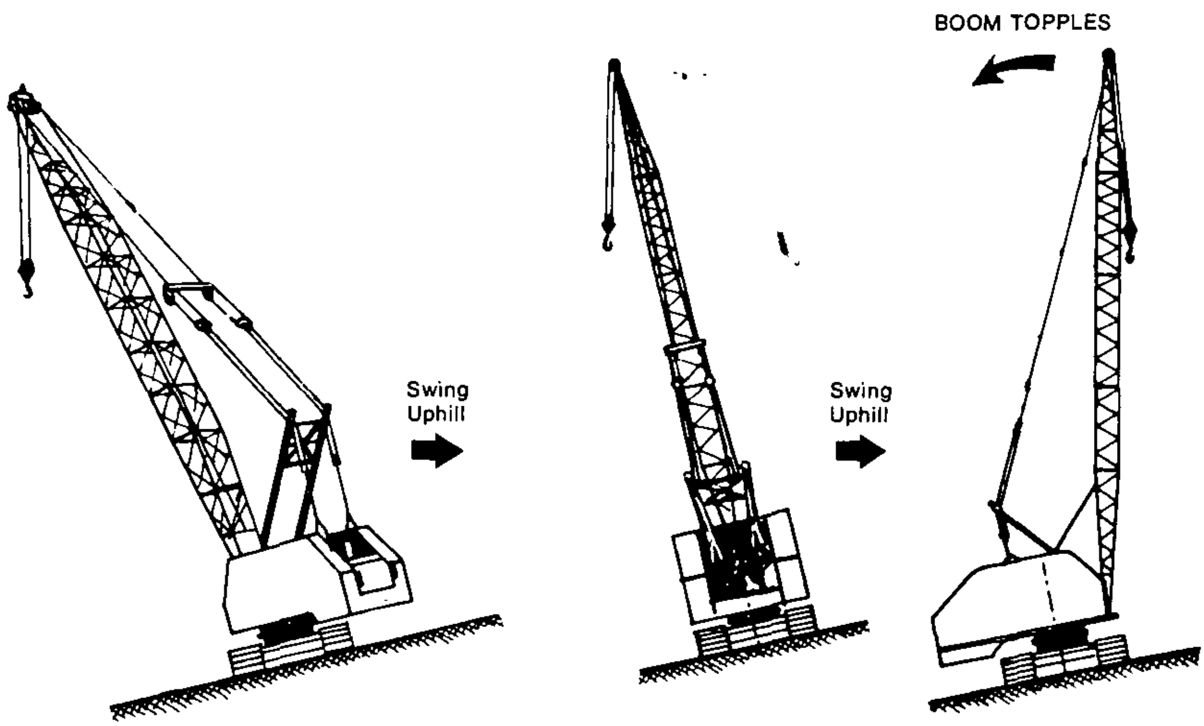
— A 5:B:C fire extinguisher. Operating and maintenance personnel should be familiar with the use and care of the fire extinguishers provided.

— Boom angle indicators on all machines having booms capable of moving in the vertical plane. The indicator must be clearly visible and readable by the operator at his control station to within 1°. (Fig. 1.36)

— Boom length indicators on all machines having telescopic booms. (Fig. 1.37)

— An effective audible warning signal mounted on the outside of the cab. The controls for the device shall be within easy reach of the operator. (Fig. 1.38)

— Shock absorbing boom stops and boom hoist safety shutoffs. (Shock absorbing boom stops are not required for telescopic booms.)



NOTE

— On 6° slope, change in boom angle to horizontal is 12 degrees (double the slope)

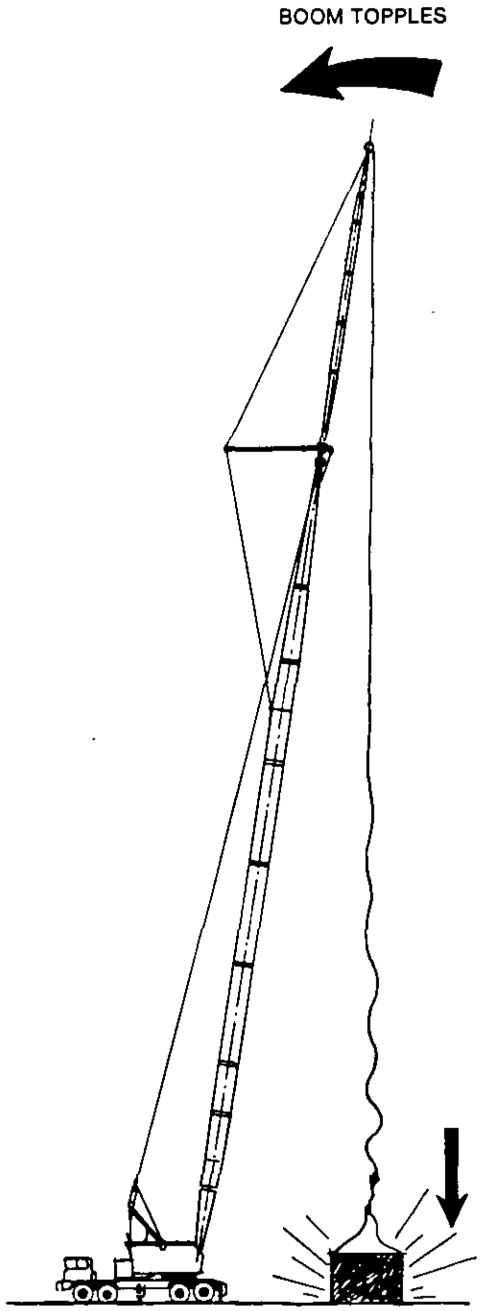
— The moment on the turntable is increased

— The load radius decreases

— The boom can topple backward.

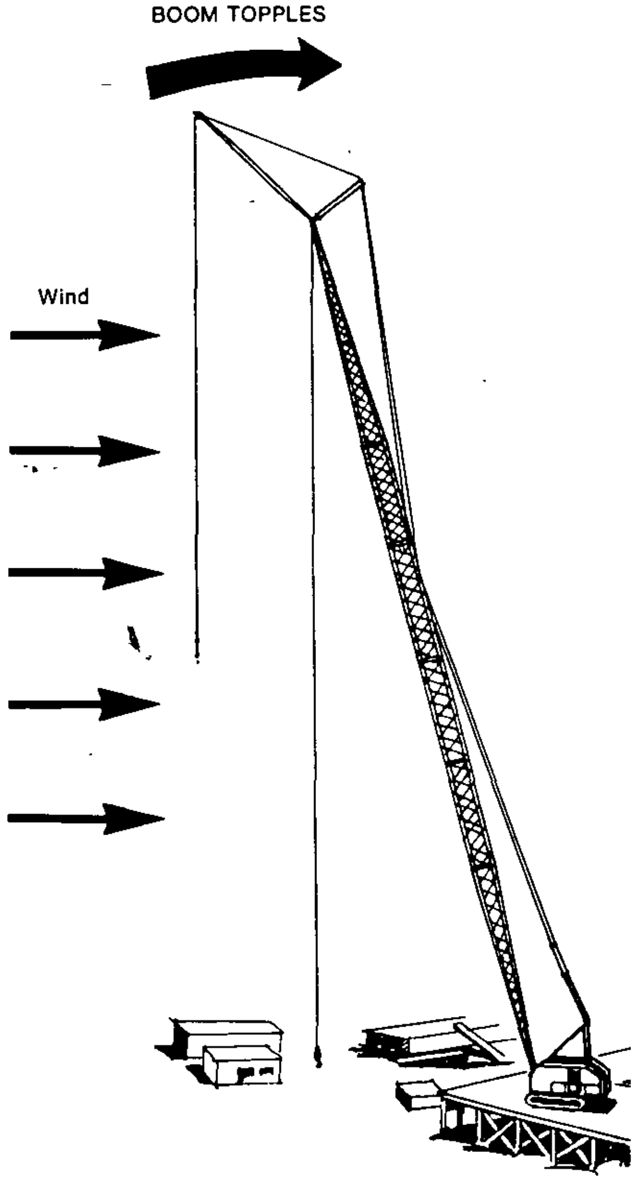
CARRYING A HIGH BOOM ON SLOPING GROUND IS DANGEROUS, FOR A SWING OF THE CAB TOWARD THE UPHILL SIDE MIGHT THROW THE BOOM OVER BACKWARD.

Fig. 1.31



IF THE LOAD DROPS QUICKLY, THE PENDANT STRETCH & BOOM DEFLECTION WILL COME OUT QUICKLY CAUSING THE BOOM TO SPRING BACK SUDDENLY.

Fig. 1.32



HIGH WINDS CAN CREATE FORCES SUFFICIENT TO TOPPLE THE BOOM.

Fig. 1.33

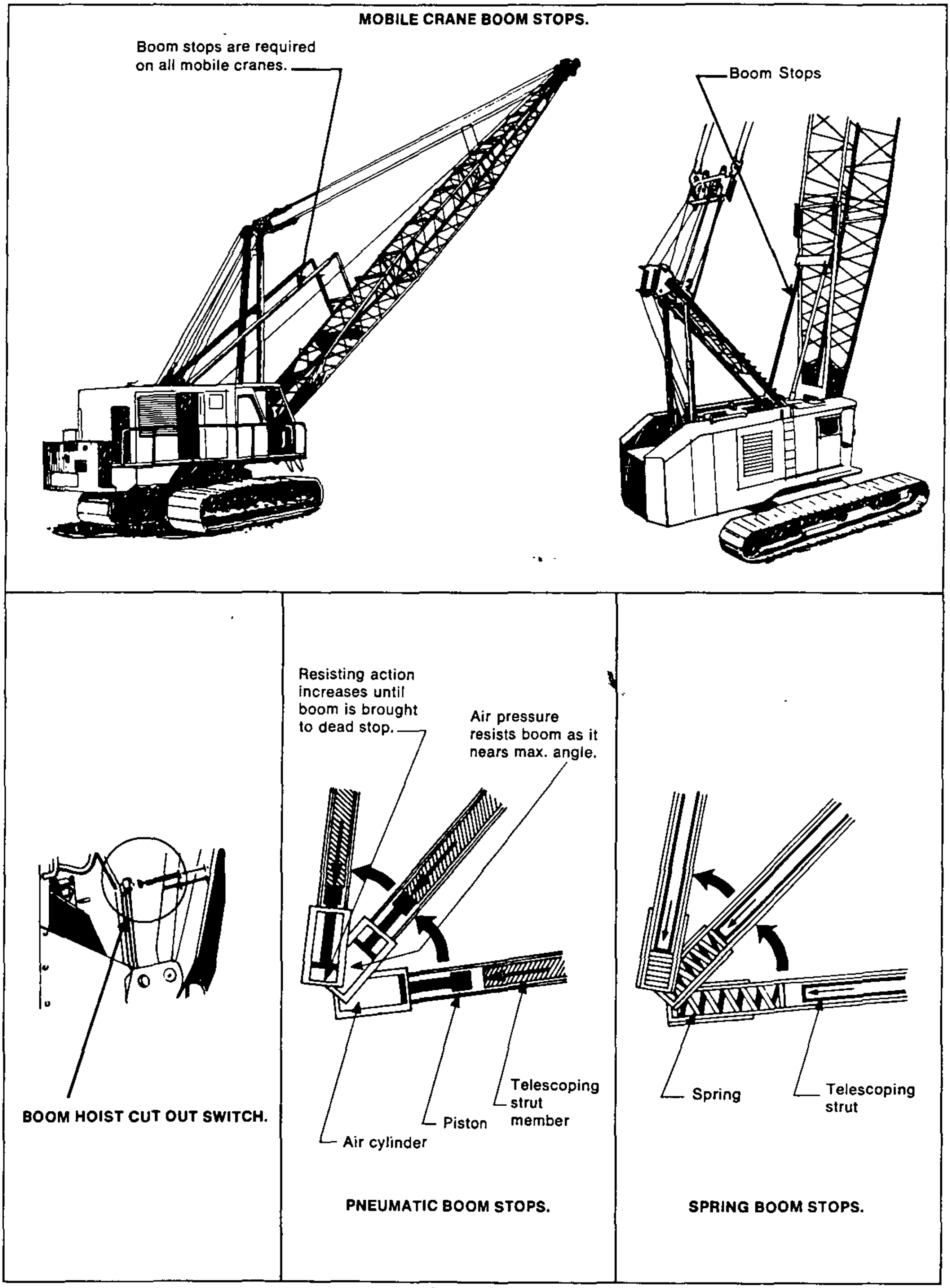

MOBILE CRANE BOOM STOPS.
Boom stops are required on all mobile cranes.
Boom Stops
BOOM HOIST CUT OUT SWITCH.
Resisting action increases until boom is brought to dead stop.
Air pressure resists boom as it nears max. angle.
Telescoping strut member
Piston
Air cylinder
PNEUMATIC BOOM STOPS.
Spring
Telescoping strut
SPRING BOOM STOPS.

Fig. 1.34

— Automatic devices to stop boom drum motion when the maximum permissible boom angle is reached.

— A spirit level at the outrigger controls for levelling the device.

— Machined surfaces on the revolving deck, parallel to the boom foot pins in the horizontal plane, on which can be placed a 4 foot carpenter's level for final precision levelling. (Fig. 1.39)

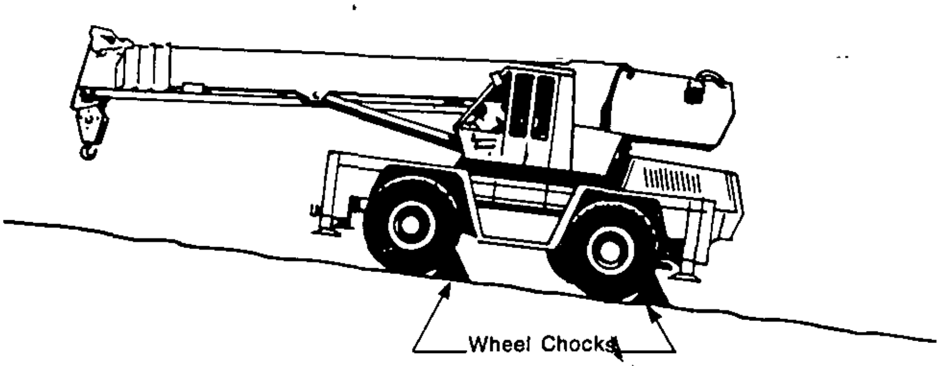
— A plate installed in the vicinity of the boom foot pins clearly indicating the distance of a well defined point from the centre of rotation. This value can be added to the distance of that point from the load center to obtain the distance of the load from the center of rotation. (Fig. 1.40)

Equipment Handbook and Records

Manufacturers' manuals containing all pertinent data relating to operation and maintenance for the specific model of crane in use must be provided with each machine. The manual should include, but not necessarily be limited to, the following information:

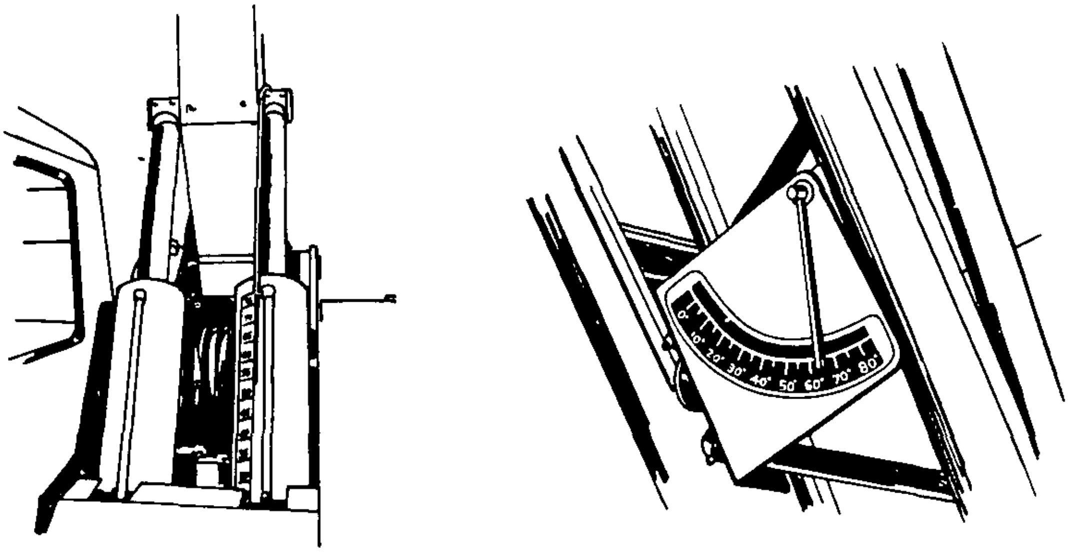
— Equipment designation or type.

— Name of equipment manufacturer.



USE WHEEL CHOCKS WHEN CRANE IS LEFT ON SLOPES.

Fig. 1.35



BOOM ANGLE INDICATORS MUST BE INSTALLED ON ALL MOBILE CRANES.

Fig. 1.36

— Name of equipment designer, if other than the manufacturer.

— Equipment model number and serial number.

— Year of original sale by the manufacturer.

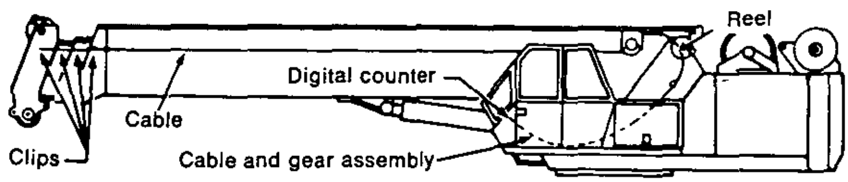
— Total weight of the unit and ground bearing pressures on tracks or tires and outriggers.

— Weight of boom sections.

— A copy of the load chart plus any and all rated combinations and variations in capacity and geometry.

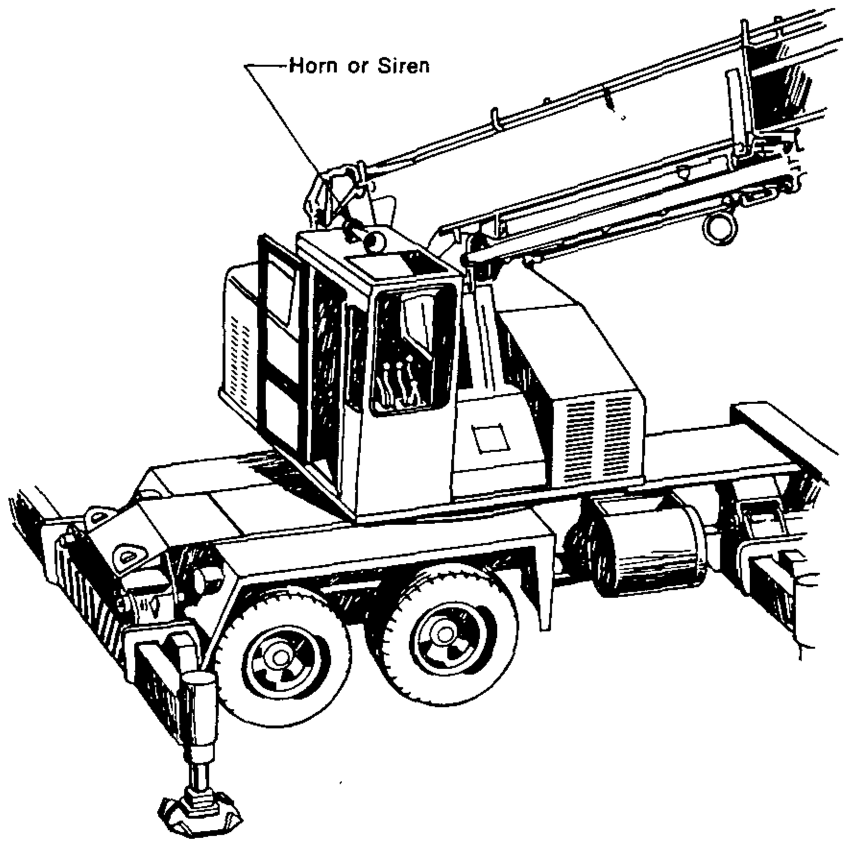
— Inspection and maintenance procedures including:

(a) Material specifications on booms and outriggers.



BOOM LENGTH INDICATORS SHOULD BE INSTALLED ON ALL HYDRAULIC CRANES.

Fig. 1.37



EVERY CRANE SHOULD HAVE A WARNING HORN OR SIREN.

Fig. 1.38

(b) Welding specifications on booms and outriggers.
(c) Bolting and torquing specifications.

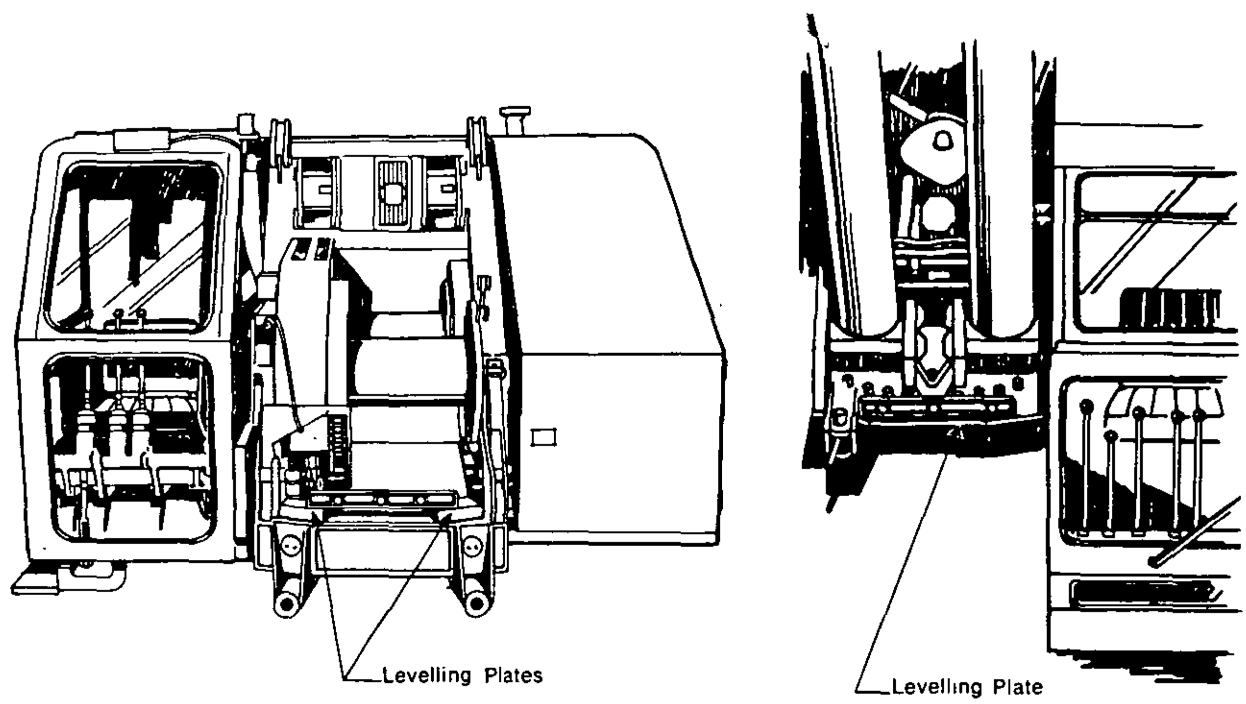
— Rigging specifications.

— Erection procedures.

— Operating precautions.

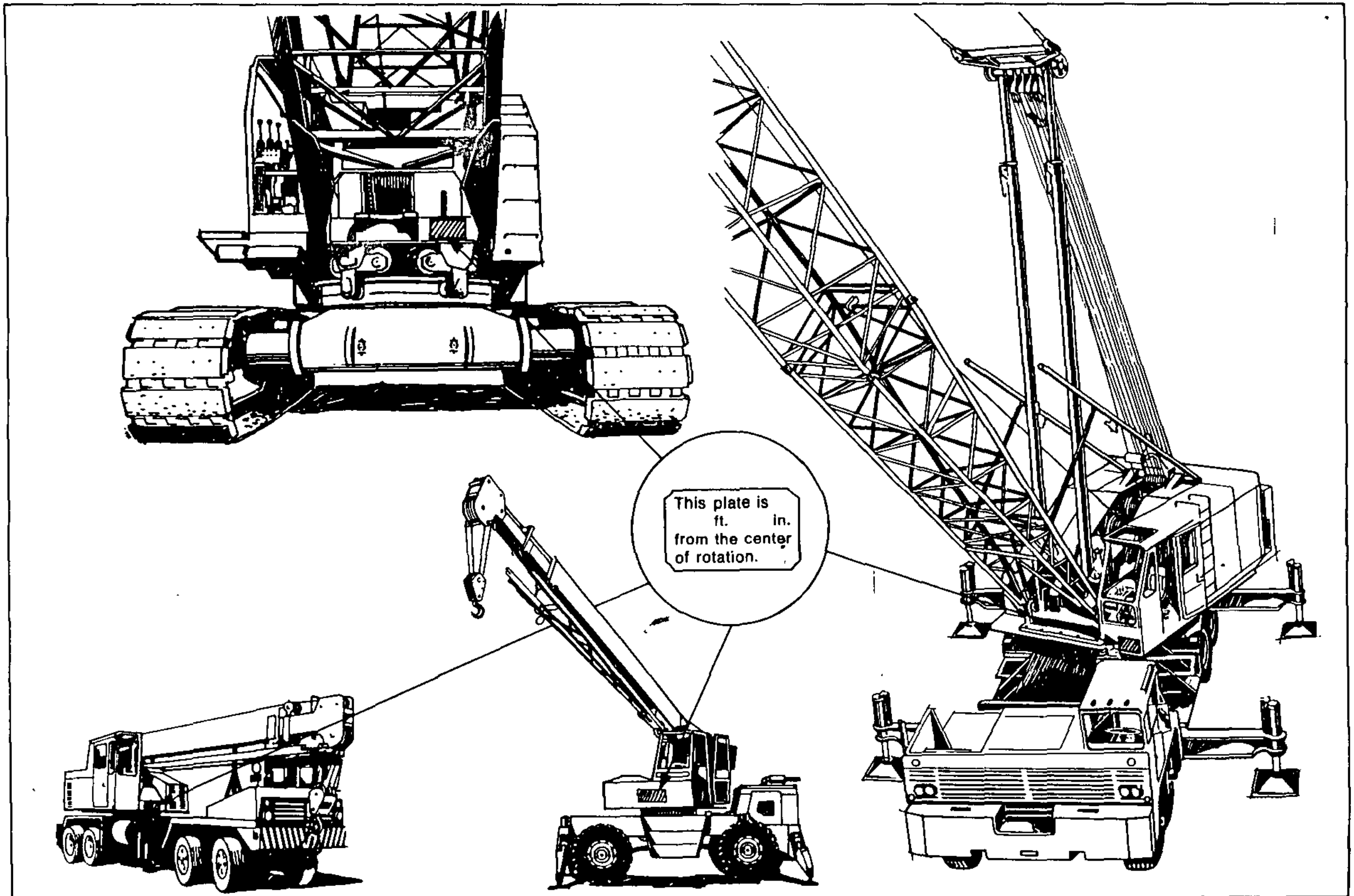
If the equipment is not supplied with a log book then one should be started, maintained and kept on the work site for the regular, periodic recording of all inspections, tests, repairs, maintenance, and hours of service related to the machine. All entries should be dated and signed by the operator, repairman and supervisor. The machine owners should ensure that the log book remains with the machine and is kept up-to-date throughout the working life of the machine.

Refuse to purchase, lease, or use any piece of equipment which has been modified, altered, or otherwise subjected to any deviation, from the original manufacturer's specifications, in any way which could affect the safety of operation unless you have documented proof that the change was engineered and certified by a competent authority. In addition, check the documentation to ensure that any piece of equipment that had been damaged in any way affecting the safety of operation was repaired by reputable persons and certified by a qualified authority.



THERE SHOULD BE A PLATE AT THE BOOM FOOT ON WHICH TO PLACE A LEVEL.

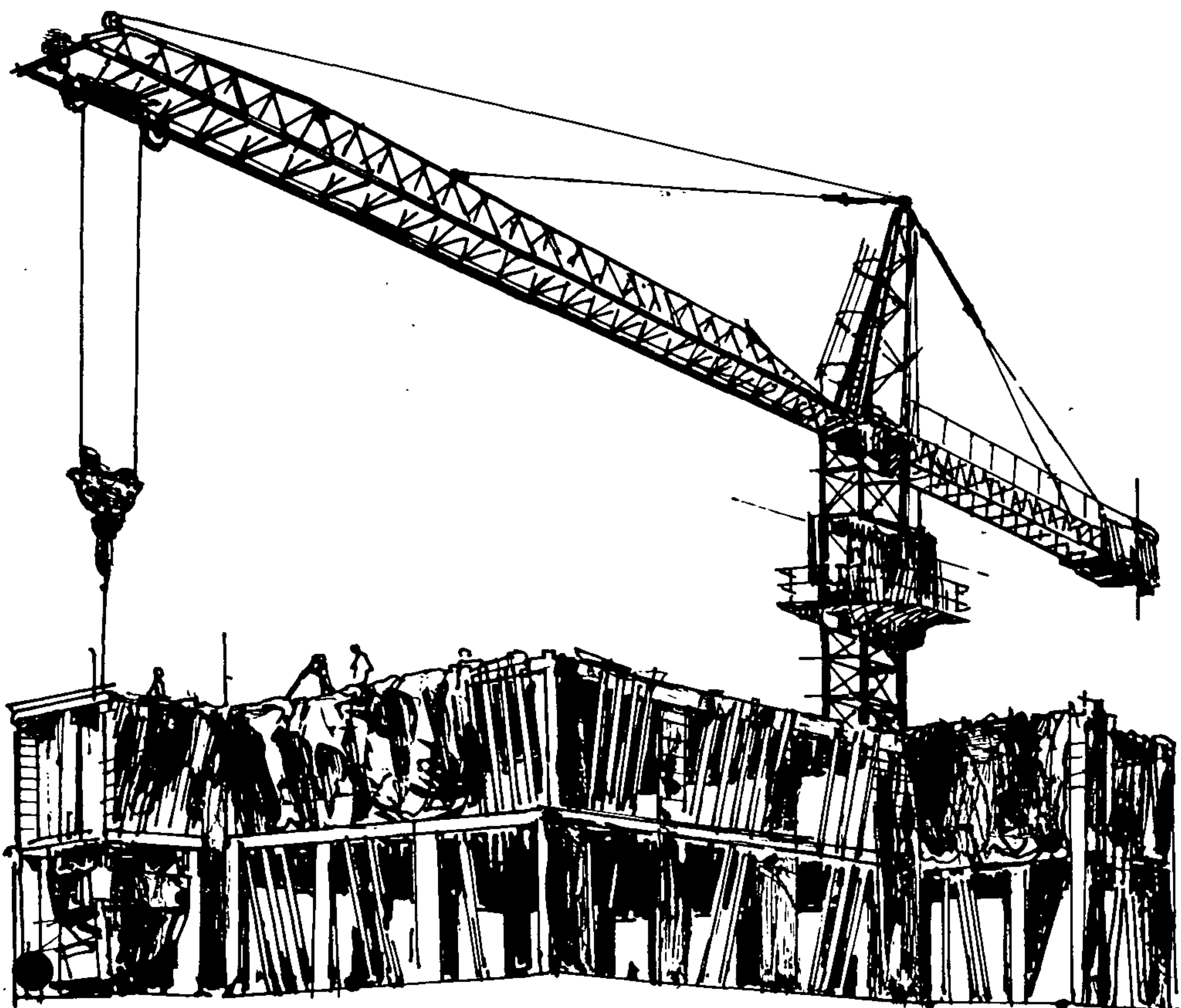
Fig. 1.39



ENSURE THAT A PLATE IS INSTALLED NEAR THE BOOM FOOT GIVING THE DISTANCE FROM THE CENTER OF ROTATION. THIS VALVE CAN BE ADDED TO THE DISTANCE OF THAT POINT FROM THE LOAD CENTER TO OBTAIN THE DISTANCE OF THE LOAD FROM THE CENTER OF ROTATION.

Fig. 1.40

PART 2
Tower Cranes



CHAPTER 5

MACHINE SELECTION AND EQUIPMENT REQUIREMENTS

MACHINE SELECTION

Like mobile cranes, tower cranes must be selected to suit the requirements of the job. If the crane's basic characteristics do not match the job's requirements then unsafe conditions will be created before any work is done. Job personnel will be forced to "make do" and improvise in a rushed atmosphere, a combination that leads to accidents.

The decision to use a tower crane rather than a mobile crane is a relatively easy one since the determining conditions are simple.

- Whenever the need for a crane is long term in a given location.
- Whenever the site is constricted or congested.
- Whenever the lift heights are extreme and the reach may be deep.
- Whenever there is little need for mobility but lift frequency is high.
- Whenever long term rentals of tower cranes are less than for mobile cranes.
- Whenever load placement in the working areas is more easily done by a crane working from within that area rather than one outside.

The use of any type of crane requires planning but tower cranes require more than usual because their structures, foundations, and presence on the site are generally permanent for as long as the heavy construction phases are ongoing.

In selecting the most suitable type, size and number of tower cranes for a particular application, the characteristics of the various machines available must be considered against the requirements imposed by the loads to be handled and the surroundings in which the crane will operate.

In addition to considering such factors as the weights, dimensions and lift radii of the heaviest and largest loads, those selecting the crane for a specific project must also consider the characteristics of the available cranes against:

- The type and size of the base for the crane.
- The maximum free standing height of the crane. (Fig. 5.1)
- The maximum braced height. (Fig. 5.2)
- The climbing arrangement.
- The weight of the crane that will have to be supported by the building.
- The jib lengths available.
- The possible necessity of auxiliary guys.
- The available head room between the maximum height position of the hook and the upper most work level. (Fig. 5.3)
- The area that has to be covered.
- The hoist and lower speeds.
- The length of cable the hoist drum carries. (Fig. 5.4)
- The number of parts of line the machine needs to do the required hoisting.
- The service availability and cost.
- The rental charges.

— The cost of operations such as erection, dismantling, off site transport and charges for climbing the machine.

— The capacity of the crane must be such that there is a least a 5% working margin on every lift.

Both static and mobile tower cranes are available in a wide variety of types and configurations according to the particular combination of tower, jib and type of base which they employ.

Tower Configurations

Tower cranes are available with either fixed (Fig. 5.5) or slewing (Fig. 5.6) towers. On the fixed tower type the slewing ring is situated at or near the top of the tower and the jib slews about the vertical axis of the stationary tower. The slewing ring on the slewing tower type is situated at the bottom of the tower and the whole of the tower and jib assembly slews relative to the base of the crane.

In addition to being either fixed or slewing, the towers can be further classified as being mono towers, inner and outer towers and telescopic towers. On the mono tower the jib is carried by a single tower structure which may be either fixed or slewing (Fig. 5.7). The inner and outer tower types are characterized by the jib being carried by a slewing inner tower which is supported at the top of the fixed outer tower (Fig. 5.8). The telescopic tower structure consists of 2 or more main sections which nest into each other to enable the height of the crane to be altered without the need for partial dismantling and re-erection (Fig. 5.9). Telescopic towers are usually of the slewing type and are more common on rail mounted and mobile tower cranes.

Jib Configurations

The main types of jibs used on tower cranes are saddle jibs, luffing jibs, fixed-luff jibs and rear pivoted luffing jibs.

Saddle jibs are supported by pendants in a horizontal or near horizontal position and the load hook is suspended from a trolley which moves along the jib to alter the hook radius (Fig. 5.10). Luffing jibs are pivoted at the jib foot and are supported by luffing cables much like the boom on mobile cranes (Fig. 5.11). The hoist rope which supports the load usually passes over a sheave at the jib head, and the hook radius is altered by changing the angle of inclination of the jib.

Fixed-luff jibs are also mounted on pivots at the jib foot but unlike the luffing jibs these are held by jib pendants at a fixed angle of inclination (Fig. 5.12). On some types the hook is suspended from the jib head and the hook radius cannot be altered while on others the hook is

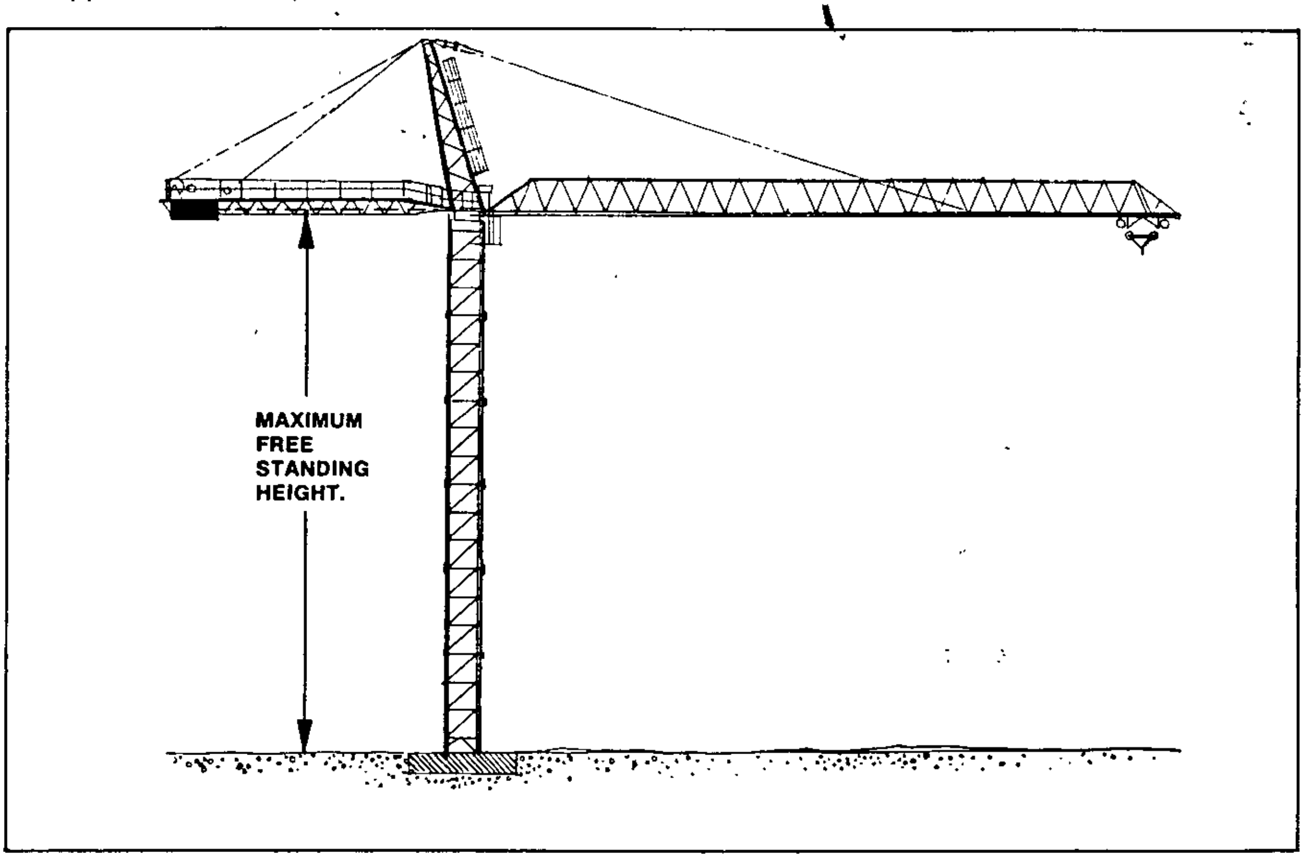


Fig. 5.1

suspended from a trolley which travels on the jib. On rear pivoted luffing jibs the jib pivot is situated towards the rear of the top of the tower and the hook is supported by the hoist rope which passes over a sheave at the jib head. (Fig. 5.13)

The saddle jib usually has a smaller minimum operating radius than the equivalent luffing jib and is thus able to handle loads closer to the tower of the crane. For a given height of tower, however, a greater height of lift is available with a luffing jib, and the jib can be raised or lowered to clear obstacles. One advantage of a fixed luff jib is that its extra height at the jib head enables it to clear objects that would obstruct a saddle jib. The advantage of a rear-pivoted luffing jib is that it has a smaller minimum hook radius than an ordinary luffing jib.

Mounting Configurations

In addition to being classified according to tower and jib configurations, tower cranes are also characterized according to their mounting configuration. They are available as rail mounted units, stationary units, climbing units and mobile units (either truck or crawler mounted).

The rail mounted units can be equipped with fixed or slewing towers and any of the jib configurations (Fig. 5.14). Because of their mobility they generally have a larger area of coverage than do climbing and stationary tower cranes. They are also better adapted to travelling with load than are mobile (truck or crawler mounted) tower cranes. Their primary advantage is maximum coverage with minimal site space. Disadvantages of the units include the expense of

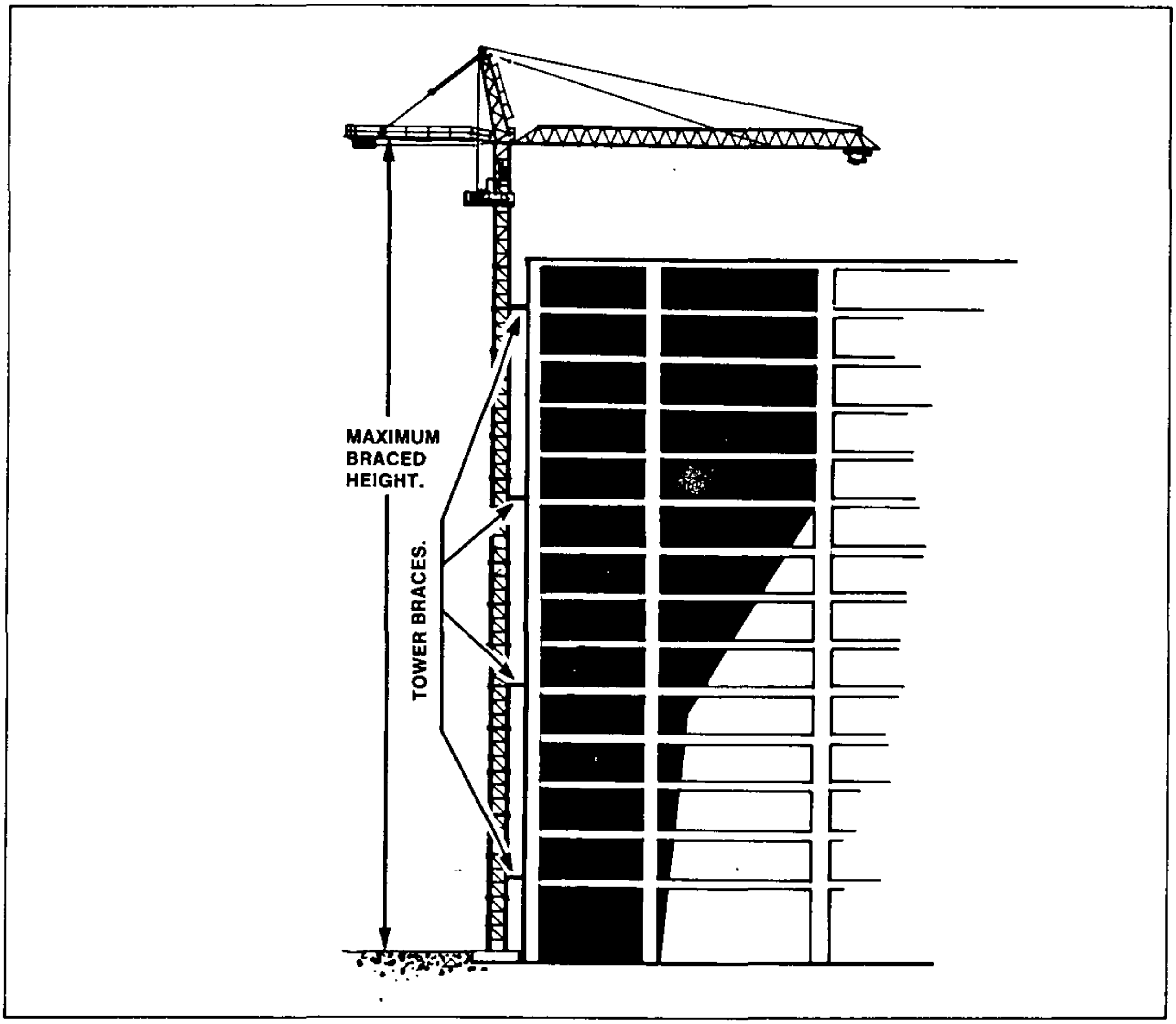


Fig. 5.2

installing the track and the inconvenience of disrupting the crane service whenever work must be done on or near the track.

The stationary tower configuration can be used to good advantage because they occupy such a limited area (Fig. 5.15). They can be set at varying heights up to their maximum free standing height and can be extended beyond this limit by tying the crane back to the structure. A tower crane on a static base must be able to cover from its fixed position all points at which the loads are to be handled. The crane's capacity decreases as the operating radius increases, therefore, caution must be exercised in selecting the crane to ensure that the maximum lift weights can be handled at the desired radius. If the crane is used at a height which is to be later extended, it is advantageous if the tower has the facility for adding its own extra sections without dismantling any of the jib/counterjib assembly. (Fig. 5.16)

The climbing tower crane is generally used to good advantage when the building structure is high, the side area is limited and the structure itself is capable of supporting the crane (Fig. 5.17). This configuration of crane is supported by the structure which it is being used to construct, and to which it is attached by support frames and wedges. The height of the crane can be extended as the height of the structure increases by means of climbing ladders attached to the frames. The cranes are usually initially mounted on fixed bases and are later transferred to climbing frames and ladders.

The mobile mounting configuration consists of either crawler or truck mounted units much like the mobile crane class. The crawler units are equipped with special booms that are set vertically and carry either horizontal or luffing jibs (Fig. 5.18). Their advantage lies in great inward reach without the long boom which would normally be required to reach over the top of the structure if a mobile crane were to be used.

The crawler mounted tower crane must be set firm and level when handling its rated loads. They are able to travel over firm and level ground in their erected state, but have limited ability to handle loads while doing so. They are also able to travel in a partially erected state over unprepared ground providing it is within certain limits of level and compaction.

The truck mounted mobile tower crane must also have its outriggers extended and be set up secure and level on its jacks when handling loads (Fig. 5.19). The majority of these machines have slewing towers and folding luffing jibs which facilitate transportation and erection. Some units are completely self-contained and self-powered and can be driven on the public highways. These machines are usually capable of comparatively rapid erection and dismantling but are generally unable to travel in their fully erected state and cannot handle loads while travelling.

The selection of a crane or cranes for any job should be made only after a thorough examination of all the factors involved. When renting a

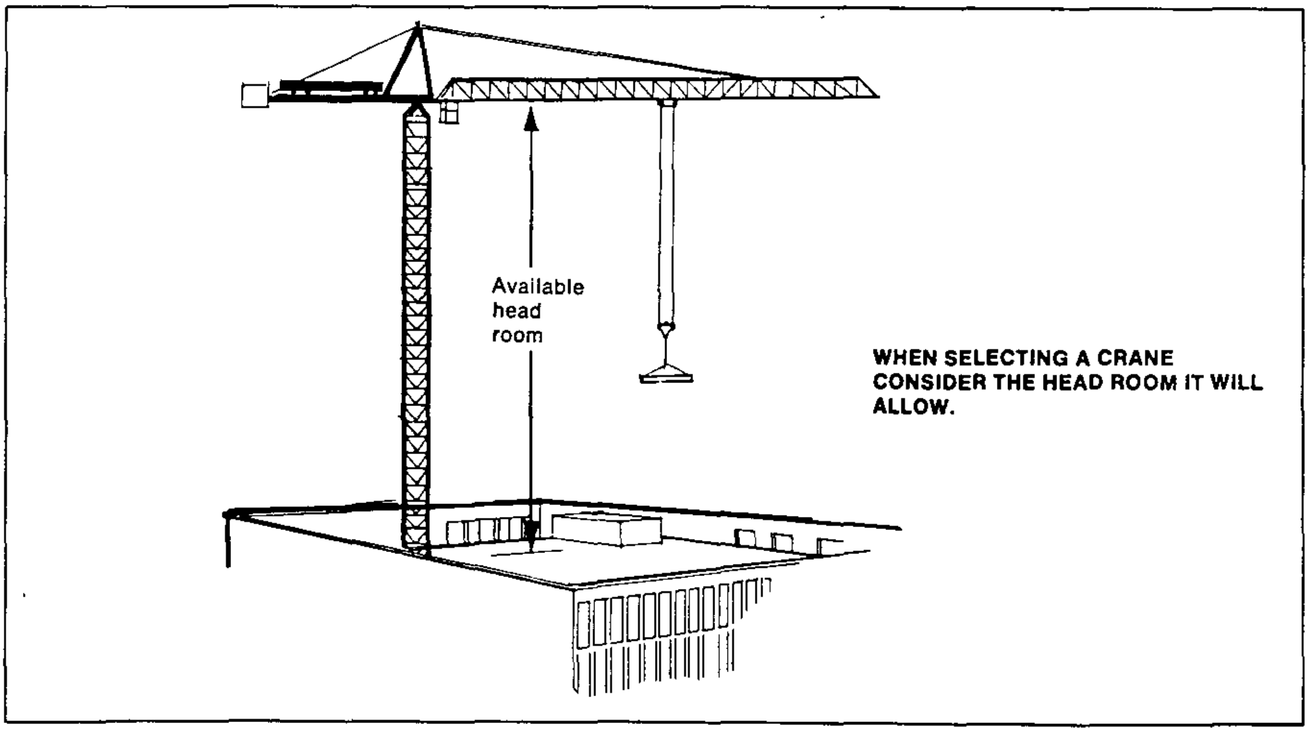


Fig. 5.3

crane be certain to let the rental agent know your requirements as their selection must be based on the data provided.

When making equipment selections, those responsible must **ensure that the unit is going to be safe and reliable for as long as it will be used** and under all anticipated conditions to which it will be exposed and operated. Nothing can take the place of experience in making these decisions, however, the guidelines set out in this section are intended to simplify the process by stressing those critical considerations that must not be overlooked.

The responsibility of equipment selection involves getting units that will not only get the job done as quickly and economically as possible, but also units that eliminate all possibility of hazard to personnel on the site, the public and the property.

Machines should be rented only from reputable firms or contractors and every effort must be made to ensure that they are in good working condition.

EQUIPMENT REQUIREMENTS

One of the prime requirements of any crane safety program is ensuring that all necessary equipment is on the machine and that it is in good working order. If the machine has been designed, manufactured, inspected, tested and maintained in accordance with Canadian Standards Association Code Z248—Tower Cranes then that is adequate assurance that all these recommendations and all Provincial and Federal Safety Regulations are met.

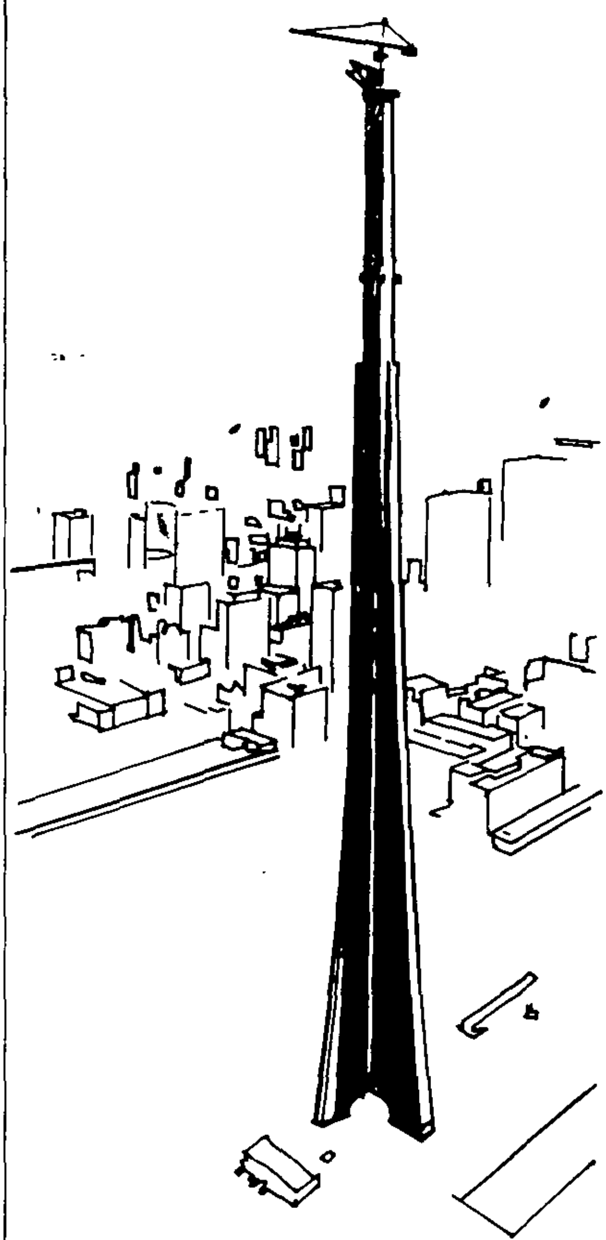
If any of this equipment or information is missing then it is the equipment owner's responsibility to see that it is made available and put on the crane and its accessories.

Identification

Every major structural, electrical and mechanical component of a tower crane should have a permanent durable plate bearing the manufacturer's name, machine model number, serial number, year of original sale by the manufacturer and weight of the unit. (Fig. 5.20)

In addition, identification numbers should be clearly marked on all basic removable components and attachments of the machine (such as counterweights etc.) to show that they belong with that machine. It is extremely important that these components be used only on that machine or identical models or on equipment for which they were specifically intended by the manufacturer.

Any components or structural sections designed and manufactured or altered by anyone other than the original equipment manufacturer or his agent must have the certificate of a qualified Professional Engineer attesting to their structural integrity to accommodate all the loads which the structure or components of the original equipment manufacturer can sustain and must be permanently identified in the same manner as the structural sections from the original equipment manufacturer.



WHEN SELECTING A CRANE, CONSIDERATION MUST BE GIVEN TO THE ROPE CAPACITY.

Fig. 5.4

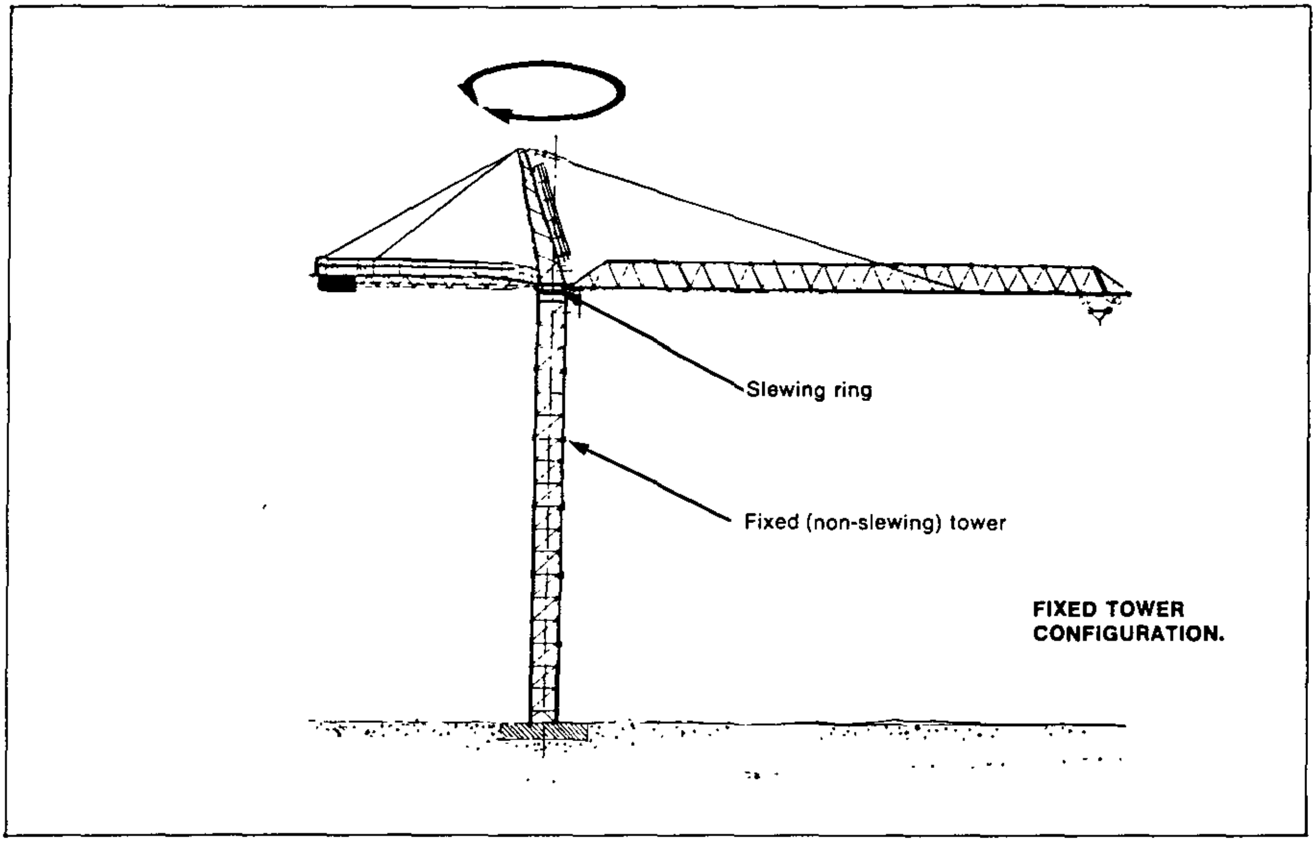

Slewing ring
Fixed (non-slewing) tower
FIXED TOWER
CONFIGURATION.

Fig. 5.5

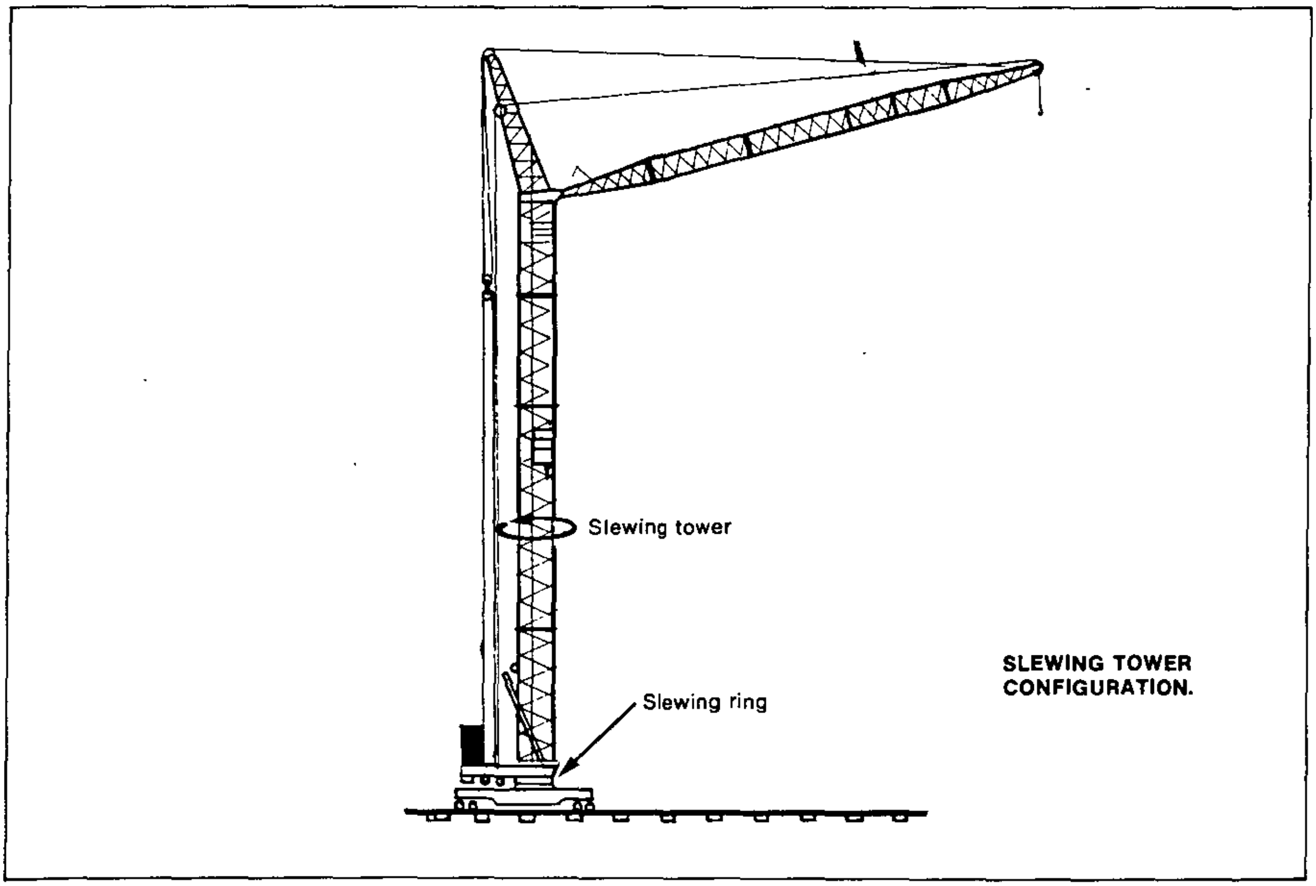

Slewing tower
Slewing ring
SLEWING TOWER
CONFIGURATION.

Fig. 5.6

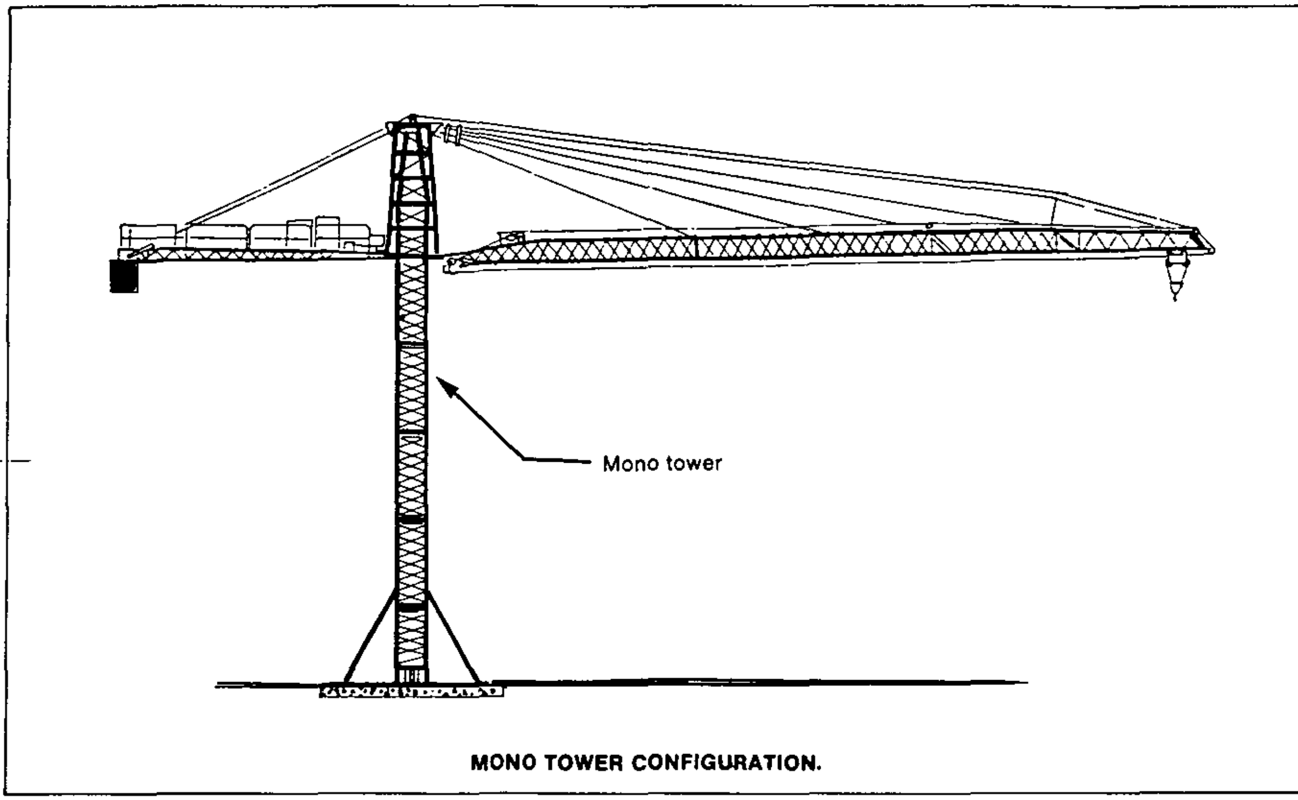

Mono tower
MONO TOWER CONFIGURATION.

Fig. 5.7

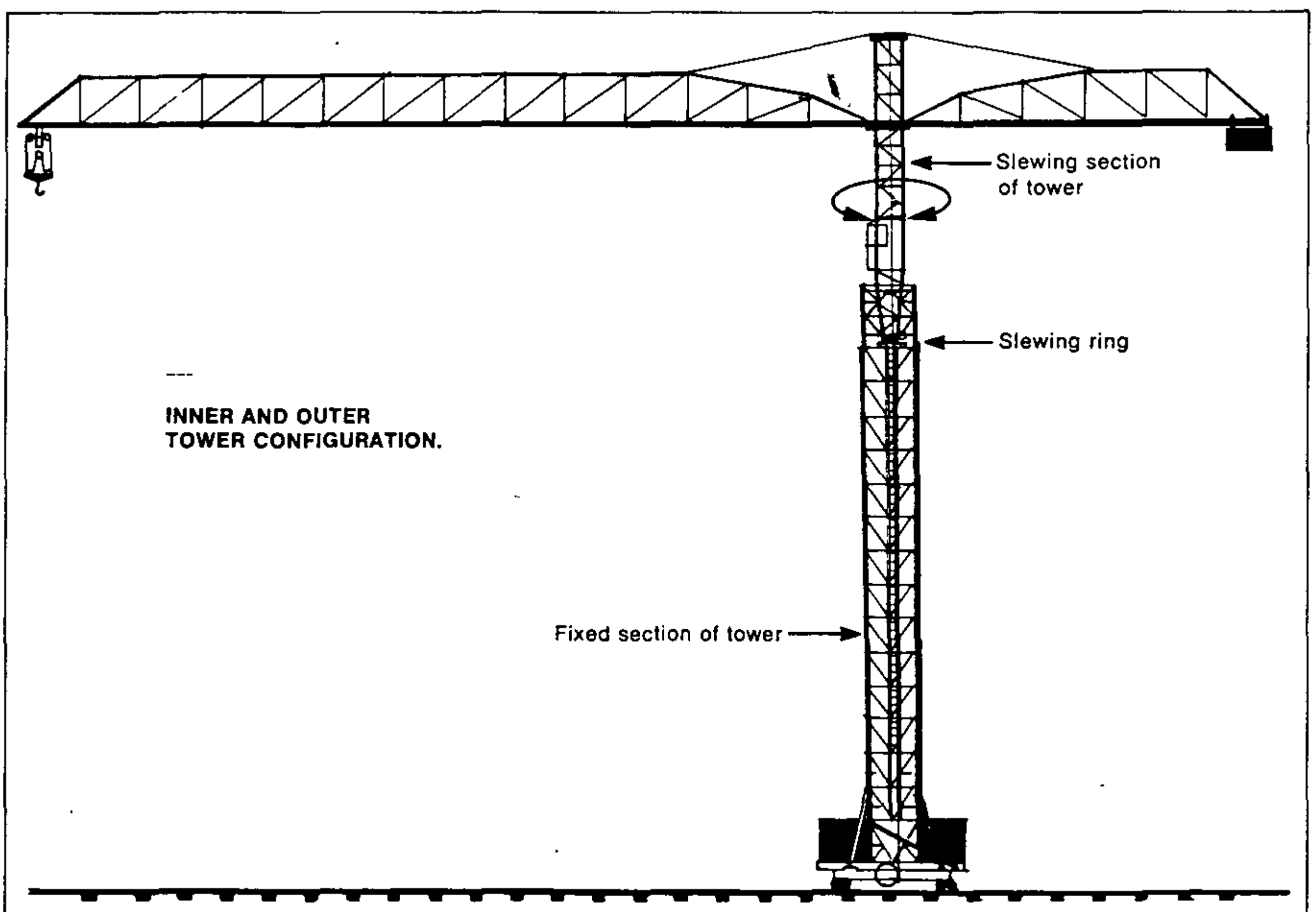

Slewing section
of tower
Slewing ring
INNER AND OUTER
TOWER CONFIGURATION.
Fixed section of tower

Fig. 5.8

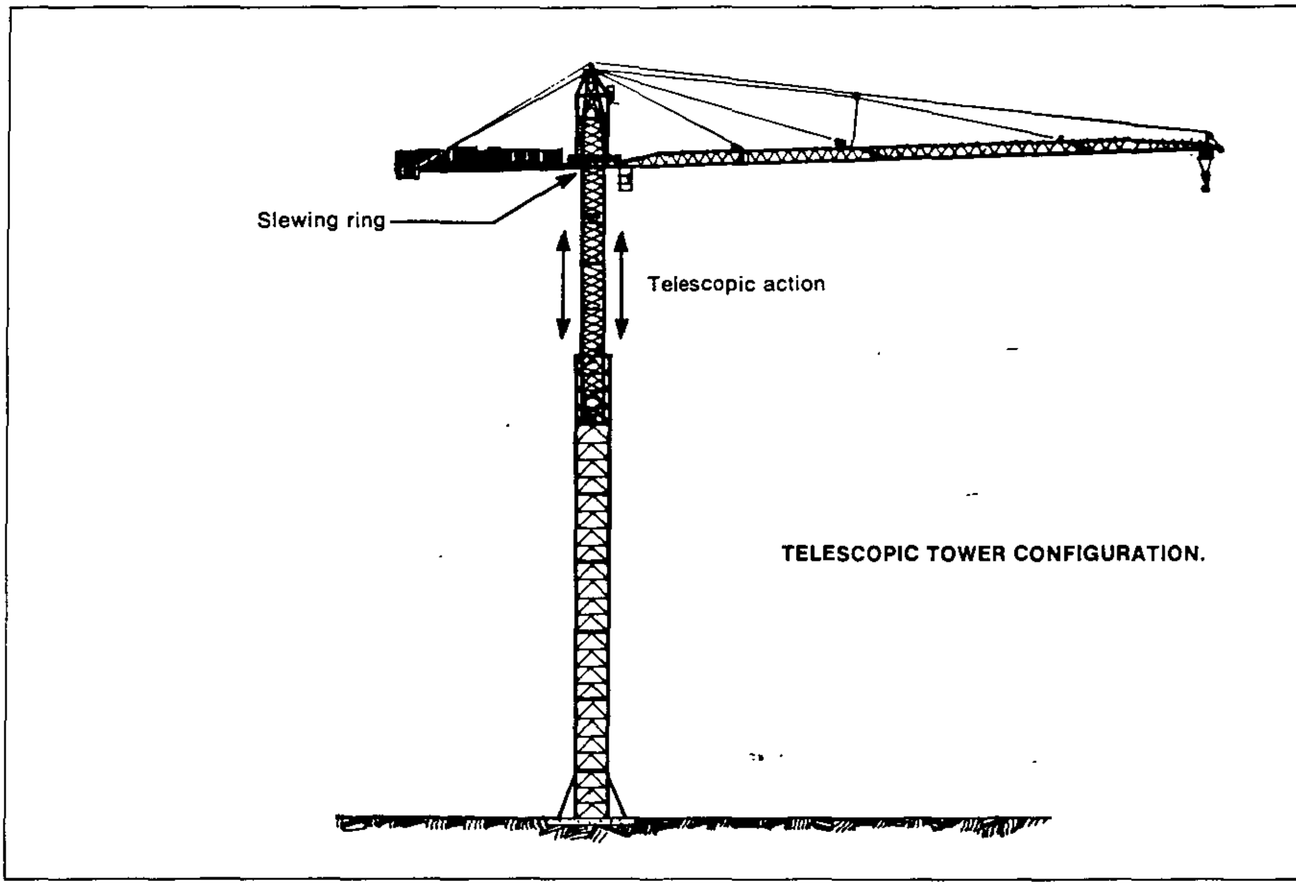


Fig. 5.9

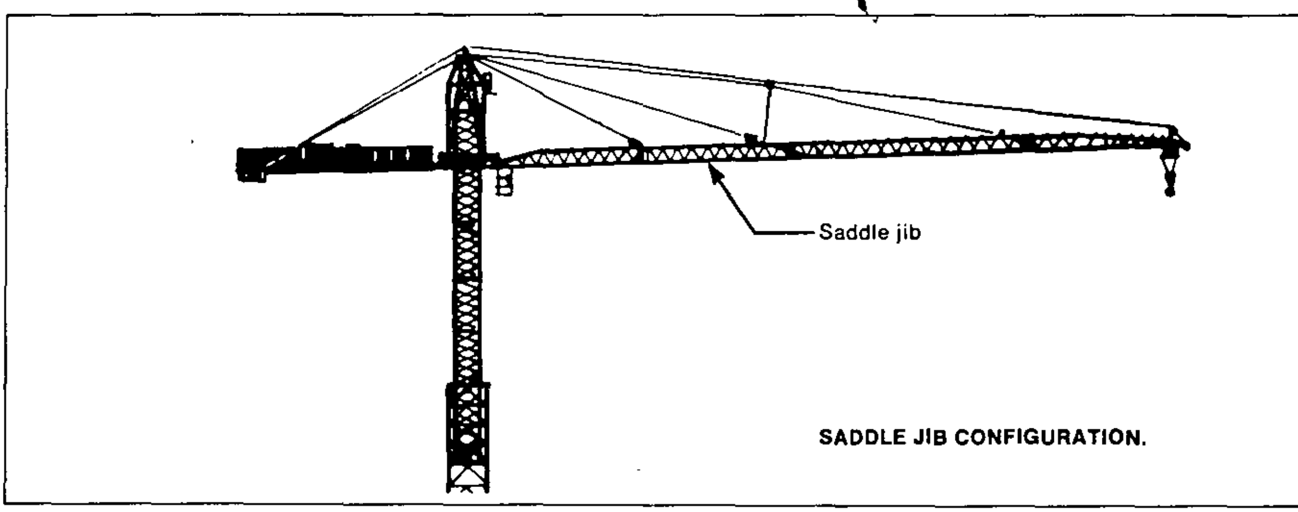


Fig. 5.10

Load Rating Information

Every tower crane **must** be equipped with a substantial and durable load chart with clearly legible letters and figures. It must be securely attached to the cab in a location easily visible to the operator while seated at his control station.

When the crane is operated from the remote control console the load chart must be attached to a substantial plate that is secured to the console. (Fig. 5.21)

The following information must be available to the operator on the load chart so that he can quickly and accurately determine the crane's capacity:

— Crane model number, serial number and date of manufacture.

— A full and complete range of the manufacturer's approved crane load ratings at all stated operating radii (or boom angles) for each recommended counterweight, boom length, tower height or other installation condition.

— Gear change instructions.

— Alternate load ratings when using permissible, optional and variable geometry equipment on the crane, such as guy wires, bracing, additional ballast, movable counterweights, etc. which affect the ratings.

— Work areas, for which capacities are listed in the chart. The manufacturer may, at his option, list capacities for one or more of these working areas or may list capacities for any combination of working areas so long as such areas or combinations of areas are identified on the capacity chart.

— Adequate warning must be indicated that no allowance is made for such factors as effects of swinging loads, tackle weight, wind and operating speeds.

— Recommended parts of hoist reeving, size, and type of rope for various crane loads.

— Essential precautionary or warning notes relative to limitations on equipment and operating procedures.

ʾrum data, available line pull, permissible ne pull, line speeds and rope spooling capacity.

— Wind velocity operating limits.

— Low temperature operating limits.

— If special materials such as high tensile steel or aluminum alloys have been used in the structure, the load chart must bear notice to this effect.

Crane Cabin (Fig. 5.22)

Every operating cab that is intended to be attached to the structure of the crane or placed at a remote location should meet the following requirements. When the machine is operated from the remote console or from a temporary work level (Fig. 5.23) attached to the tower and the operator is not enclosed by a cab, these recommendations do not apply.

The cabin should:

— Be constructed of fire proof materials.

— Be designed and constructed to protect the operator and the controls from the weather.

— Be of sufficient size to allow operation without inconvenience particularly with regard to ›of height and floor space. (Fig. 5.24)

— Be provided with a roof of adequate strength to protect the operator from falling objects.

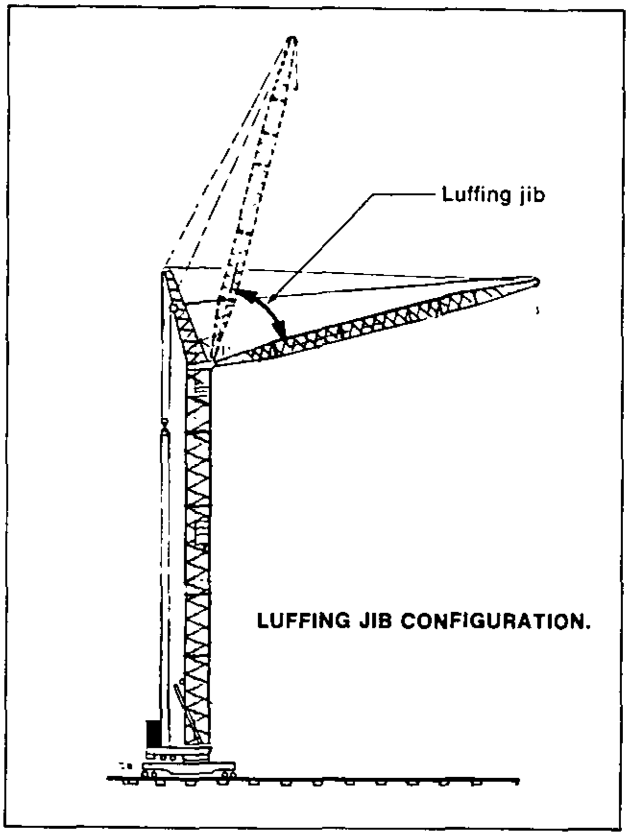


Fig. 5.11

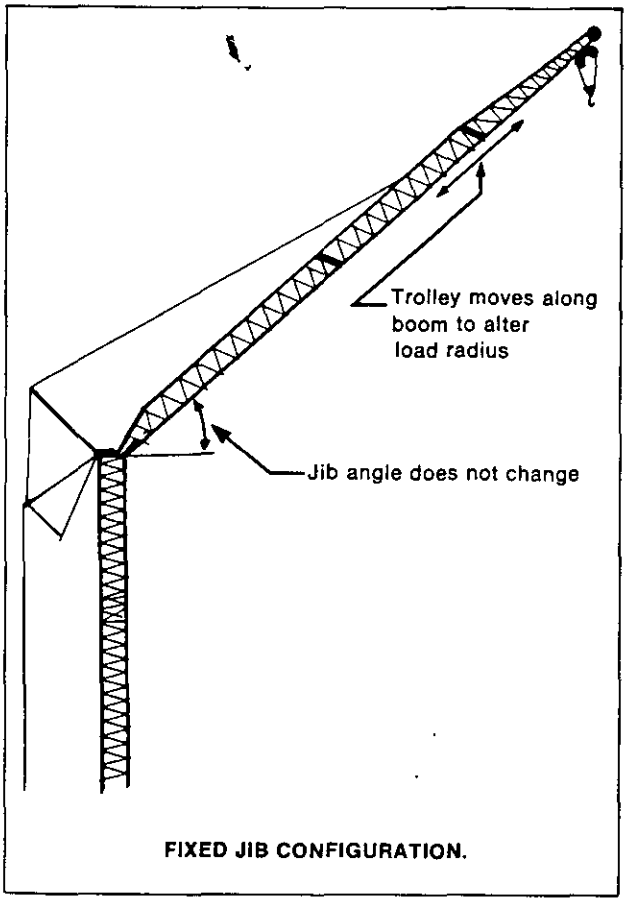


Fig. 5.12

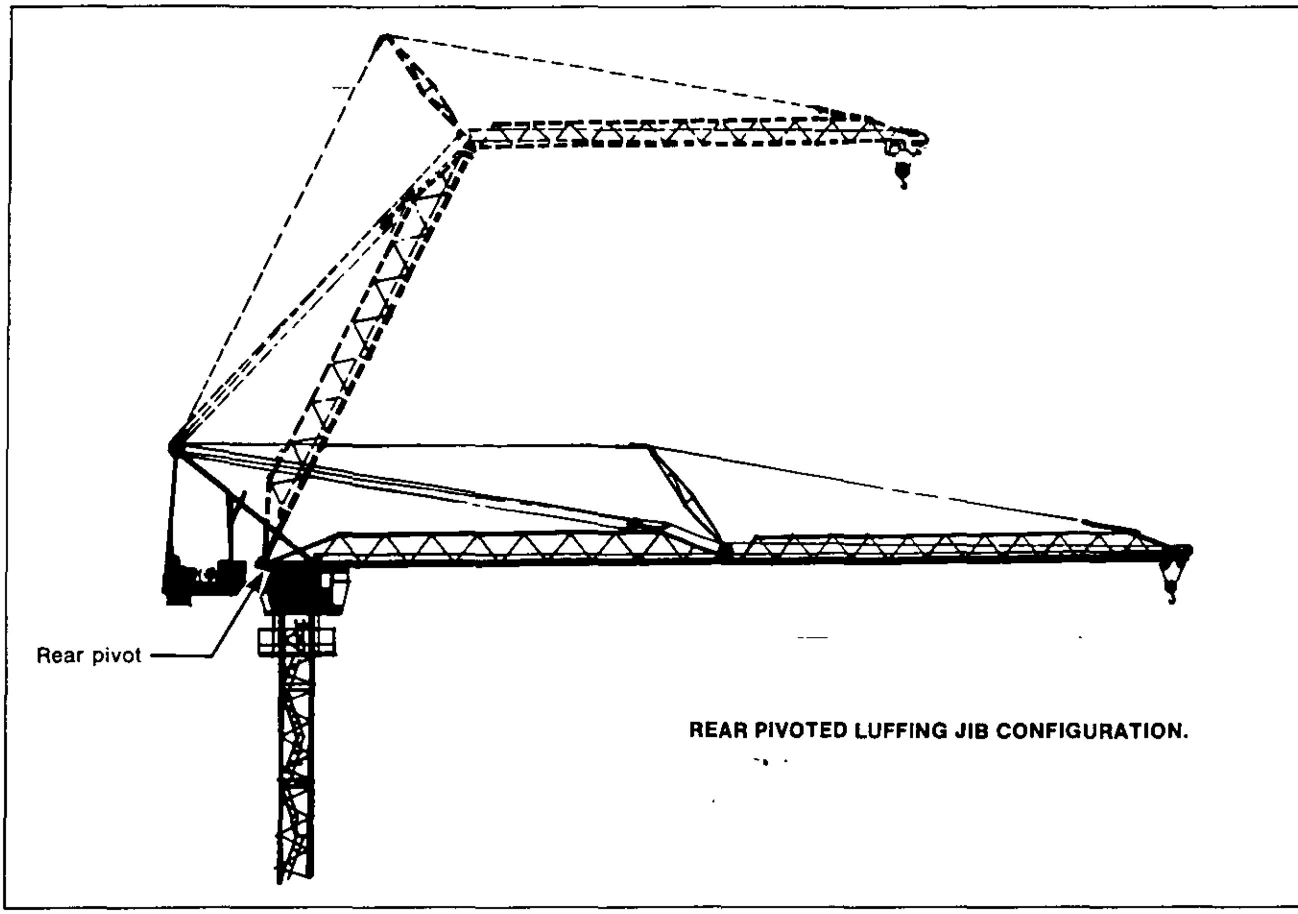
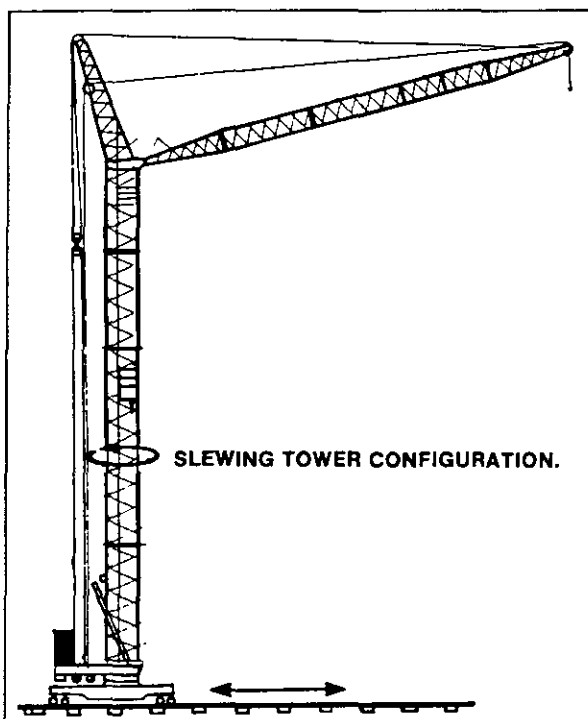
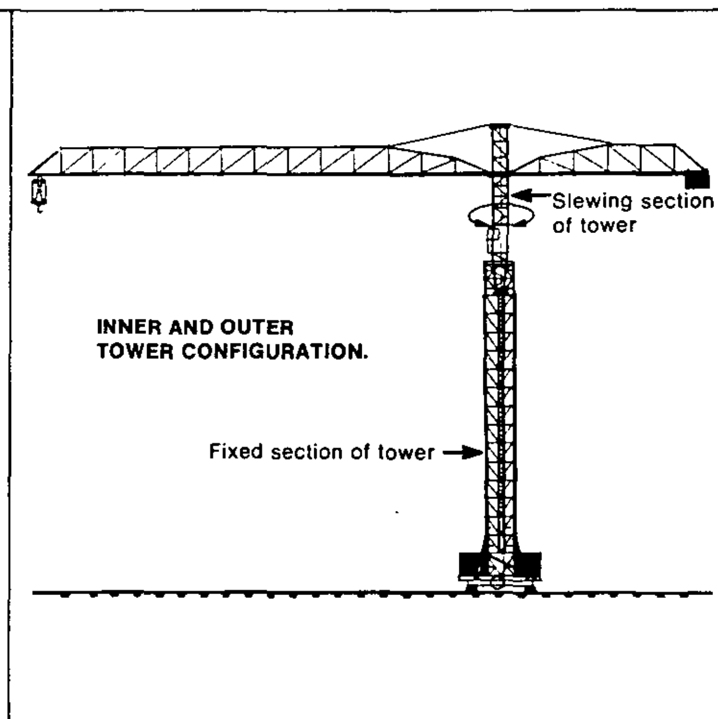


Fig. 5.13





RAIL MOUNT CONFIGURATIONS.

Fig. 5.14

— Be securely attached to whatever structure it is located on.

— Be designed and constructed to provide the operator a clear and unrestricted view of the load and boom point in all normal working positions, visibility to either side and as clear a view of the job site as possible. (Fig. 5.25)

— Have windows constructed of safety glass or equivalent and designed to provide ventilation as needed. The front window should have a section which can be removed or held open if desired. If the section is of the type held in the open position it must be well secured to prevent accidental closure. The frames of the windows should be designed so that the panes can be cleaned without danger to personnel.

— Be fitted with a lock to prevent unauthorized entry when the unit is left unattended, unless the control unit can be separately locked. The cab doors must be restrained from opening and closing accidentally whenever the machine is in use.

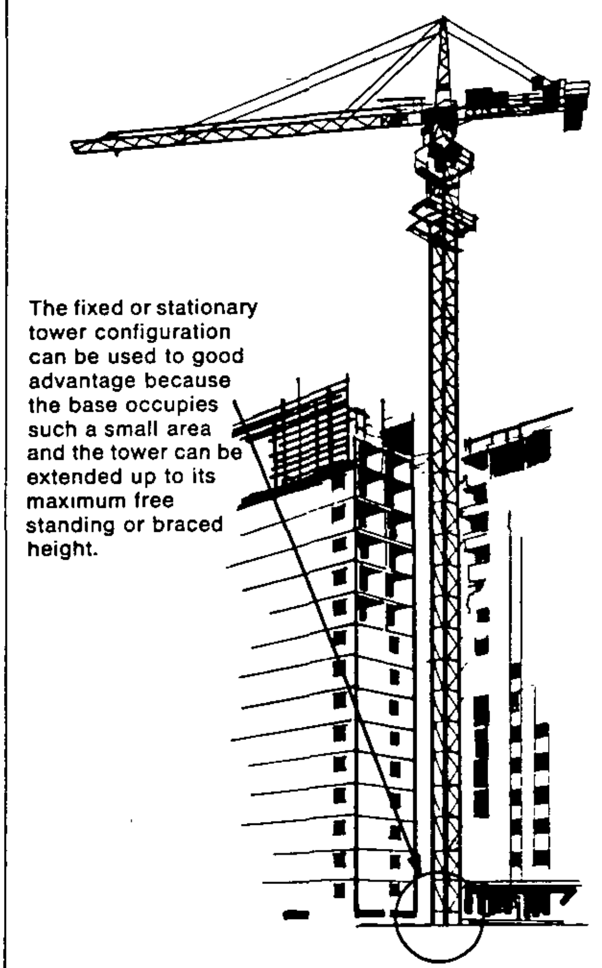
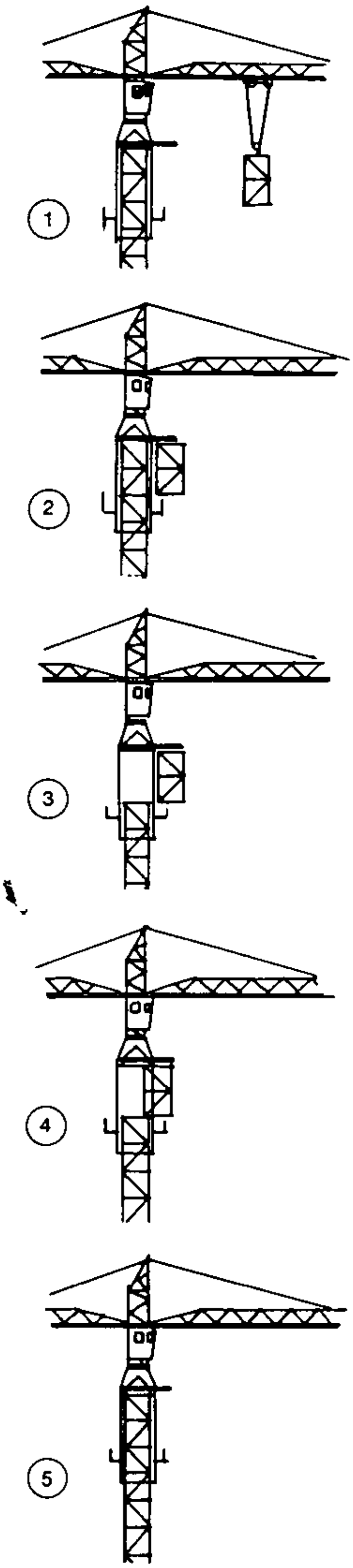
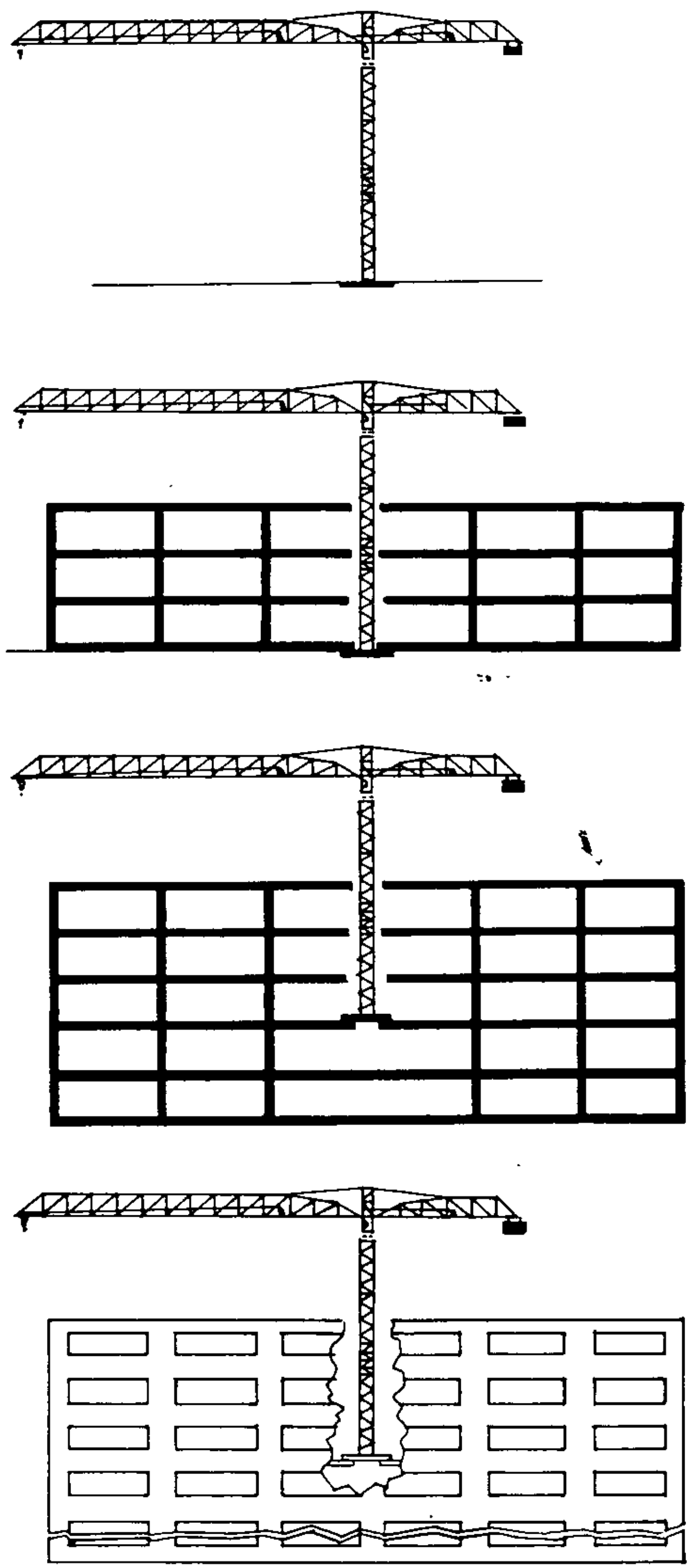


Fig. 5.15



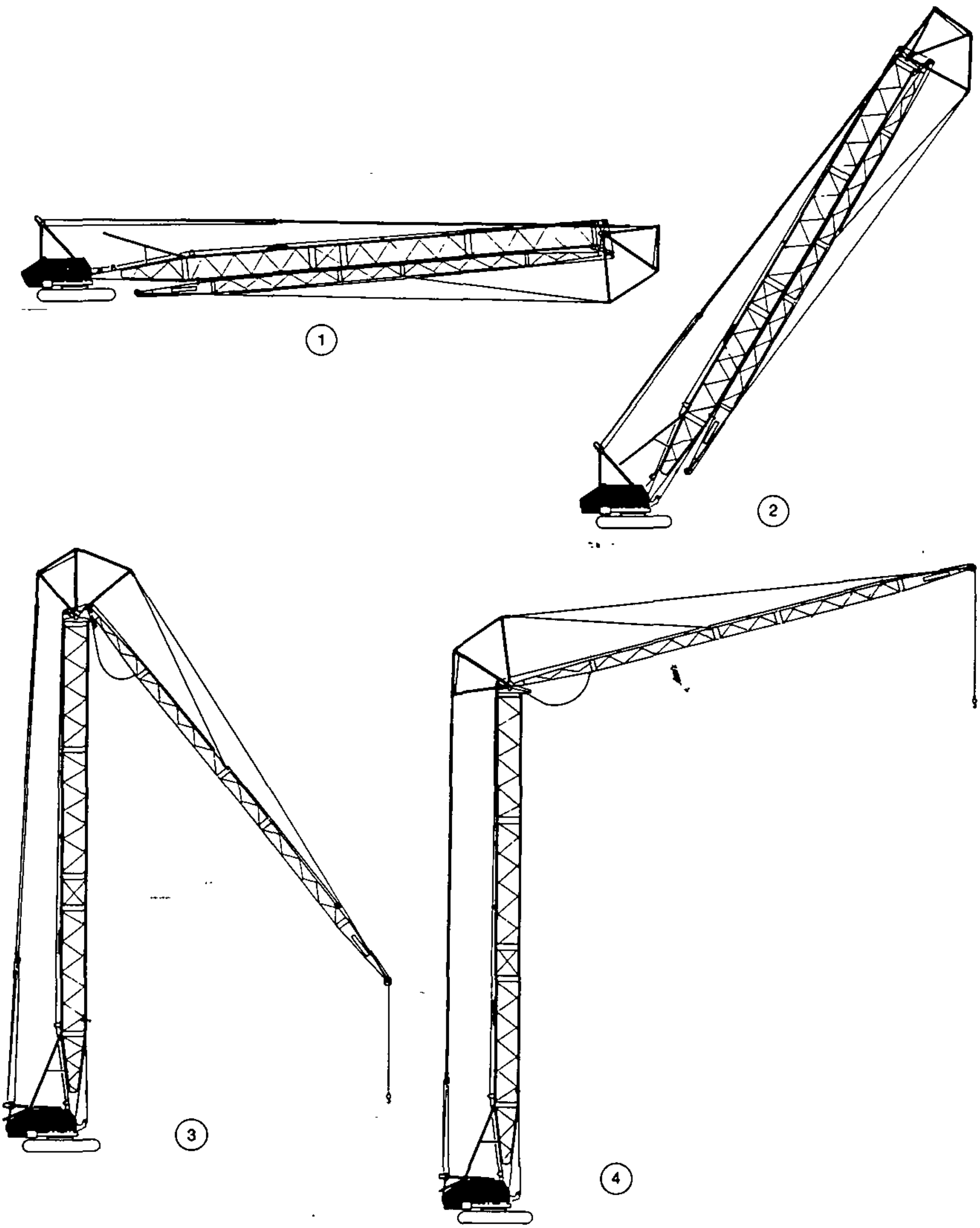
IT MAY BE ADVANTAGEOUS, WHEN USING A "FIXED BASE" CRANE, TO SELECT ONE THAT HAS THE ABILITY TO INCREASE ITS OWN HEIGHT BY ADDING SECTIONS.

Fig. 5.16



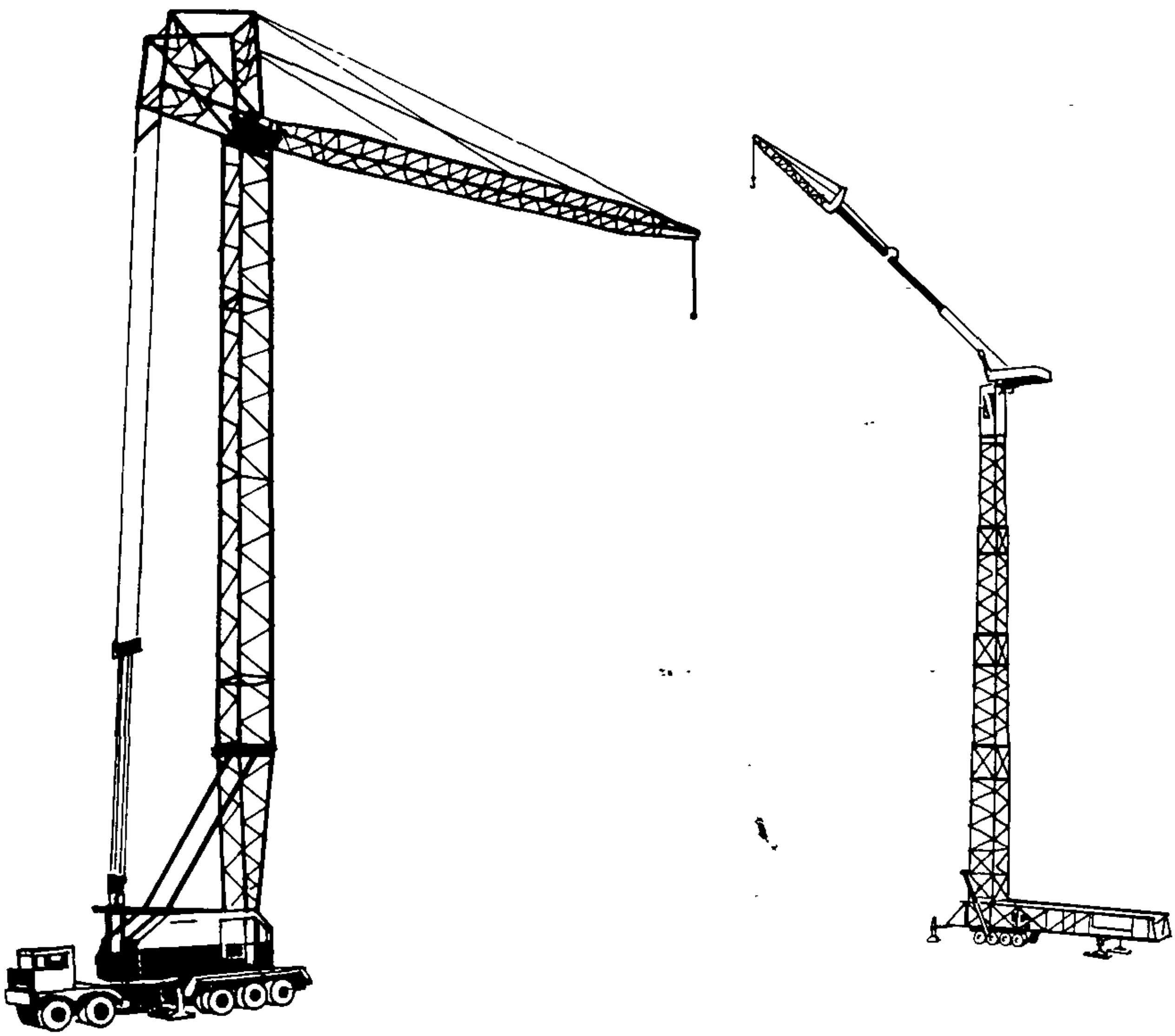
CLIMBING CRANE CONFIGURATION.

Fig. 5.17



CRAWLER MOUNTED TOWER CRANE.

Fig. 5.18



TRUCK MOUNTED AND TRAILER MOUNTED TOWER CRANES.

Fig. 5.19

— Have a safe access route to and from the cab (Fig. 5.26). The means of access to the cabin should ensure that there is no danger of the operator being trapped in the cabin. Where access is through the floor, there must be sufficient room in the cabin for the driver to stand beside the trap and raise it without difficulty, and the trap door must be of adequate size.

— If the vertical position of the control cabin is adjustable, the means of access and exit must be effective at all levels. Means should be provided for supporting the cabin without imposing a load on the cabin hoisting rope and if the hoisting mechanism for the cabin is hand operated, an effective holding arrangement such as a ratchet and pawl should be provided.

— Have lighting in the cab adequate to enable the operator to see clearly enough to perform his work.

— Be provided with an operator's seat that is fully adjustable (fore-aft, up-down, and tilt).

— Have all walking surfaces to and from the cab of the anti-skid type. The most common accidents that happen to crane operators are slips and falls while walking, climbing or working on the structure. A spray-on anti-skid paint would prevent most of them if it were applied to all surfaces to which they have access.

— Have guardrails provided on all outside and access platforms (Fig. 5.27). If they are too narrow for guardrails, hand holds, steps or safety lines should be provided at convenient points above the platform.

— Have hand holds and steps to facilitate entrance to and exit from the cab.

— Have the following accessories:
  (a) Windshield wipers to cover the operator's normal viewing area.
  (b) A cab heater capable of maintaining the temperature in the cab at 50°F minimum when the outside temperature is —20°F.
  (c) A windshield defroster.
  (d) A CO_2 dry chemical or equivalent fire extinguisher. It is recommended that it be a 5:B:C type.

Wind Balance

When tower cranes are left overnight they must be able to weathervane, i.e., their main jib turns freely in the wind and aligns itself with the direction the wind is blowing, much like a flag (Fig. 5.28). They are able to do this because the surface area of the main jib is greater than the counter jib (Fig. 5.29). If, however, someone hangs a large name board on the counter jib, this wind balance is upset. This practice may have serious consequences.

It may mean that the crane will not be able to swing into the wind when working or the result may be crane failure during high winds because of the extra sail area of the crane. (Fig. 5.30)

The wind balance can be checked as follows: When the wind speed is around 6 - 12 mph slew the jib side on to the wind and release the coupling of the slewing gear units. If the wind balance is correct the jib should slew in the direction of the wind. If the jib remains in position reduce the sail area of the counter jib by removing the advertising signs. If the crane still does not slew into the wind, put the signs on the main jib at its tip. (Fig. 5.31)

Operating Controls

— All controls used during the normal operating cycle must be located within easy reach of the operator and allow him ample room for operation.

— All controls should be of the dead man type in that they return to neutral automatically when released. (Fig. 5.32)

— The controls should be arranged so that accidental displacement is prevented and inadvertent pressure on them does not cause the crane to be set into motion.

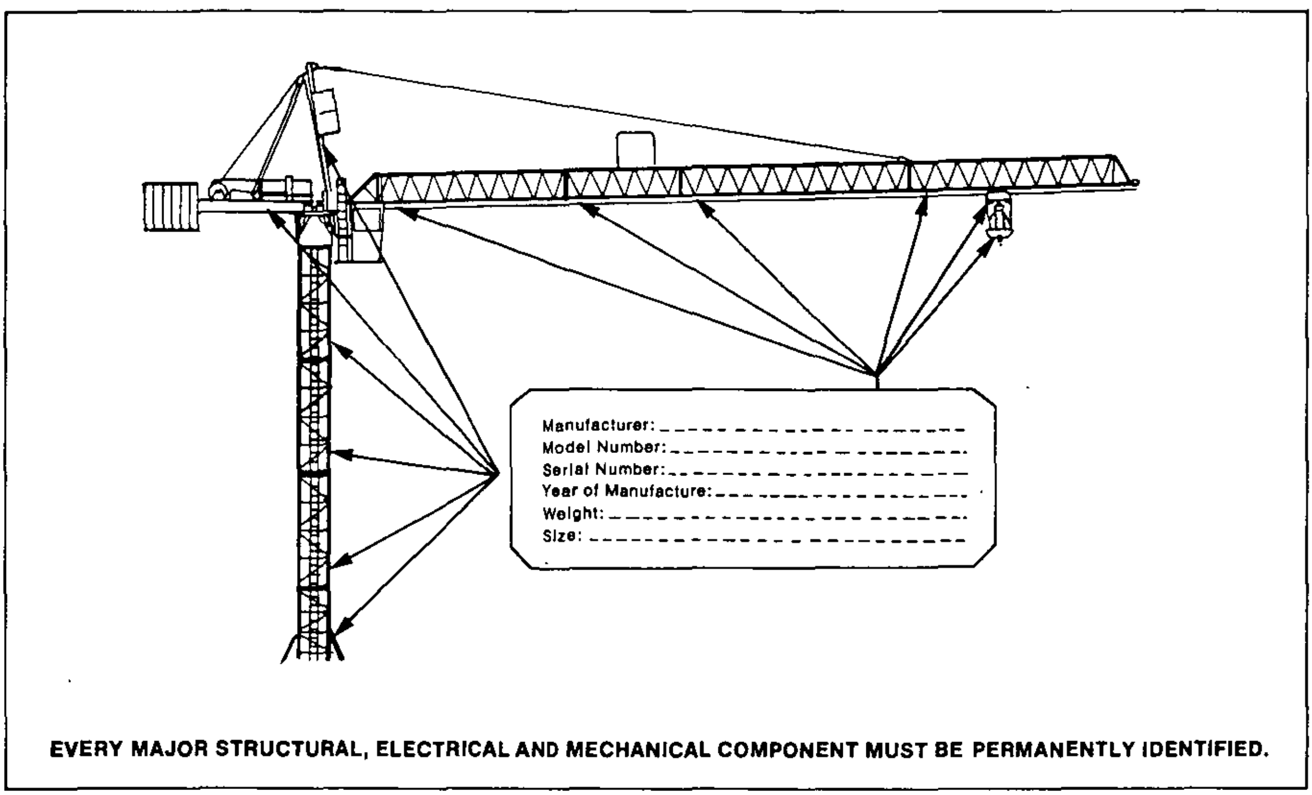
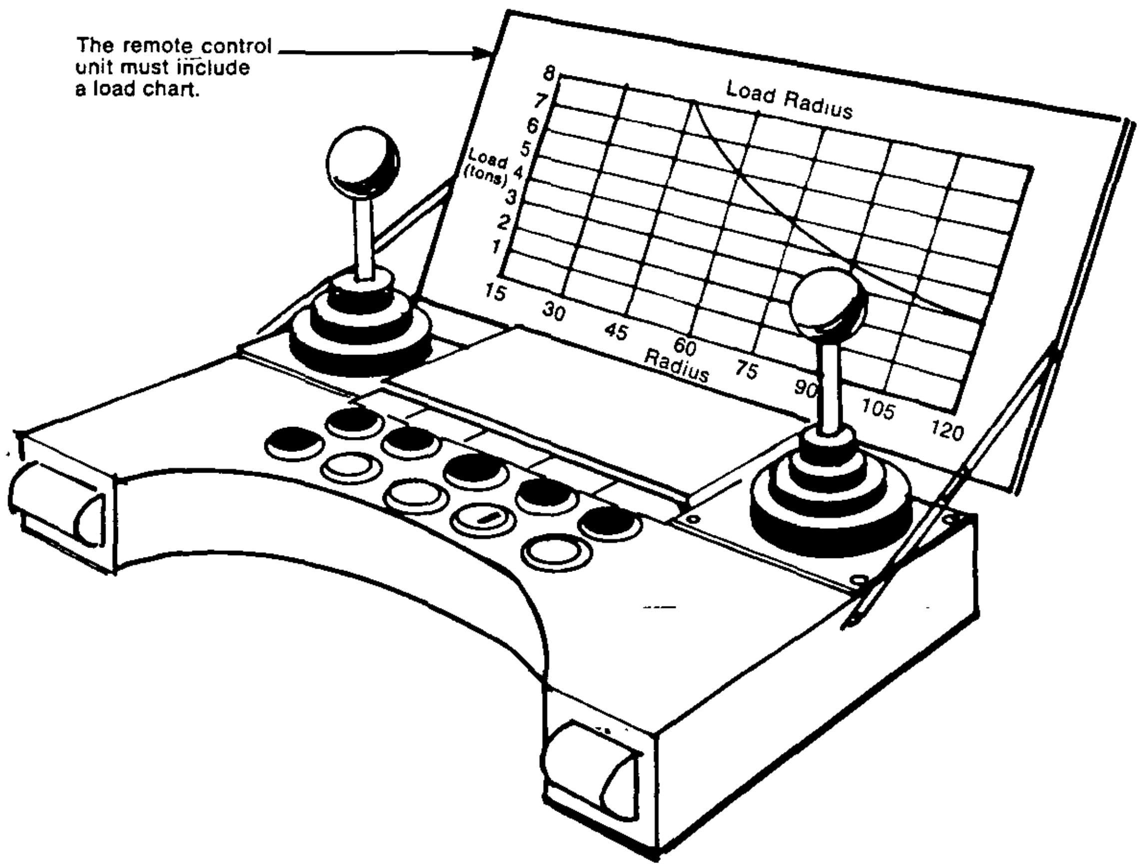
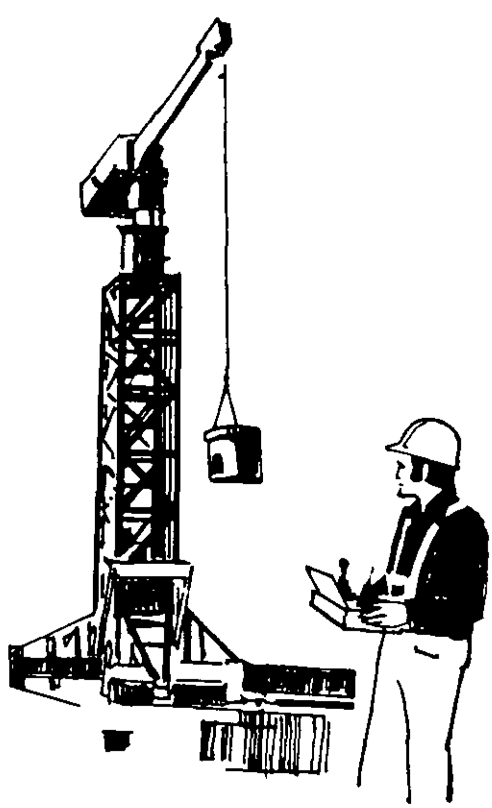


Fig. 5.20





REMOTE CONTROL UNIT.

THE LOAD CHART MUST BE VISIBLE TO THE OPERATOR AT ALL TIMES.

Fig. 5.21

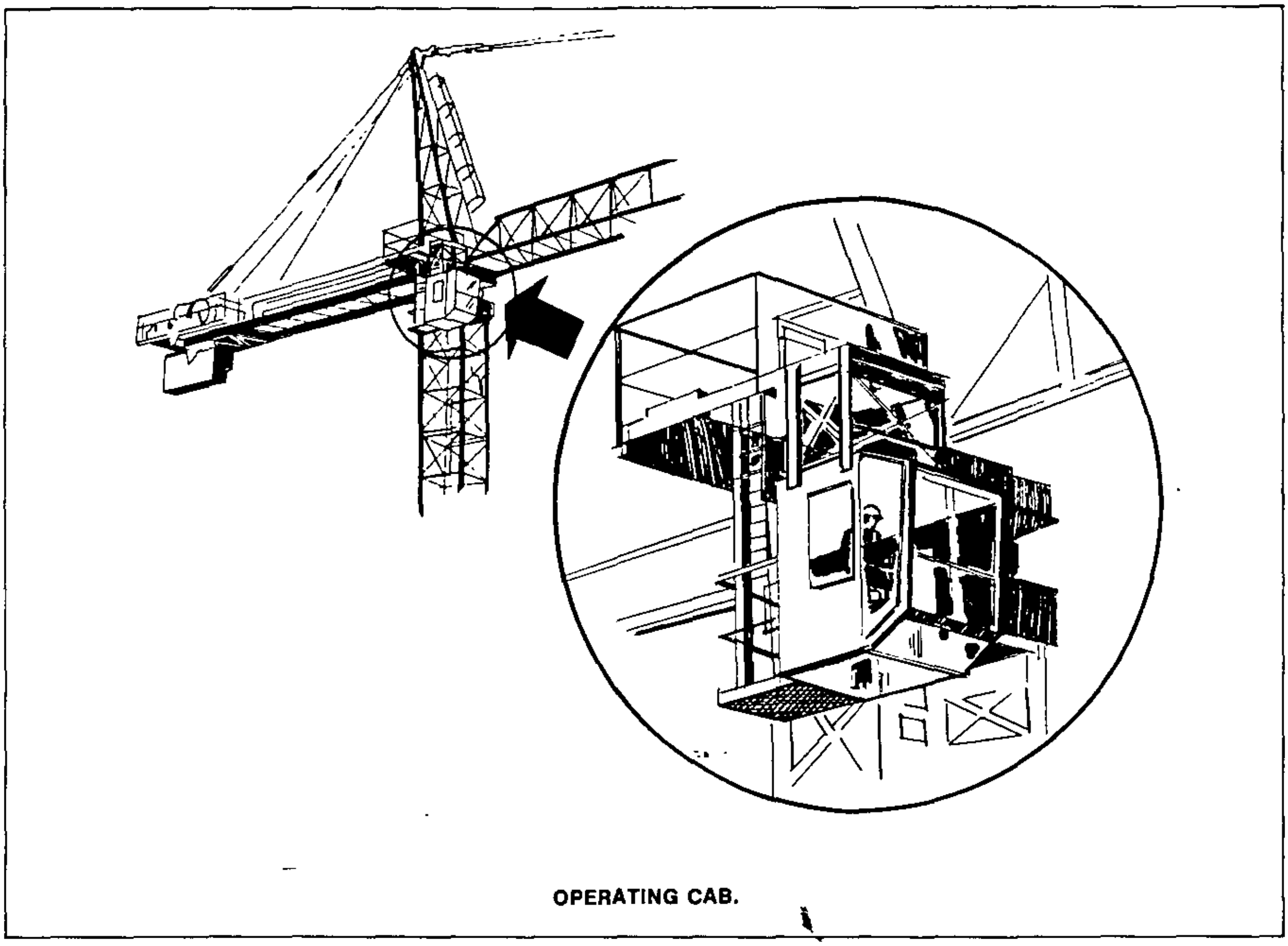


Fig. 5.22

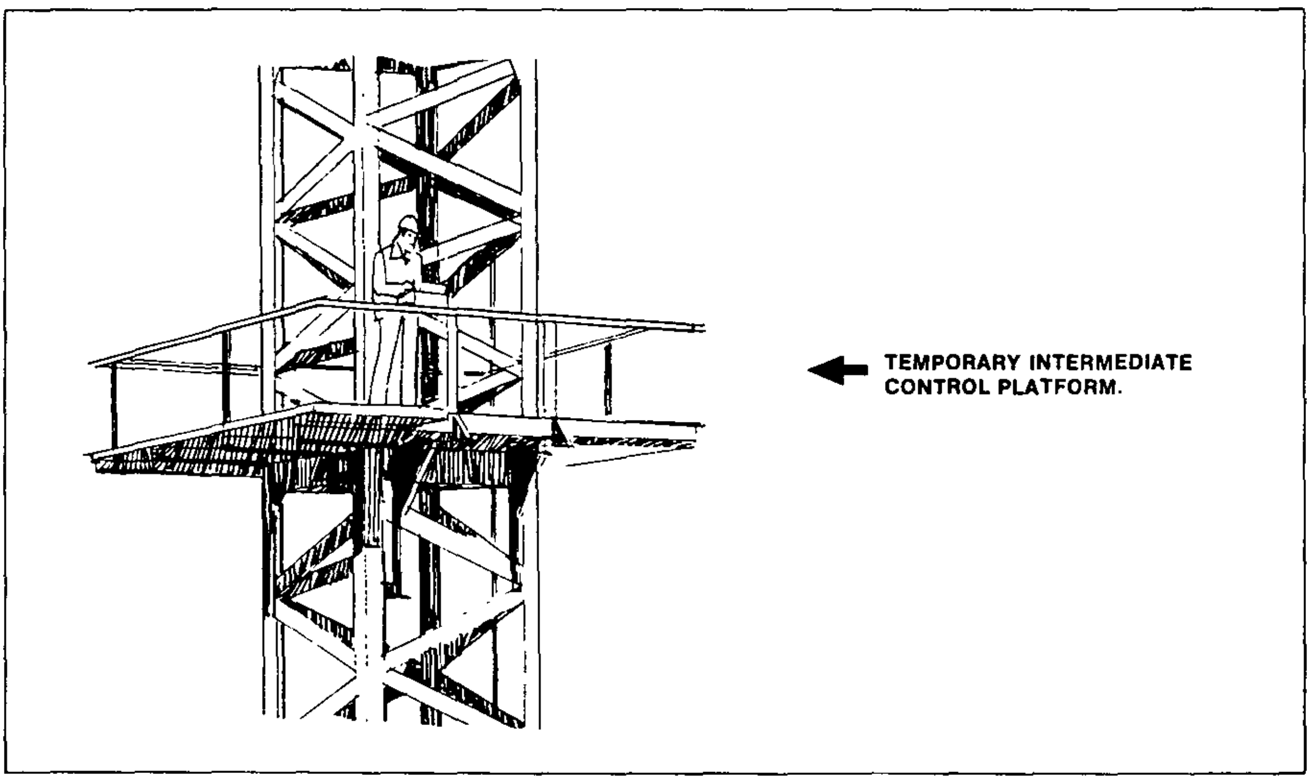


Fig. 5.23

— The voltage used in the control circuits should not exceed 55 volts.

— The main power switch should be lockable and located within easy reach of the operator.

— Each control must be clearly marked to indicate its function. (Fig. 5.32)

— All controls should be installed so as to move in the direction of the resultant load movement or machine movement. (Fig. 5.33)

Electrical Equipment and Wiring

— All electrical components must be well grounded to the crane's structure which must in turn be connected to ground. (Fig. 5.34)

— All electrical equipment and wiring must be installed in accordance with the Canadian Electrical Code.

— Strain-relief connectors should be used at the connection of the power cable to the crane tower connection to protect the cable at this point.

— All electrical equipment and connectors must be weather proofed.

— Power feeders for the crane which run inside the crane tower must be securely fastened at regular intervals and properly grounded.

Gear Boxes

— Gear boxes should be designed so that the gears will be automatically lubricated. The gears should also be readily removable and the boxes oil tight. They should be rigidly constructed and fitted with inspection covers and lifting lugs where necessary. Facilities for oil refilling, adequate breathing, drainage and a means for inspection of oil levels must be provided.

— The bases should be machined and seated and positively located on machined surfaces unless the gears are shaft mounted and the gear boxes form an integral part of the gear. In the latter case proper shaft alignment must be ensured.

— On manually operated gear changing levers provision must be made to permit them to be positively locked in position. Provision should also be made to prevent release of the hoisting block while changing gear, and the operating instructions should warn the operator to either lock or lower the block to the ground before attempting to change gears.

Drum Assemblies

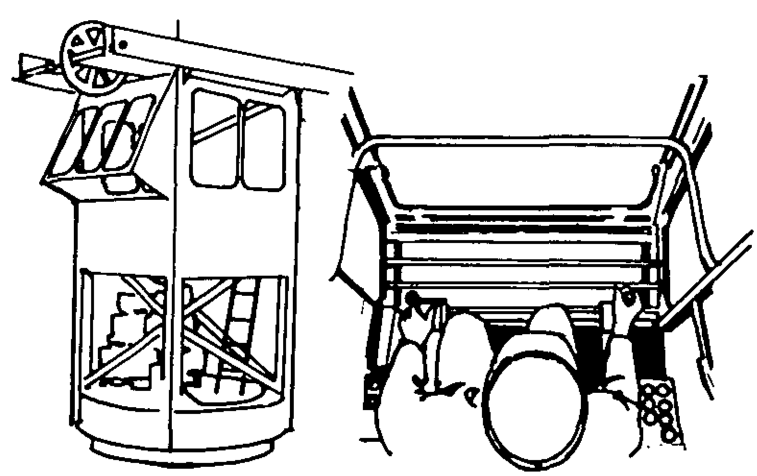
The requirements for tower crane drum assemblies are identical to those for mobile cranes. Please refer to page 8 for the appropriate information.

Brakes

— Every brake on the crane must be fail safe in that the brake will be automatically applied wherever there is a loss of power (pneumatic, hydraulic or electric) (Fig. 5.35). These brakes must not release until the power has been

THE CAB MUST BE OF SUFFICIENT SIZE TO ALLOW OPERATION WITHOUT INCONVENIENCE.

Fig. 5.24



THE CAB MUST BE DESIGNED TO PROVIDE EXCELLENT VISIBILITY.

Fig. 5.25

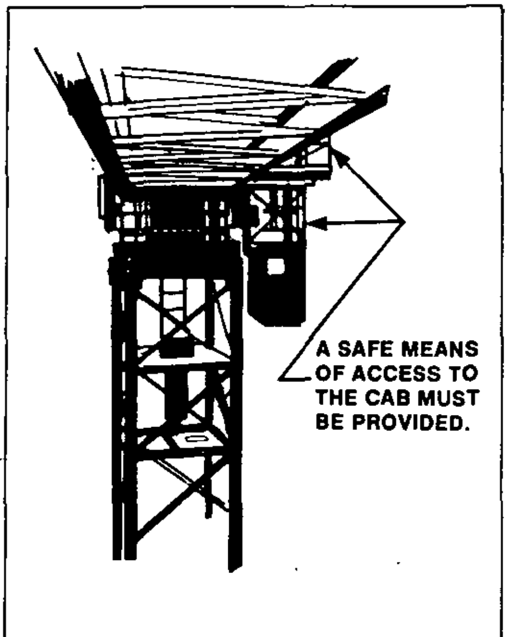


Fig. 5.26

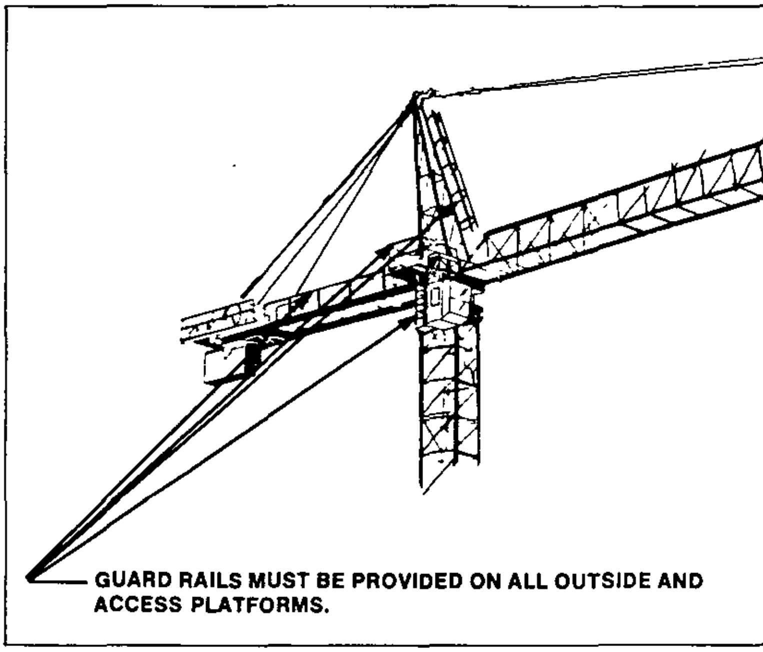
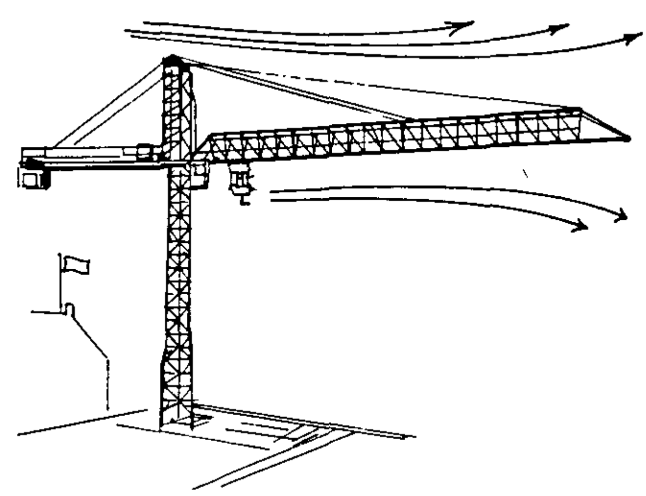


Fig. 5.27

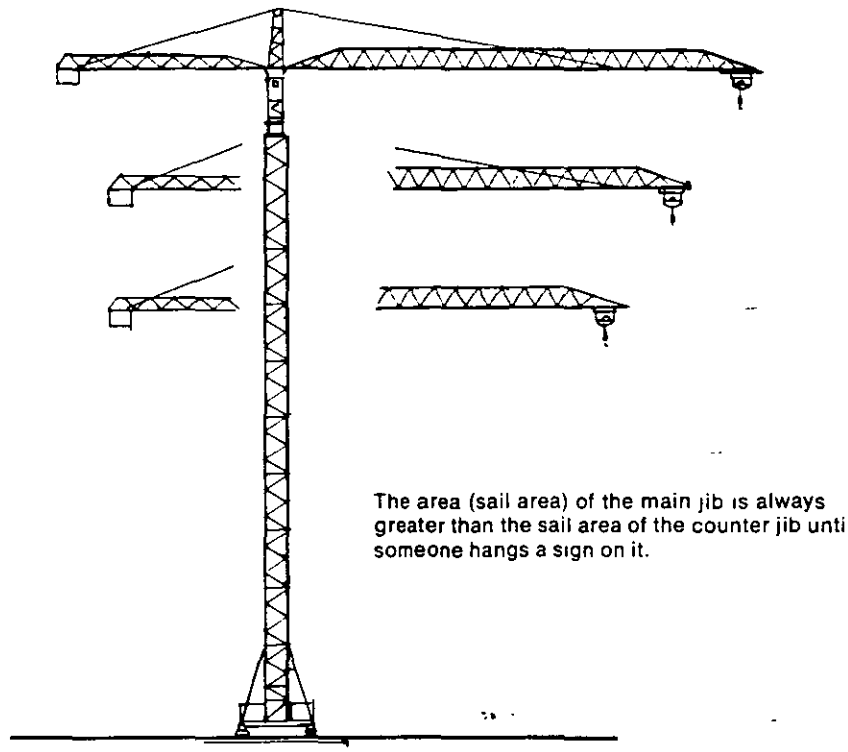
restored and then only when deliberately released. The application of the brake must have a direct effect on the hoisting drum and as such no belts or chains are allowed between the brake and the drum (Fig. 5.36). The resistance torque of the brake should be not less than 150% of the running torque of the prime mover. In hydraulic drives using a positive direct system of holding the load, the hoisting brake may be used only as an emergency fail-safe device and its application and torque must be as recommended by the manufacturer.

— Where electro-mechanical brakes are used, they should be designed so as to apply a braking torque as soon as the current is cut off. Arrangements must be made to prevent the brake magnet from being energized by secondary current when the main supply is interrupted. The brakes must not be released until the driving unit is energized and capable of transmitting torque to the drum.



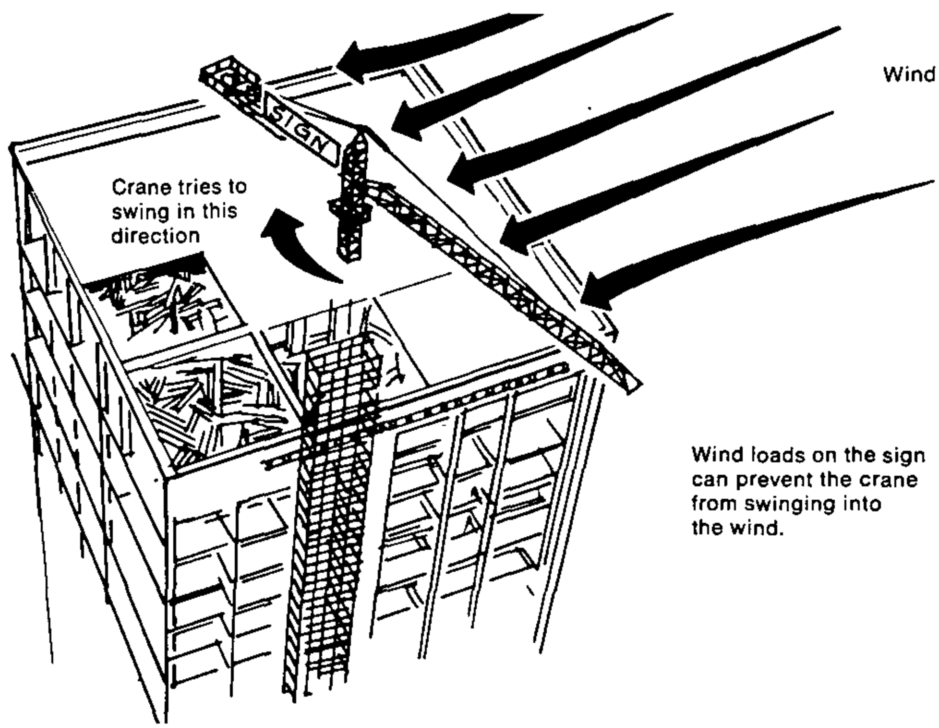
WHENEVER A TOWER CRANE IS LEFT UNATTENDED IT MUST BE ABLE TO WEATHERVANE, I.E. ALIGN ITS BOOM WITH THE DIRECTION IN WHICH THE WIND IS BLOWING.

Fig. 5.28



TOWER CRANES ARE ABLE TO WEATHERVANE BECAUSE THE SAIL AREA OF THE MAIN JIB IS GREATER THAN THE SAIL AREA OF THE COUNTER JIB. ADDING A LARGE SIGN TO THE COUNTER JIB MAY UPSET THIS CONDITION.

Fig. 5.29



IF THE SIGN ON THE COUNTER JIB IS LARGE THE CRANE MAY NOT BE ABLE TO SWING THE COUNTER JIB INTO THE WIND.

Fig. 5.30

— All load holding brakes and clutches must have sufficient size and thermal capacity to control all rated hoist loads with minimum recommended reeving.

— The springs that apply the brakes should be of the compression type and should not be stressed in excess of 50% of the torsional elastic limit of the material.

— The stresses in any part of the brake structure (other than the springs), should not exceed 50% of the allowable stress for the material while the braking torque is being applied.

— The wearing surfaces of all brake drums must be machined and smooth and free from defects.

— The brake weights must be fixed securely to their levers.

— The brake blocks and lining must be protected from rain, grease and oil.

— All brakes must be provided with simple and easily accessible devices to compensate for the wear of lining except where the adjustment is automatic and not required during the life of the lining.

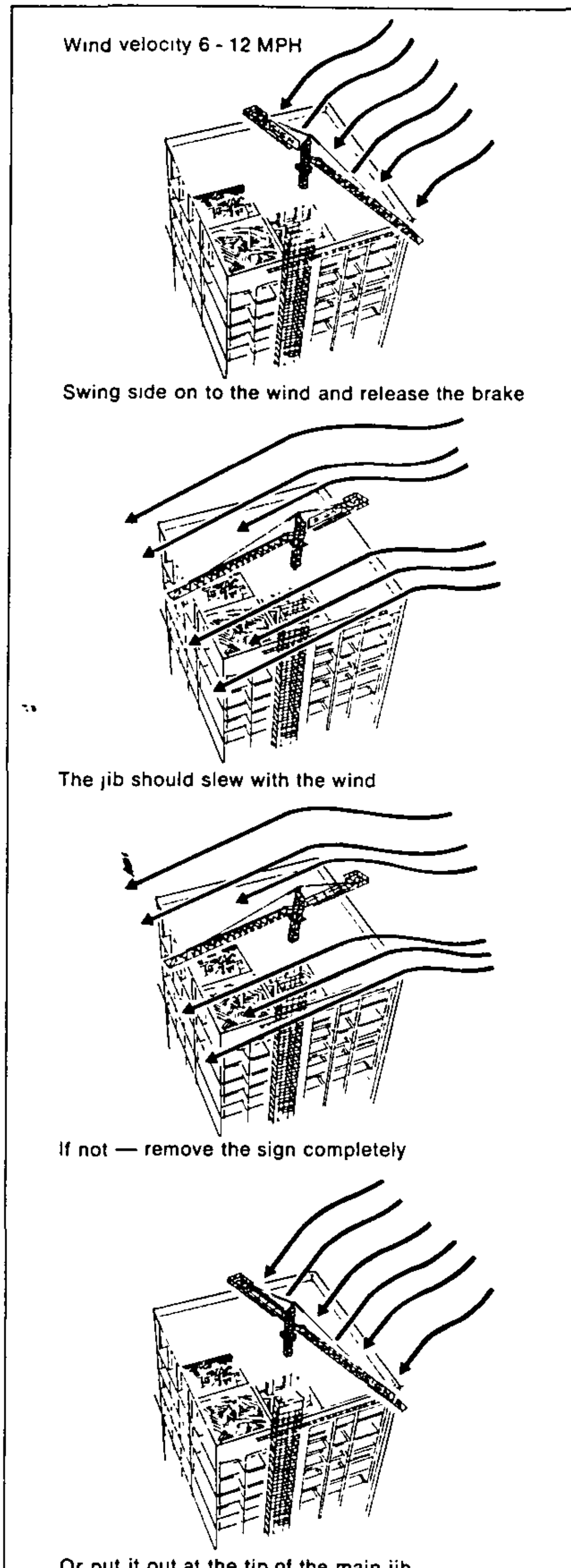
— The brake on the slewing drive must be capable of preventing the jib of the crane from drifting under a wind pressure up to the maximum operating wind pressure specified by the manufacturer. The brake must be designed so that the jib shall weather-vane with the wind when its velocity is greater than 40 mph.

— On rail mounted cranes each carriage drive must be equipped with a hydraulic or magnetic brake of sufficient size to stop the crane within a distance in feet equivalent to 10% of the travelling speed of the crane in feet per minute. An arrangement must also be provided to prevent the crane from moving during high winds.

Note: Uncontrolled lowering of loads is a hazardous procedure. The lowering of loads solely under brake control should be done only if the crane is equipped with a speed limiter (a device which limits the line speed that can be reached when the brake is released) and only if the brakes are continuously controlled by the operator.

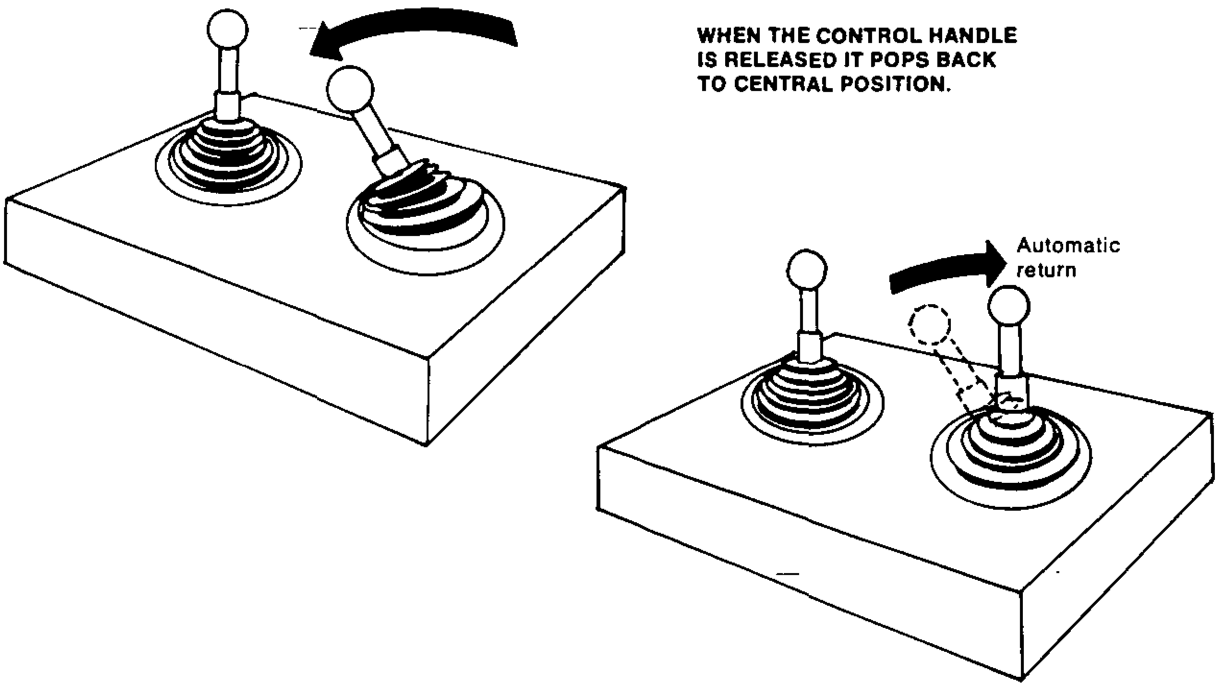
Ropes, Rigging, Reeving and Accessories

For complete information on these items please refer to the Construction Safety Association of Ontario "Rigging Manual".

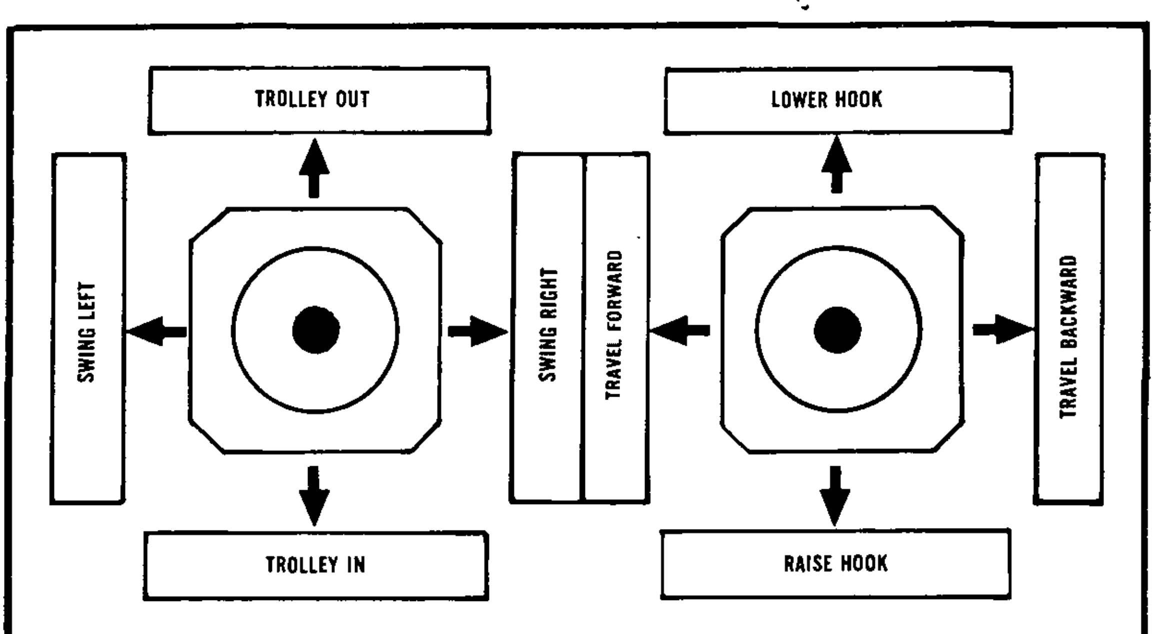


METHOD OF CHECKING WIND BALANCE.

Fig. 5.31

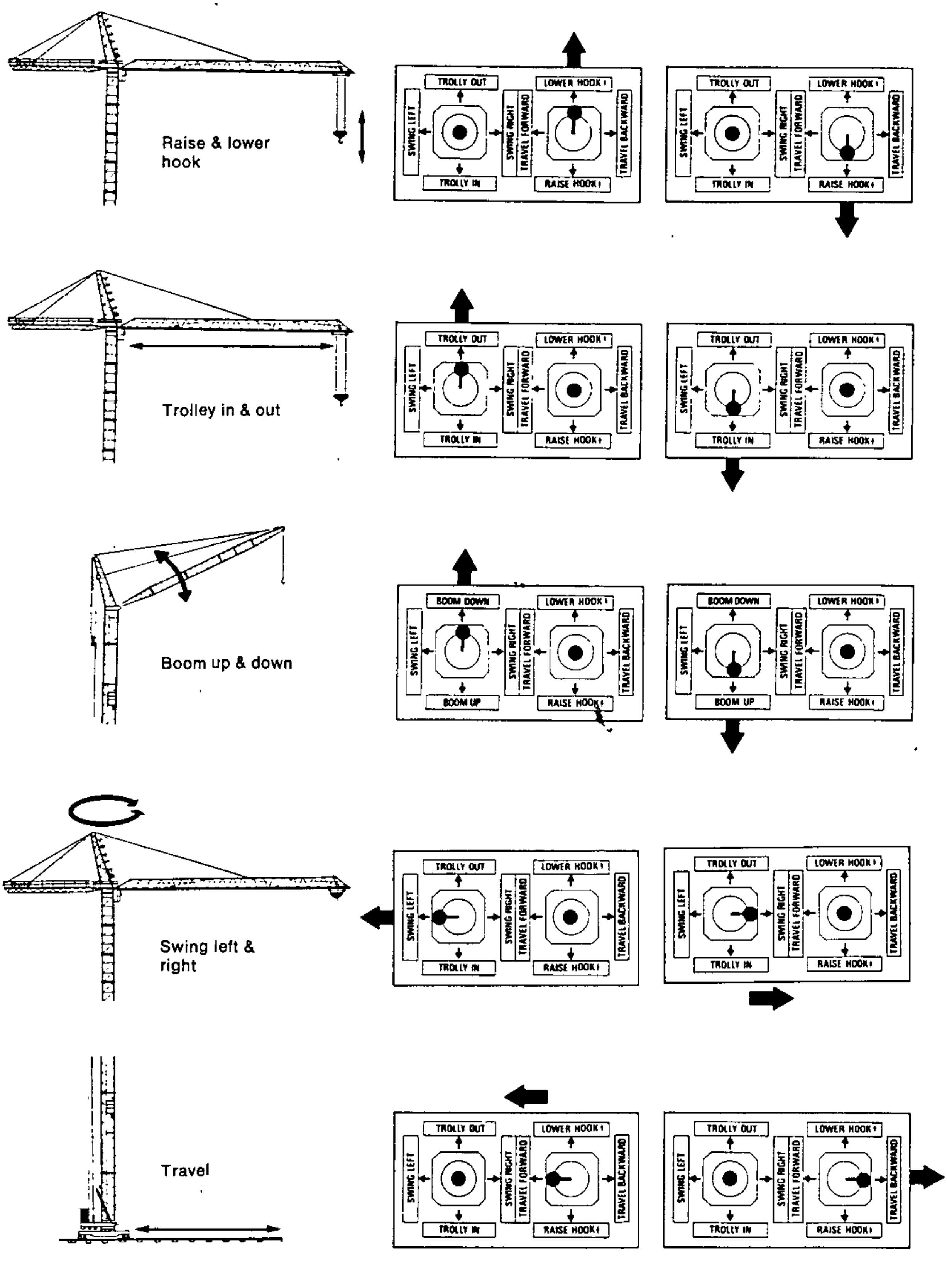


'DEAD MAN' CONTROLS



EVERY CONTROL SHOULD BE OF THE "DEADMAN" TYPE AND MUST BE CLEARLY MARKED.

Fig. 5.32



ALL CONTROLS SHOULD BE INSTALLED SO AS TO MOVE IN THE DIRECTION OF THE RESULTING LOAD OR MACHINE MOVEMENT.

Fig. 5.33

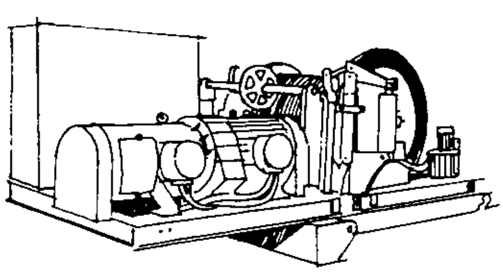
Sheaves

The sheave requirements on a tower crane are identical to those for mobile cranes. Please refer to pages 13-16 for the appropriate information.

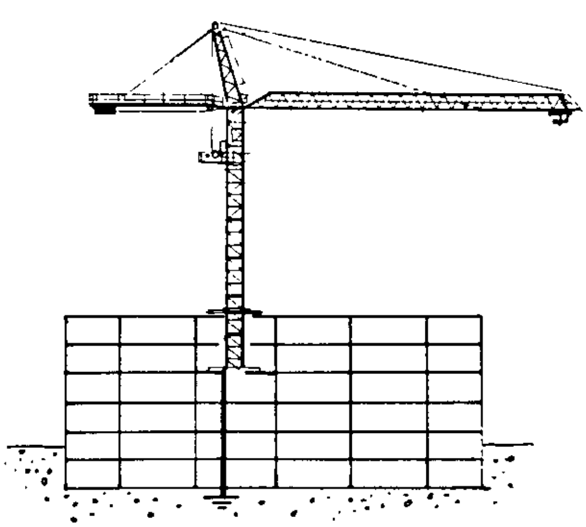
Boom Stops

Luffing boom tower cranes, like mobile cranes, require boom stops which will effectively prevent the boom from toppling or being pulled backwards over the tower. (Fig. 5.37)

The best type of boom stop is one that combines the functions of disengaging the boom hoist motor and physically stopping the boom as it reaches a predetermined maximum angle.



ALL ELECTRICAL COMPONENTS MUST BE GROUNDED TO THE CRANE STRUCTURE.



THE CRANE STRUCTURE MUST BE CONNECTED TO A SUITABLE GROUND.

Fig. 5.34

This type of stop usually takes the form of a spring or pneumatically loaded piston running in a closed cylinder that is mounted on the mast cap to intercept the boom as high above the boom hinges as possible. When the boom reaches a predetermined high angle it trips a cut-out switch that causes the boom hoist drum to stop.

Some devices employing the principles of limit switches which disconnect the boom hoist motor when the boom reaches a predetermined angle are available but are not recommended for use because they do not stop the boom from being thrown backward by boom whip or high winds. They are ineffective in preventing common boom accidents. Those devices are recommended for use in conjunction with the boom stopping devices however.

Guards and Protective Structures

The owner of the crane must ensure that all exposed moving parts such as gears, pulleys, belts, chains, shafts, flywheels, etc. which might constitute a hazard under normal operating conditions are guarded or fenced. As a rule of thumb, each guard should allow for routine inspection and maintenance and be capable of supporting, without permanent distortion, the weight of a man unless it is located where it is impossible to step on it. Lubricating points should be accessible without the necessity of removing the guards, and all friction brakes and clutches should be protected from the weather as much as practical.

Check that all electrical panels, components and wires are insulated in areas where the operator or maintenance personnel could accidentally come in contact with them.

Limit Switches

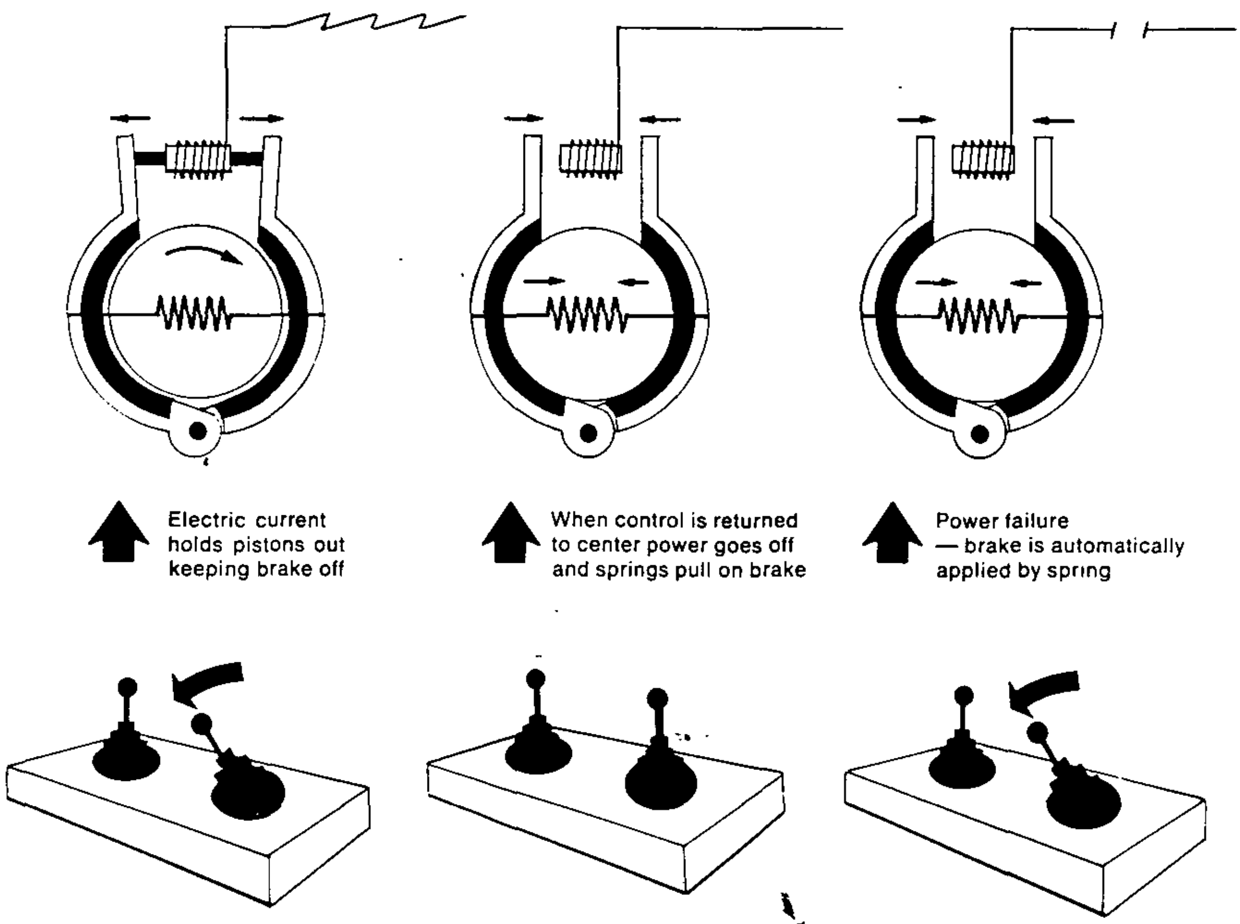
All tower cranes of every configuration must be equipped with built-in safety devices which operate automatically to prevent damage to the machine should the operator make an error. The most important of these are the limit switches which when properly installed and set virtually eliminate the possibility of crane overload or damage. (Fig. 5.38)

Every tower crane must have a:

— Hook height limit switch that causes the hoist drum to stop whenever the load hook reaches a predetermined maximum height position. (Fig. 5.39)

— Luffing boom limit switches that cause the boom hoist drum to stop whenever the boom is raised to too high an angle or lowered to too low an angle. (Fig. 5.40)

— Trolley limit switches that cause trolley mo-



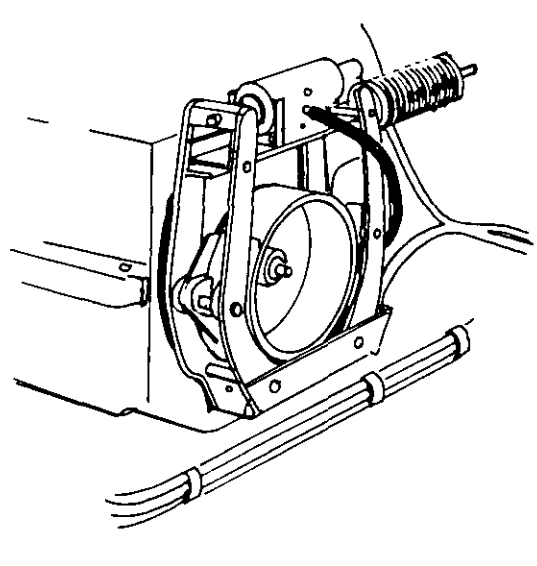
TYPICAL FAIL-SAFE BRAKE OPERATION.

Fig. 5.35

tion to stop whenever the trolley reaches a predetermined maximum out or maximum in position. (Fig. 5.41)

— Overload-limit switches that cause the hoist drum to stop whenever the load being hoisted exceeds the maximum rated load for any radius or boom angle or whenever the overturning moment exceeds the rated load moment.

The overload limits should consist of 2 cut outs. The first should be a hoist cable overload cut out that is set to cut out at 5% suspended overload (Fig. 5.42). The adjustment of this switch is independent of the length of the jib and corresponding permissible load. Its function is to protect the hoist rope and the hoist winch against overload (it accounts for the horizontal cut-off line seen on all load curves). The second overload limit switch should be a moment overload cut out (Fig. 5.43). This switch senses the tension in the jib pendants. This tension increases as loads move out on the jib or as



NO BELTS OR CHAINS ARE PERMITTED TO LIE BETWEEN THE BRAKE AND THE DRUM.

Fig. 5.36

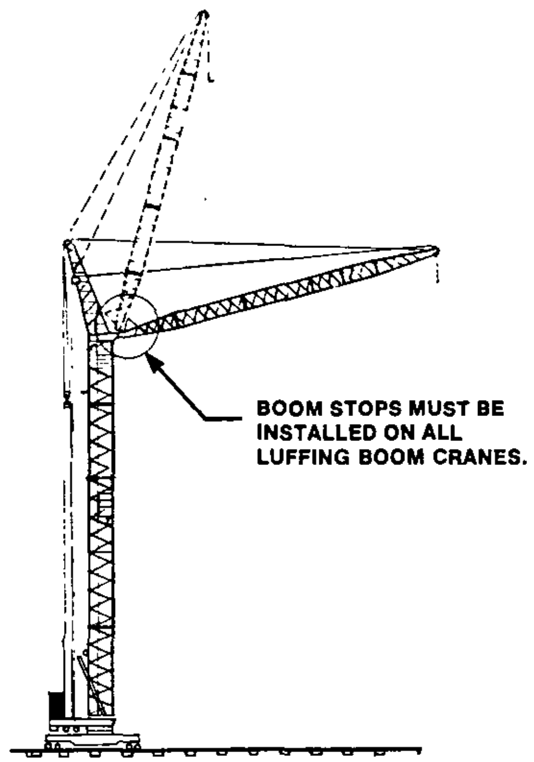


Fig. 5.37

heavier loads are lifted. It also must cut out at 5% overload (it accounts for the curved portion of the load curve). This switch must also cut out the trolley motion for it is possible to overload the crane with a load that is within the ratings at close radius simply by moving the trolley out along the boom. (Fig. 5.44).

— Travel limit switches for rail mounted cranes that apply the carriage brakes whenever the crane nears the ends of the tracks. (Fig. 5.45)

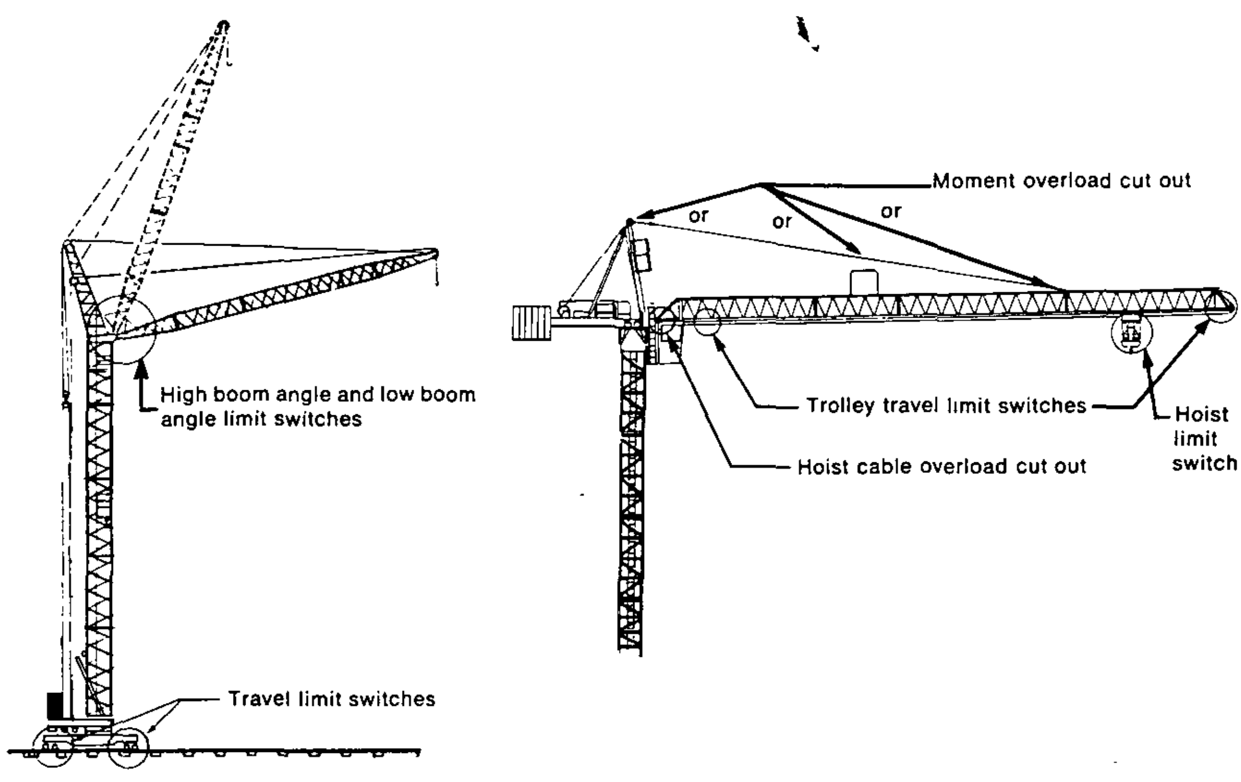
These devices are essential for the safe operation of the tower crane and once set they must never be tampered with. All craning should be stopped if any of them are inoperative.

Safety Features

It is strongly recommended that all tower cranes be equipped with the following safety features and devices:

— A metal receptacle secured permanently to the machine for storing tools and lubricating equipment.

— Adequate lighting for night operation. (Fig. 5.46)



TOWER CRANE LIMIT SWITCHES.

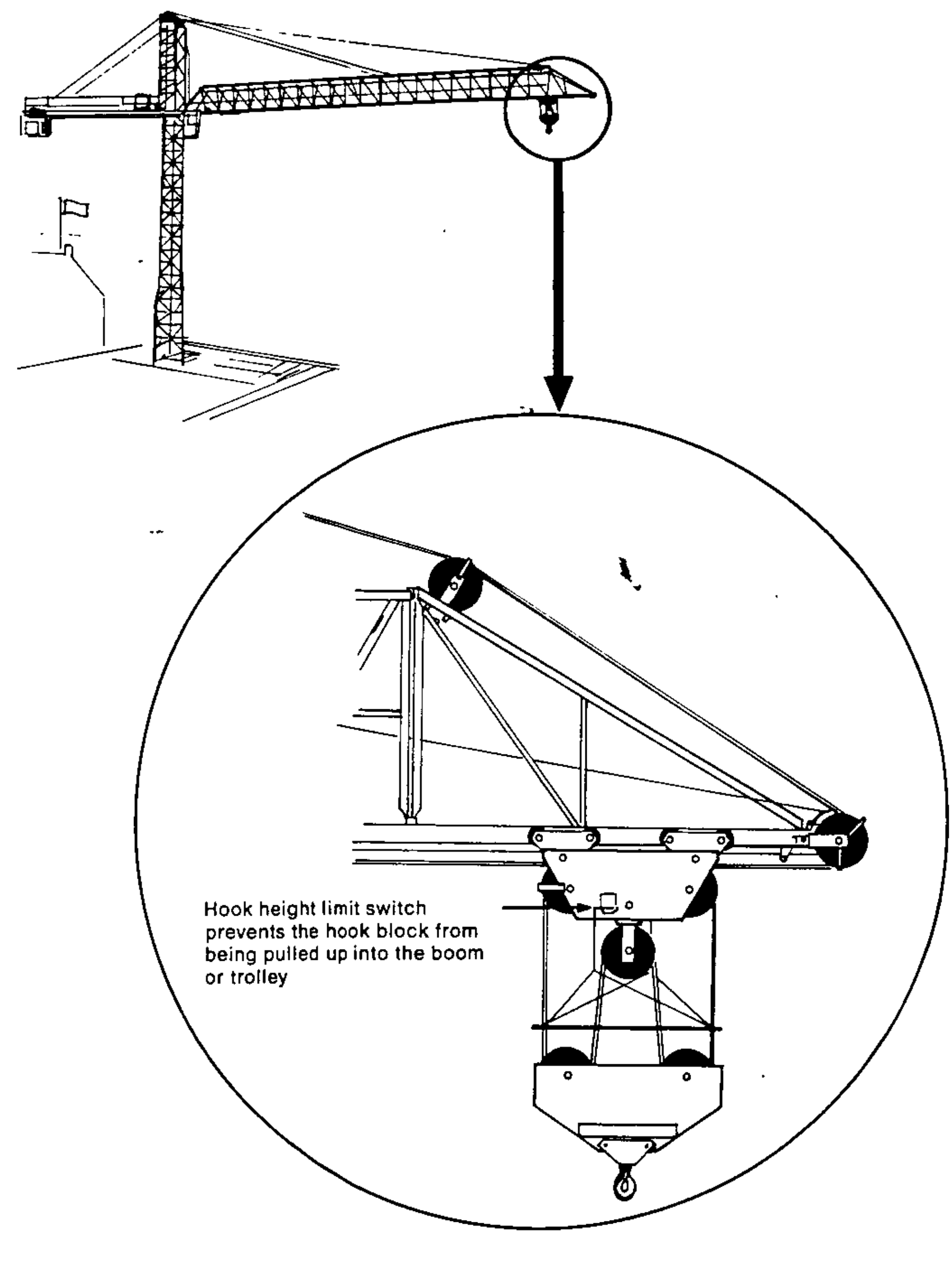
Fig. 5.38

— A 5:B:C fire extinguisher. Operating and maintenance personnel should be familiar with the use and care of the fire extinguishers provided.

— Boom angle indicators on all machines having booms capable of moving in the vertical plane. The indicator must be clearly visible and readable by the operator at his control station to within 1°.

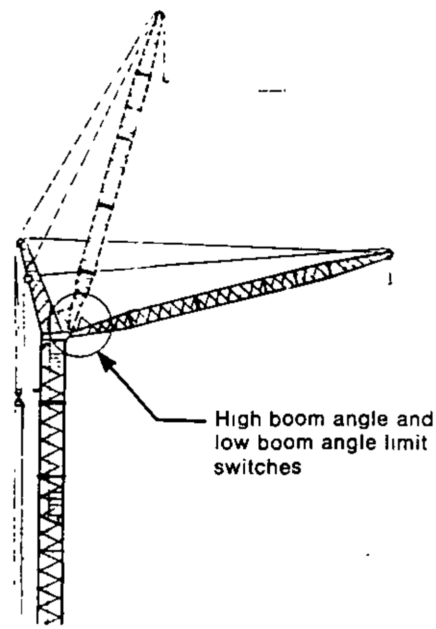
— An effective audible warning signal mounted on the carriage of rail mounted cranes. The controls for the device shall be within easy reach of the operator. (Fig. 5.47)

— Shock absorbing boom stops and boom hoist



HOOK HEIGHT LIMIT SWITCH.

Fig. 5.39



BOOM ANGLE LIMIT SWITCHES.

Fig. 5.40

safety shutoffs on all luffing boom cranes.

— In addition to providing a safe means of access to the crane, it is recommended that safety lines with runners for the attachment of safety lanyards be fitted to tower crane jibs, and that safety platforms be attached to the trolleys of saddle jibs to facilitate inspections and maintenance. (Fig. 5.48)

— Trolley radius markers that will relatively accurately inform the operator the radius of the load hook. (Fig. 5.49)

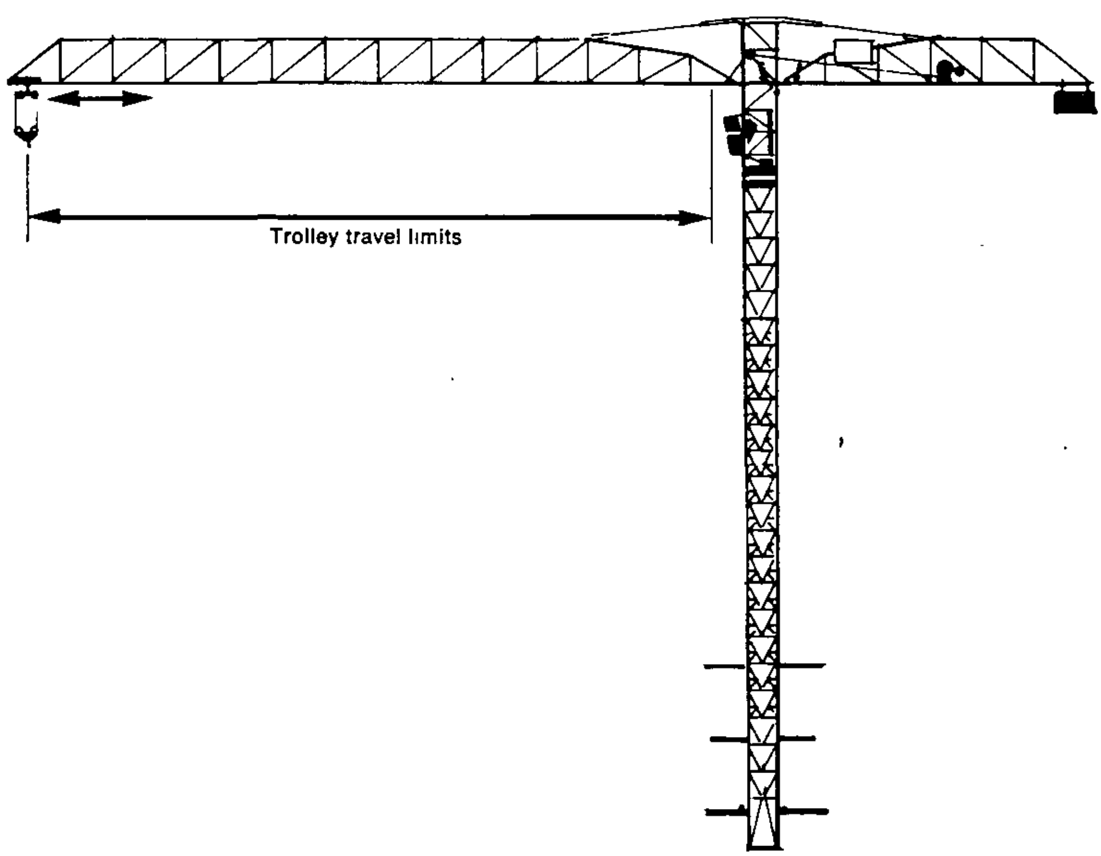
Equipment Handbook and Records

Manufacturers' manuals containing all pertinent data relating to operation and maintenance for the specific model of crane in use must be provided with each machine. The manual should include, but not necessarily be limited to, the following information:

— Equipment designation or type.

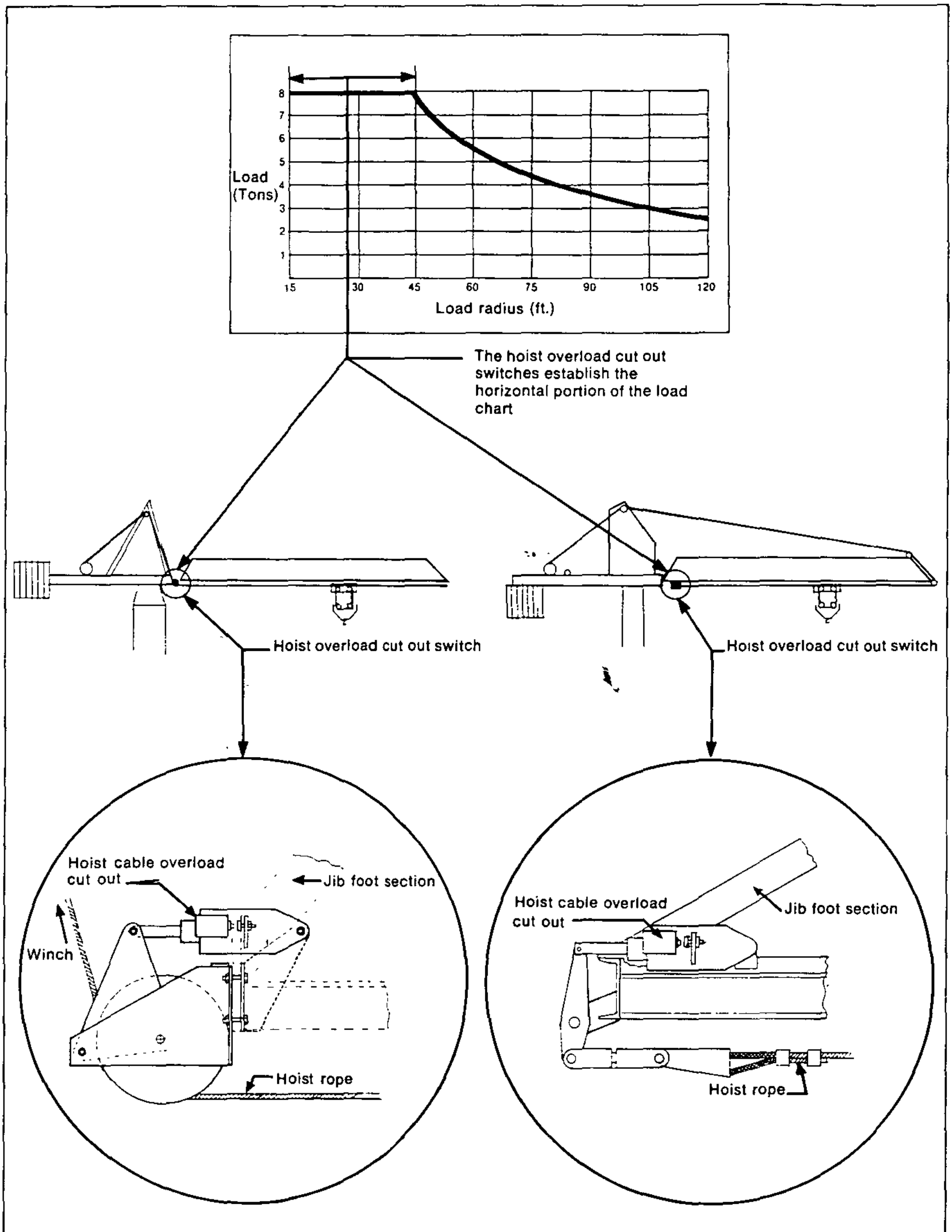
— Name of equipment manufacturer.

— Name of equipment designer, if other than the manufacturer.



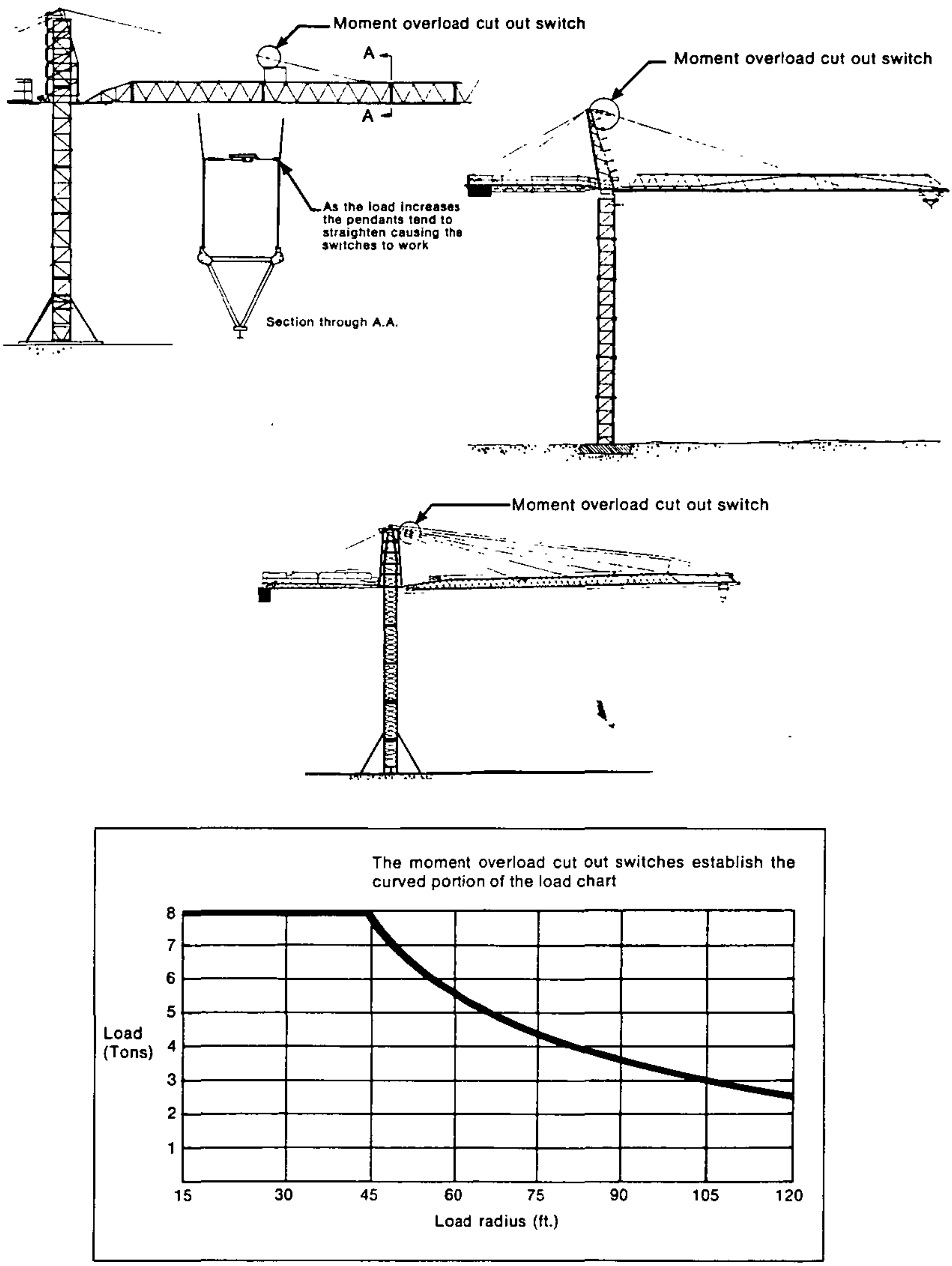
TROLLEY LIMIT SWITCHES.

Fig. 5.41



HOIST CABLE OVERLOAD LIMIT SWITCH.

Fig. 5.42



MOMENT OVERLOAD LIMIT SWITCHES.

Fig. 5.43

— Equipment model number and serial number.
— Year of original sale by the manufacturer.
— Weight of individual structural elements, mechanical components and individual counterweights.
— A copy of the load chart plus any and all rated combinations and variations in capacity and geometry.
— Inspection and maintenance procedures including:
  (a) Material specifications on jib and tower elements.
  (b) Welding specifications for all structural components.
  (c) Bolting and torquing specifications.
— Rigging specifications.
— Erection procedures.
— Climbing procedures.
— Operating precautions.
— Dismantling procedures.

If the equipment is not supplied with a log book then one should be started, maintained and kept on the work site for the regular, periodic recording of all inspections, tests, repairs, maintenance, and hours of service related to the machine. All entries should be dated and signed by the operator, repairman and supervisor. The machine owners should ensure that the log book remains with the machine and is kept up-to-date throughout the working life of the machine.

Refuse to purchase, lease, or use any piece of equipment which has been modified, altered, or otherwise subjected to any deviation, from the original manufacturer's specifications, in any way which could affect the safety of operation unless you have documented proof that the change was engineered and certified by a competent authority. In addition, check the documentation to ensure that any piece of equipment that had been damaged in any way affecting the safety of operation was repaired by reputable persons and certified by a qualified authority.

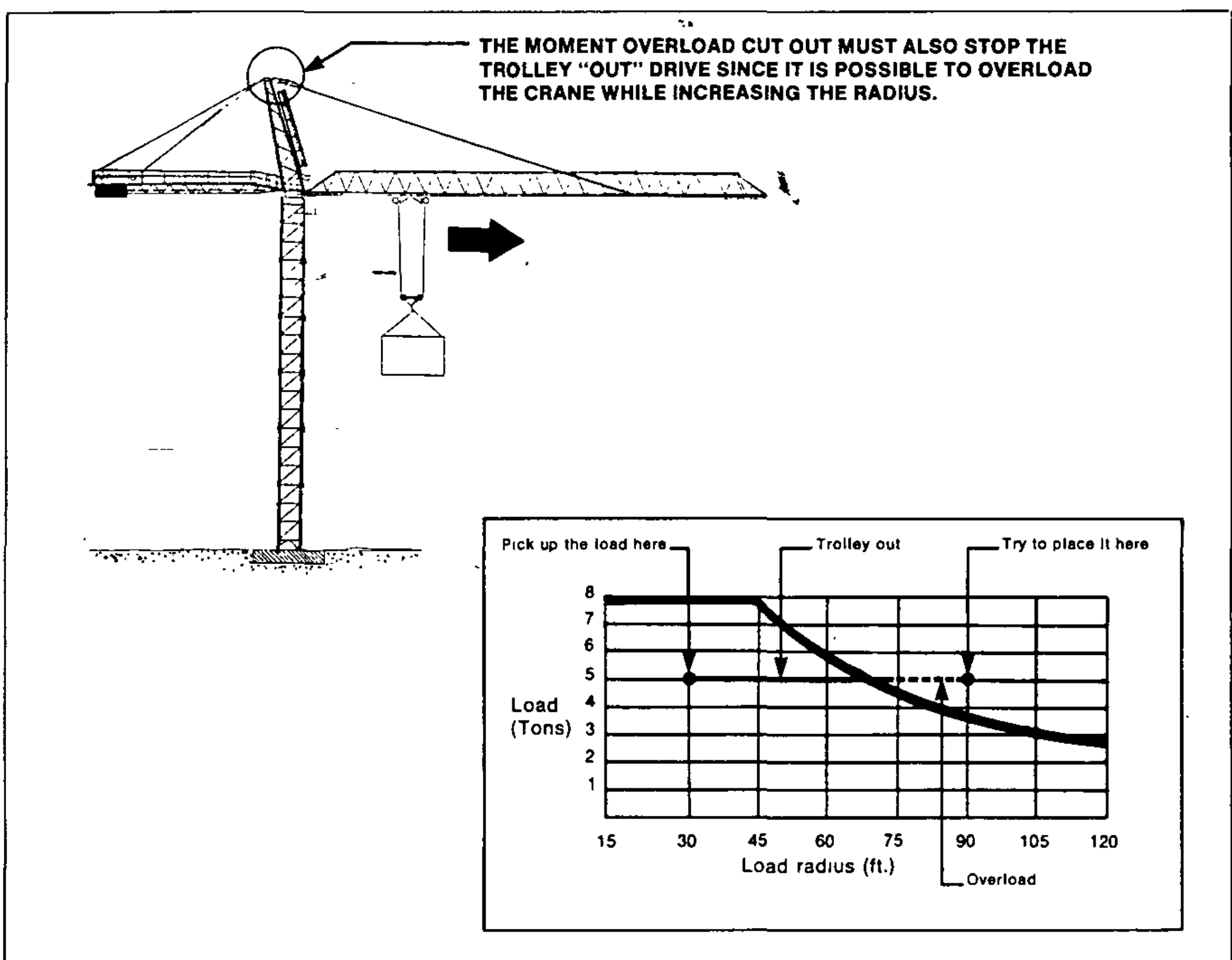
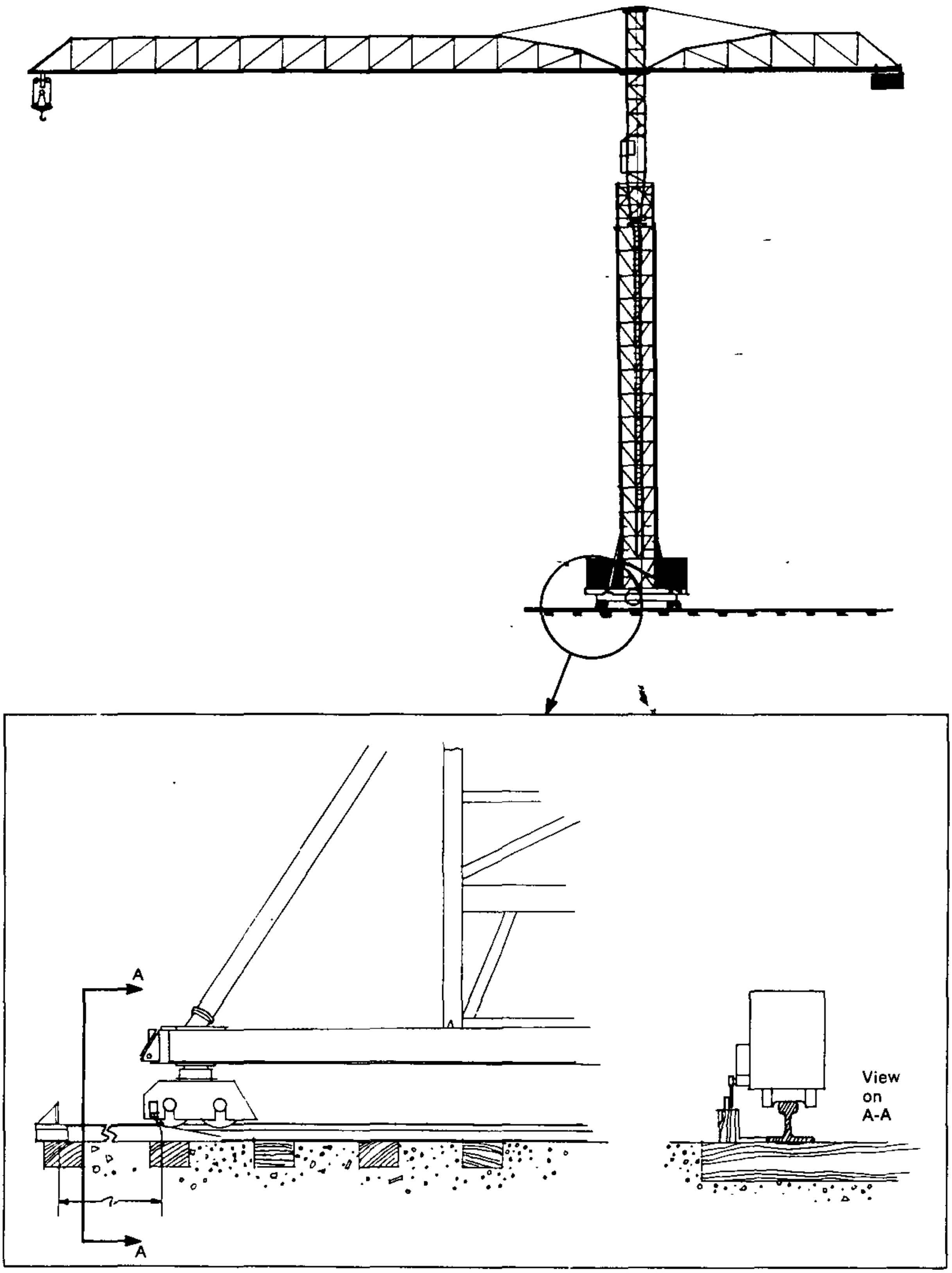
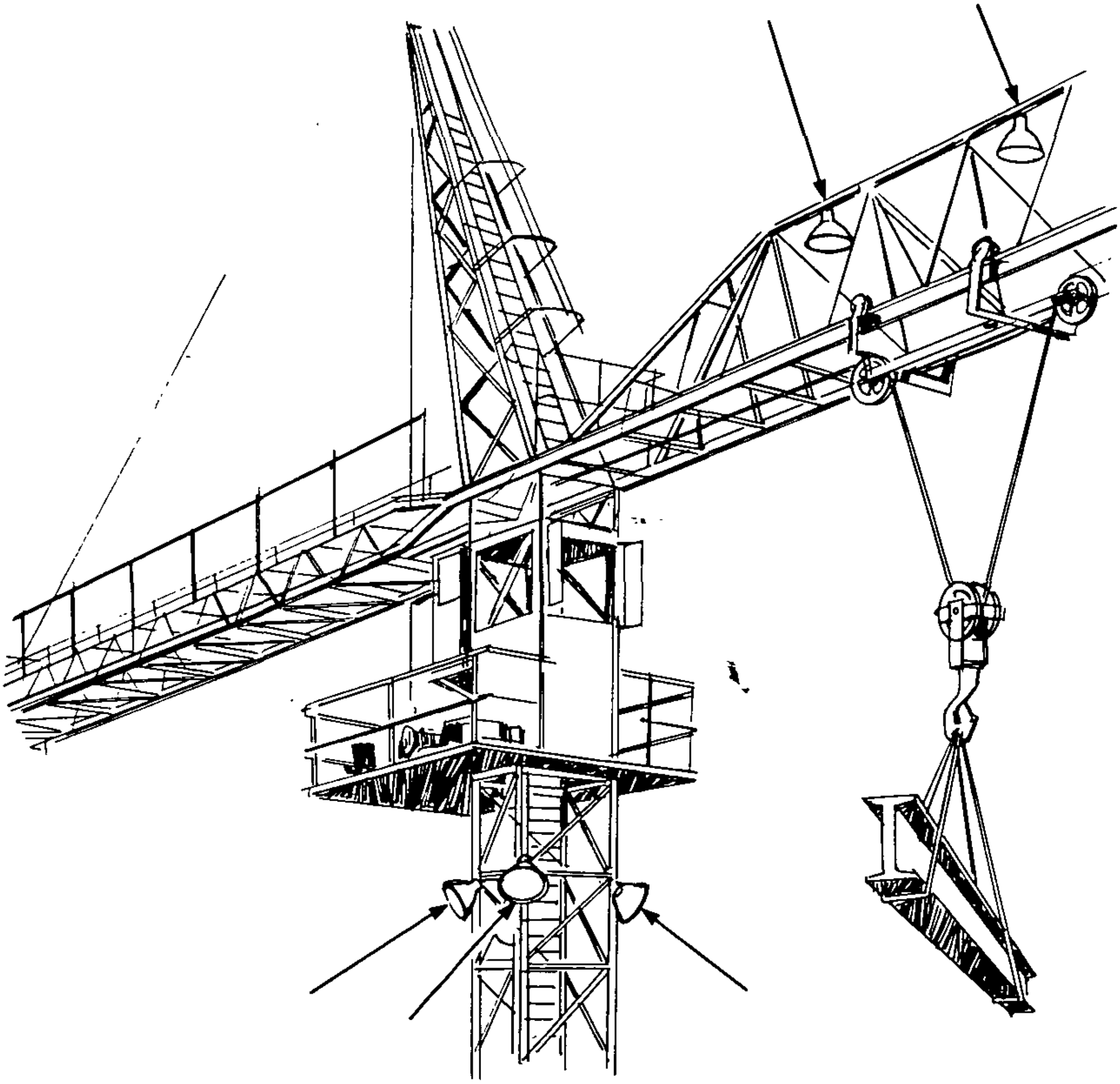


Fig. 5.44



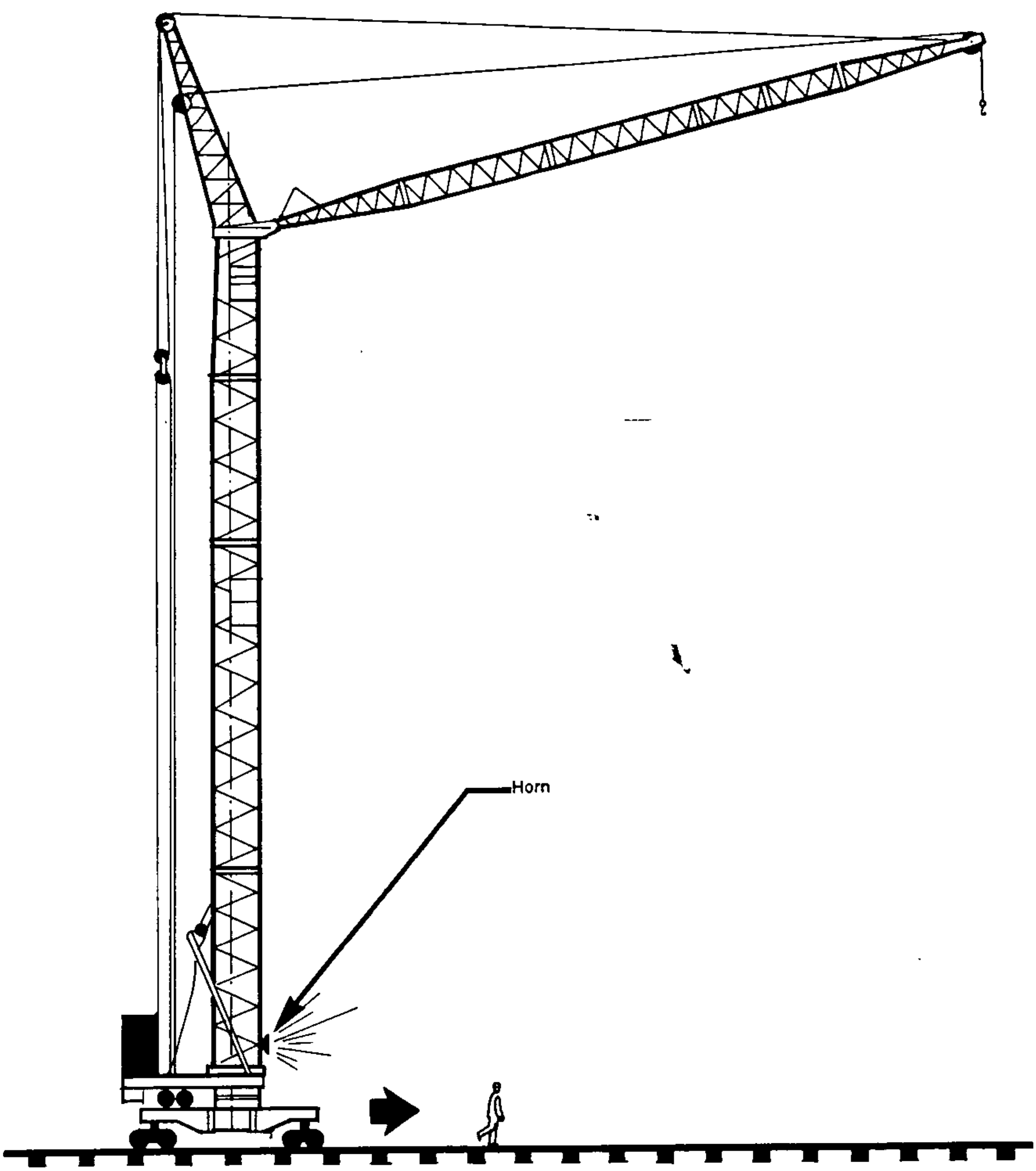
TRAVEL LIMIT SWITCHES FOR RAIL MOUNTED CRANES.

Fig. 5.45



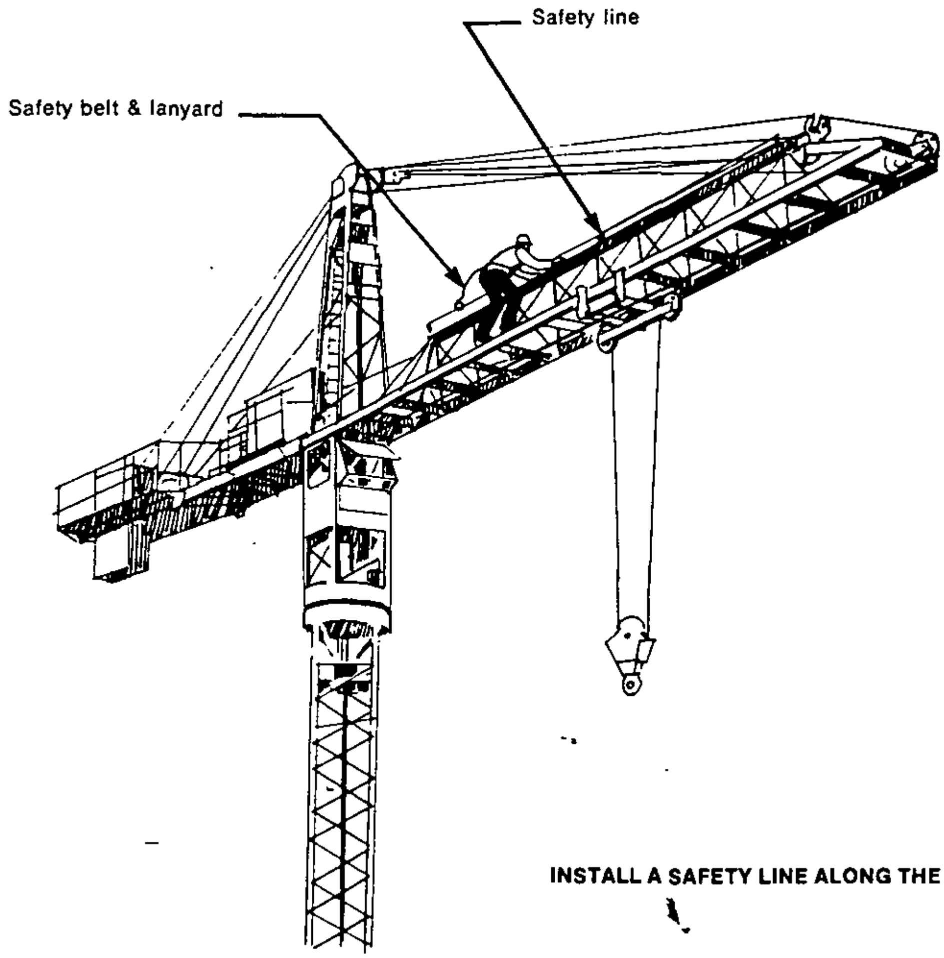
EVERY TOWER CRANE SHOULD BE EQUIPPED WITH SPOT LIGHTS TO PROVIDE ADEQUATE LIGHTING FOR NIGHT OPERATION.

Fig. 5.46

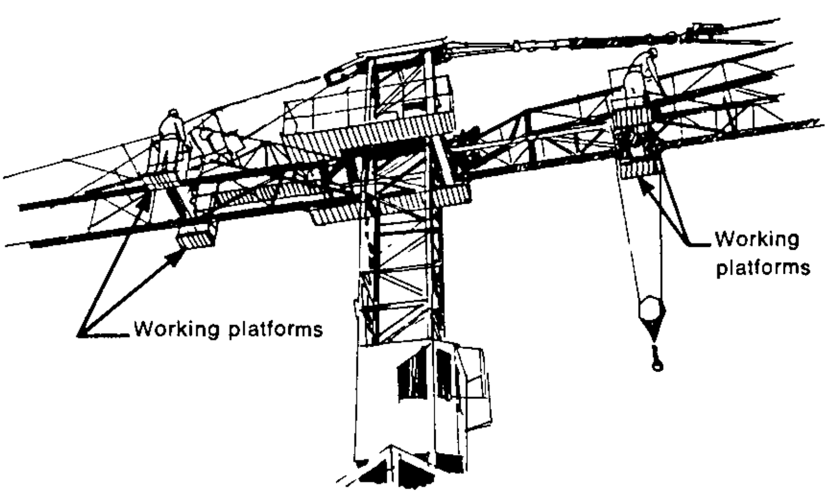


EVERY RAIL MOUNTED CRANE SHOULD BE EQUIPPED WITH A HORN THAT SOUNDS WHENEVER THE CRANE IS TRAVELLING.

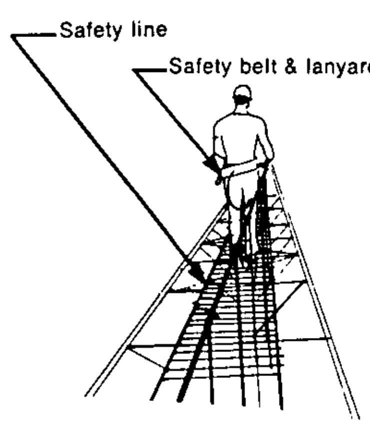
Fig. 5.47



INSTALL A SAFETY LINE ALONG THE JIB.

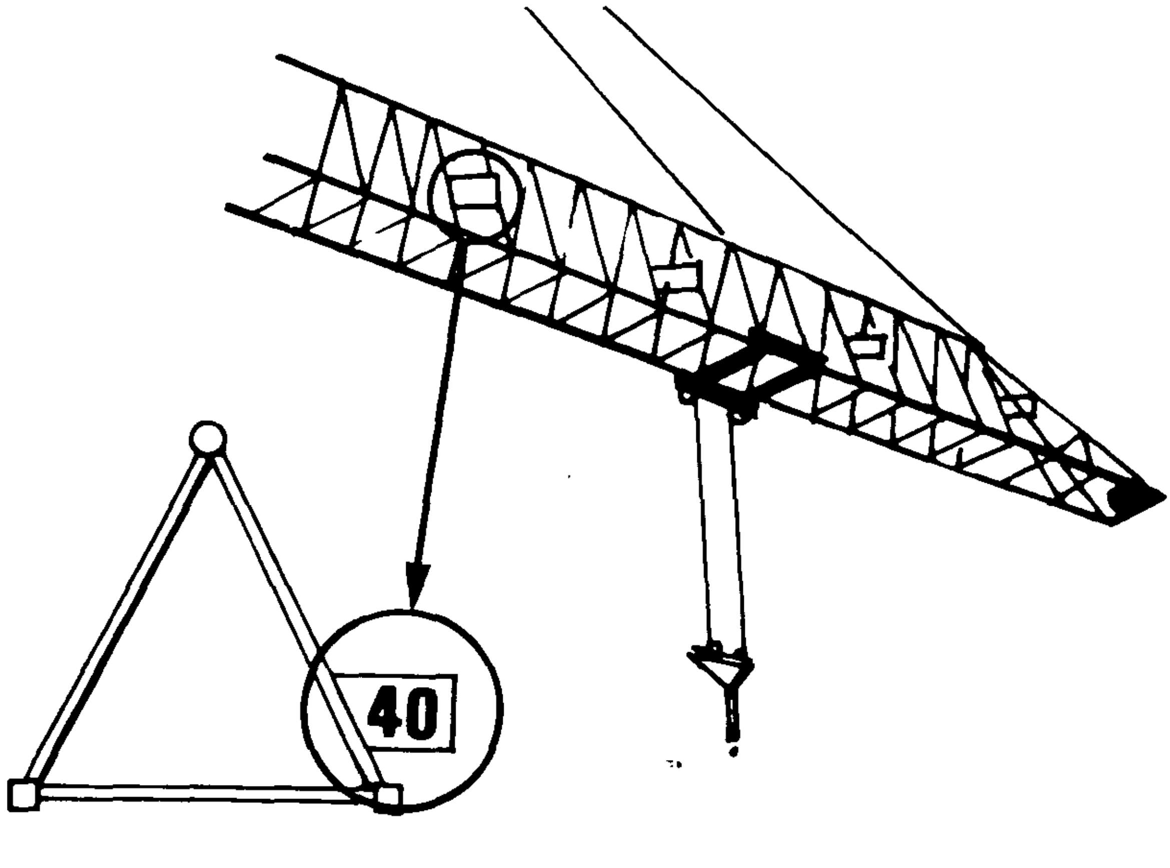


PROVIDE WORKING PLATFORMS ON THE TROLLEY.



PROVIDE WALKING PLATFORMS.

Fig. 5.48



TROLLEY RADIUS MARKERS SHOULD BE INSTALLED ON THE JIB.

Fig. 5.49

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DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

REQUISITOS MÍNIMOS DE CONOCIMIENTO
DE LAS ESTRUCTURAS DE ACERO
PREVIOS AL MONTAJE

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

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SECTION 1.2 STEEL FABRICATION AND CONSTRUCTION

1.2.1 Flowchart of Fabrication of Steel Structures

For the fabrication of steel structures, the flow of work at the factory is shown in summary form in Fig. 1-1.

1.2.2 Material Inspection of Steel

To confirm that the steel to be used matches the design and is suitable for the welding method specified, inspection of mill sheet is carried out. Test pieces may be cut from the end of material to be used in fabrication, these test pieces being then subjected to tensile testing, bending test, Charpy test, high-tension bolt testing, etc. In some cases, the test pieces may be cut and processed in the inspector's presence, to allow later verification. (Figs. 1-2 and 3)

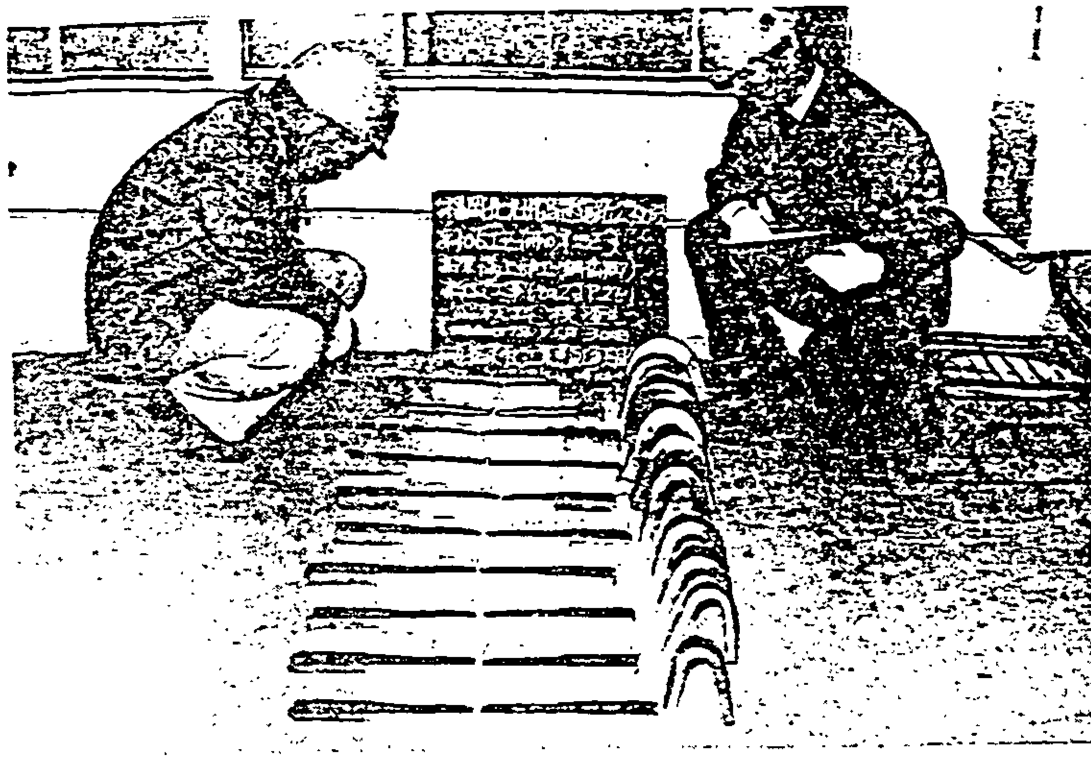


Fig. 1-2

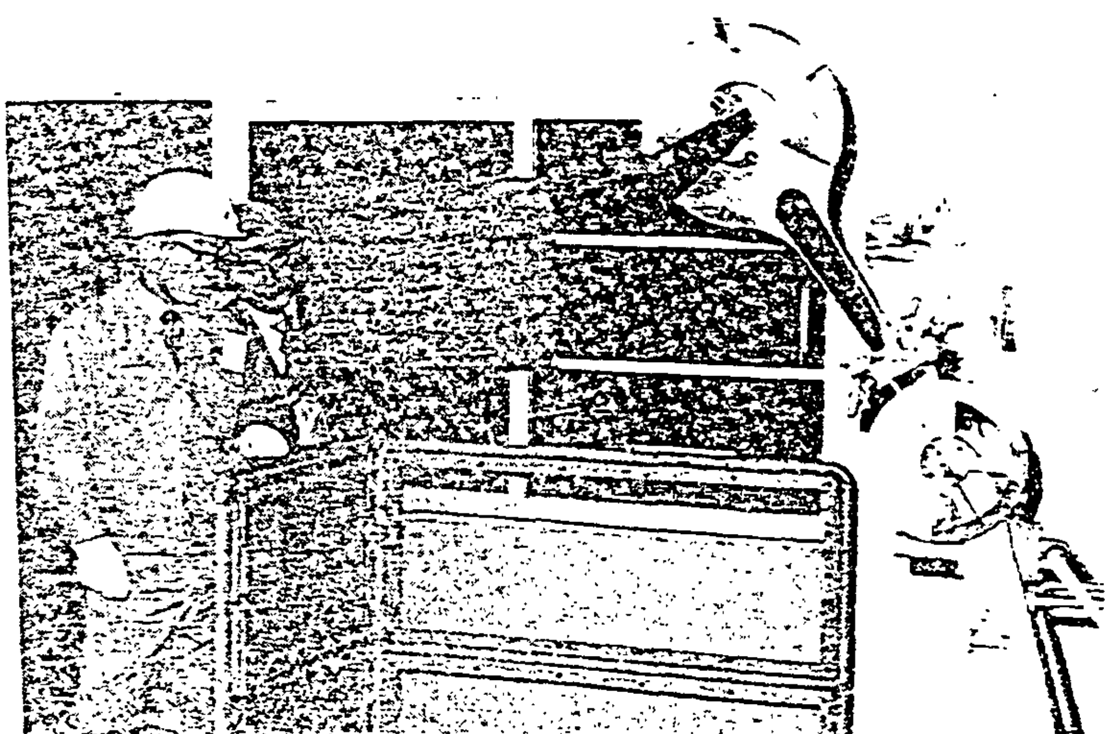
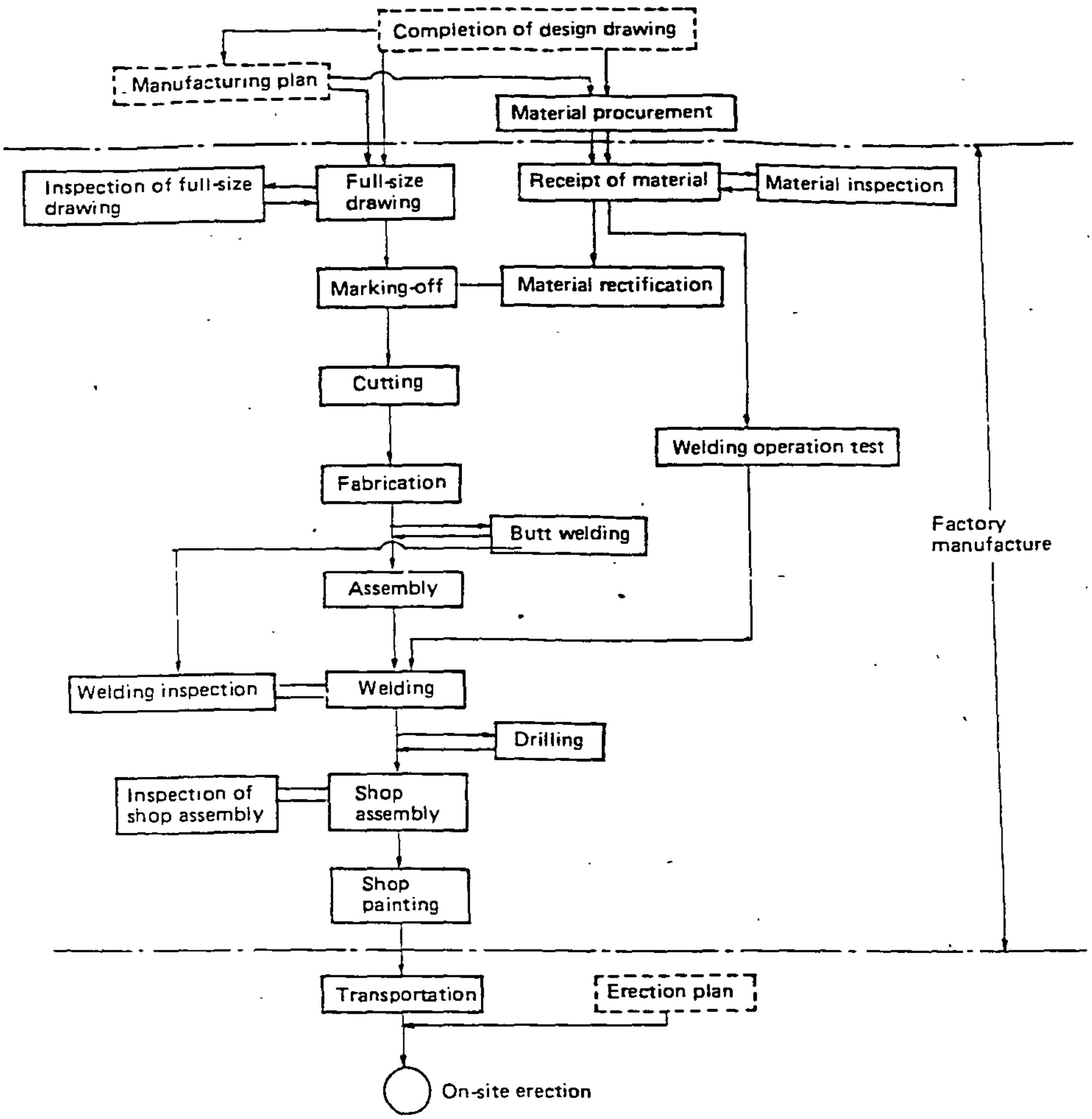


Fig. 1-3

Fig. 1-1. Flowchart of factory manufacture



1.2.3 Full-size Drawing

Before a full-size drawing is prepared, the fabricating controller makes a careful study of the design drawing and writes a set of fabrication instructions for use by the draftsman who will lay out the full-size drawing.

A first-grade steel tape conforming to JIS B7512, with 5 kg added tension, is used to prepare a flexible French curve, templates and other aids employed in preparing the full-size drawing. (Fig. 1-4)

Fig. 1-4

1.2.4 Marking-off

Marking-off on steel is done with the curve and templates mentioned above. Before this work is begun, the steel to be used is checked for material quality, size, surface defects and strains. In marking-off, the cutting line, beveling shape, finishing line, fitting position of members, boring position, diameter of bore and other data are indicated on the steel, along with shrinkage by welding and estimated size of gas-cutting allowance. (Fig. 1-5)

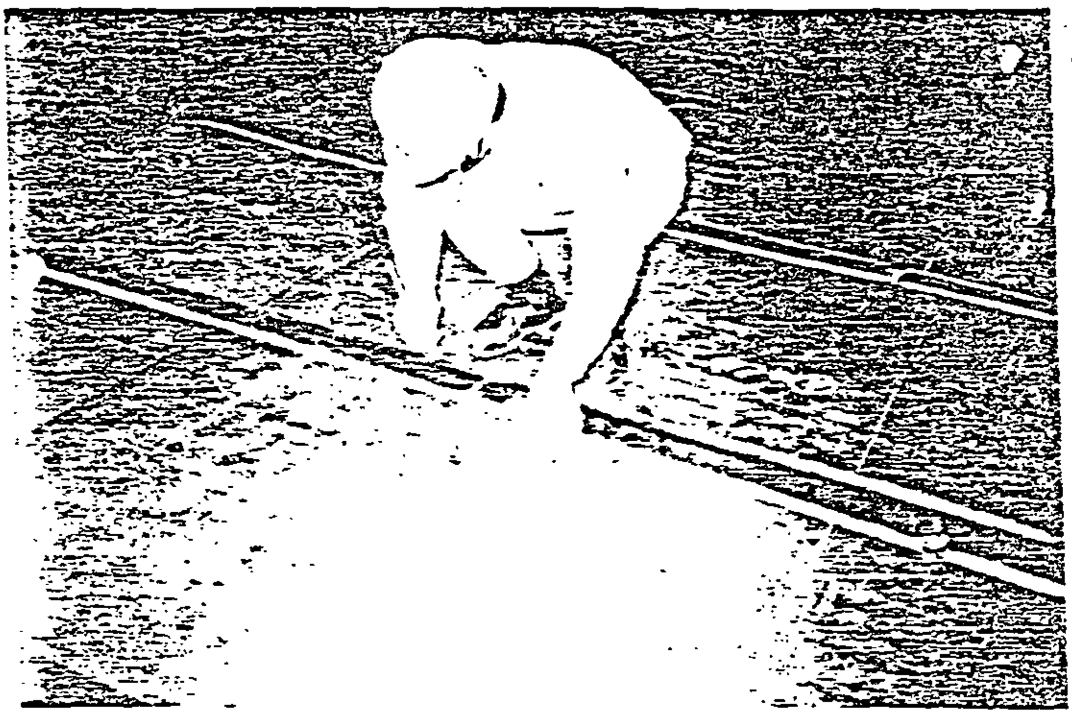


Fig. 1-5

1.2.5 Cutting

1) Method of cutting and cautions

Three methods of cutting are in use:

Mechanical cutting	—	cold saw, band saw, etc. (shapes, pipe, round bars)
Oxygen cutting	—	oxygen-acetylene, oxygen-propane, oxygen-propylene (plates, shapes, pipe)
Fusion cutting	—	arc cutting, plasma cutting, laser cutting (stainless steel, plates)

For cutting steel plates and shapes, oxygen-acetylene and oxygen-propane cutting are widely used.

To avoid strains in the material caused by cutting (mostly by gas-cutting), several methods can be used:

- Binding of cut parts to restrict movement
- Appropriate choice of cutting order and cutting direction during layer of pattern on the plate
- Simultaneous cutting, with multiple cutters
- Leaving for last the cutting of parts susceptible to deformation, which are cut after cooling

2) Quality of gas cutting

(Table 1-7)(Fig. 1-6)

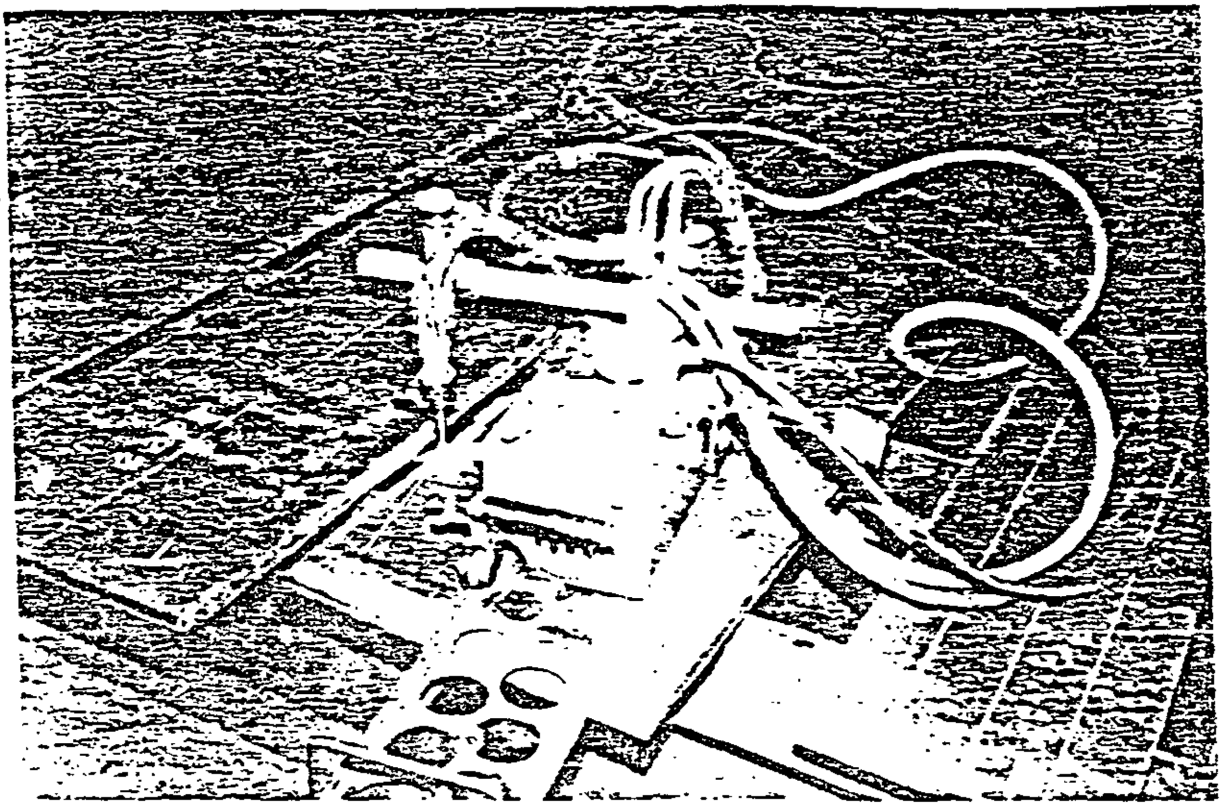


Fig. 1-6

Table 1-7. Quality of gas-cutting plane

Types of members	Main members	Secondary members
Surface roughness (1)	50S	100S
Notch depth (2)	Not permitted	1 mm
Slug	Dotted stickings of lumpish slug can easily be removed without leaving any trace.	
Upper edge melt	A slightly roundish, yet smooth melt	

Notes: (1) Surface roughness means the roughness of surface specified in JIS B 601-1955. 50S means a surface roughness of 50/-1000 mm.

(2) Notch depth means the depth from the upper edge of notch to the trough.

1.2.6 Press Work

To correct angular deformation of the flange due to fillet welding, reverse strain is applied and cold bending is carried out by press work to restore the normal shape after welding. Since angular deformation occurs during butt welding as well, strain is corrected after welding by press work.

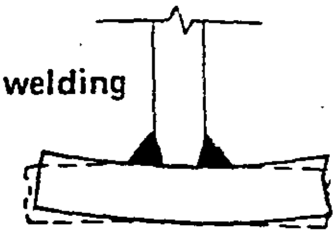
In the bending of a curved I-beam flange, where the width of the flange is not very large, horizontal bending by cold-working is carried out by press work.

Fig. 1-7 shows examples of the amount of deformation encountered during fillet welding.

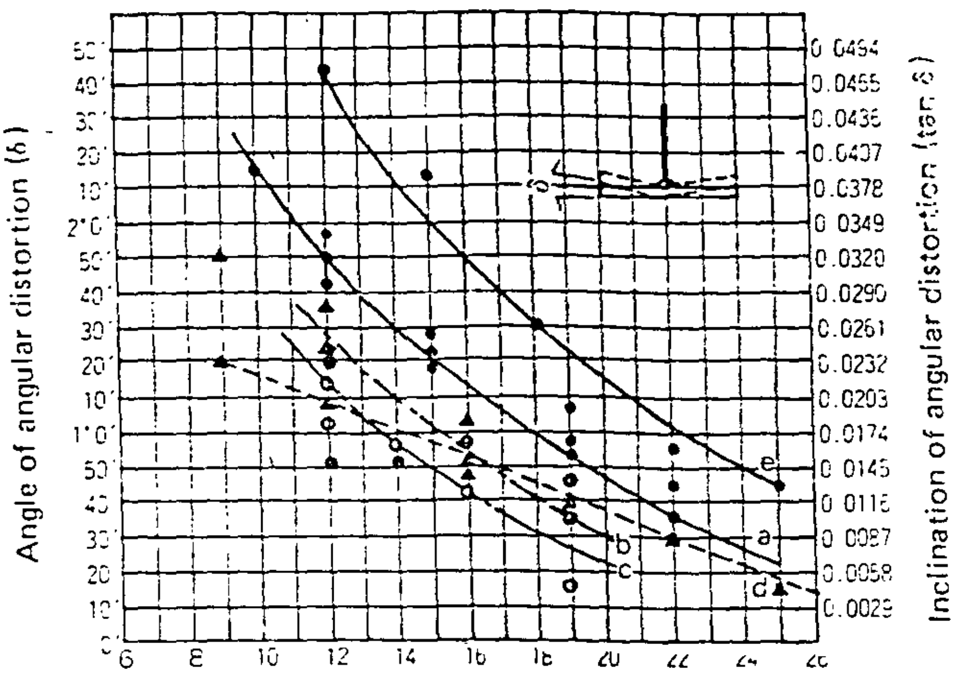
The extent of cold bending of steel plate is determined by the mechanical properties of the material and plate thickness. For structural steel, the inside radius is specified as more than 15 times the plate thickness.

Butt welding

Fillet welding



Angular distortion measurement chart



Symbol	Leg length	Number of layers (mm)	Electrode used
a	6	2	Ilmenite type
b	6	1	Ilmenite type
c	6	1	Deep-penetration type 4 mm ϕ
d	6	1	Union melt type
e	9	2	Ilmenite type

Fig. 1-7. Examples of angular distortion

Table 1-8. Welding methods for bridge fabrication

- Welding methods
  - Manual welding
  - Automatic welding
    - Automatic flat welding
      - Single-electrode submerged arc-welding
      - Multi-electrode submerged arc-welding
      - Various high-efficiency submerged arc-welding (Cut-wire, KKX, KIS, FIN and others)
      - CO_2-gas automatic welding
    - Automatic vertical welding
      - Electro-slag welding
      - Electro-gas welding
    - Automatic horizontal welding
      - Horizontal submerged arc-welding
      - Horizontal CO_2-gas arc-welding
  - Semi-automatic welding
    - CO_2-gas arc-welding
    - Non-gas arc-welding
    - Submerged arc-welding
  - High-efficiency welding using coated electrodes
    - Gravity welding
    - Inclination welding with electrode set at lower angle
    - Firecracker welding
  - Stud welding
    - Nelson-type welding
    - Philips-type welding

1.2.7 Welding

1) Method of welding

The various methods of welding listed in Table 1-8 are used in the fabrication of steel structures such as bridges. In general, automatic welding is most efficient, but it is not yet possible with all welding methods. In some applications, in fact, use of automatic welding may lower productivity.

(1) Characteristics of submerged arc-welding

Arc-welding is carried out in powdered flux. Because the welding is shielded by granular flux and the gas produced, very large currents can be used and high efficiency attained, as well as excellent quality of deposited metal and good bead appearance. Since there is little loss of thermal energy, weld penetration is large and thus the amount of deposited metal can be made smaller.

Optimum welding conditions are necessary, however, and grave defects can occur if these conditions are not met. Precise beveling preparation and checking of welding conditions are particularly important.

(2) Welding conditions

(2)-1 X-shape bevel welding

To join plates, welding is carried out after the plates are beveled. An example is shown in Table 1-9.

(2)-2 Horizontal fillet weld

When horizontal fillet welds are made by submerged arc-welding, proper weld position and proper angle of holding the wire are important. An example of these conditions is given in Table 1-10 and Fig. 1-8.

Table 1-10. Horizontal fillet welding

Leg length mm	Electric current A	Voltage V	Welding speed cm/min	Wire diameter mm	Wire melting amount kg/m
4	350 ~ 375	25	71 ~ 140	3.2	0.09
5	400 ~ 550	26 ~ 28	56 ~ 107	3.2 ~ 4.0	0.13
6	450 ~ 625	26 ~ 30	51 ~ 91	3.2 ~ 4.0	0.21
8	500 ~ 700	27 ~ 33	46 ~ 66	4.0	0.30

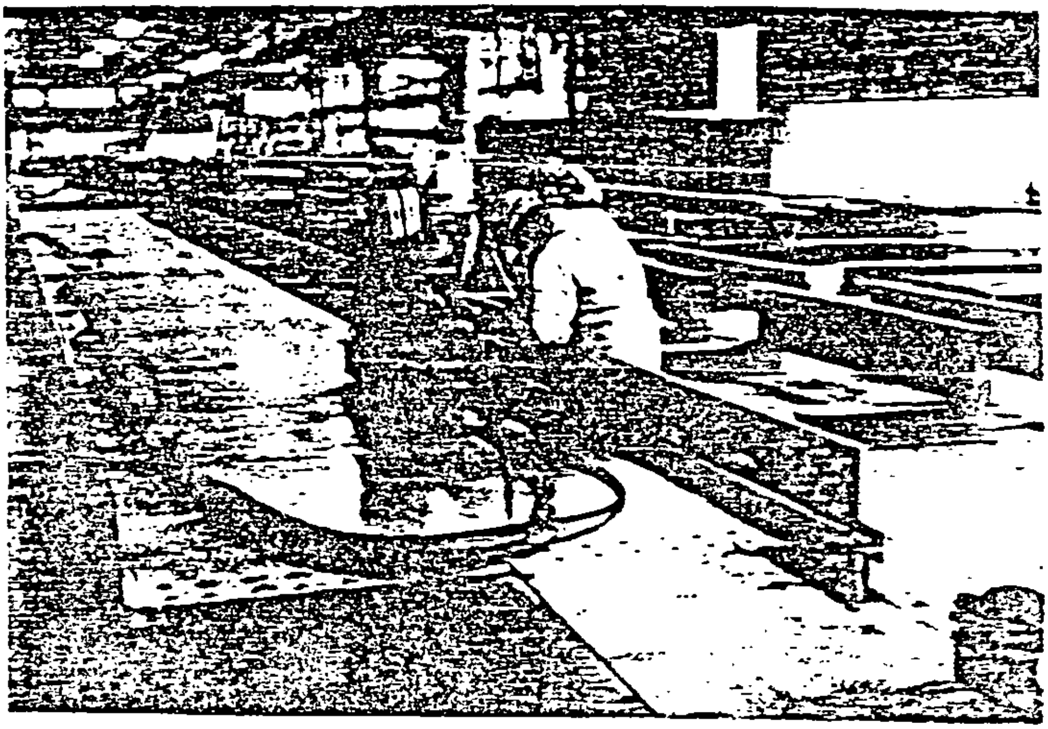


Fig. 1-8

2) Preheating

The main objectives of preheating are to prevent hardening and cracking of the welded part, and to remove moisture and inflammable substances from the beveled face prior to welding. Preheating reduces the rate of cooling of

Table 1-9. X-shape both-side single-layer butt welding

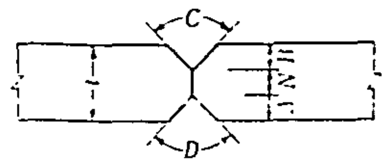


Plate thickness (t) mm	Finish weld						Root face (N) mm	Back run						Melting amount kg/m
	Groove depth (B) mm	Groove angle (C)	Electric current A	Voltage V	Welding speed cm/min	Electrode diameter mm		Groove depth (A) mm	Groove angle (D)	Electric current A	Voltage V	Welding speed cm/min	Electrode diameter mm	
19	9	60	1,000	36	41	5.6	6	4	60	700	35	56	5.6	–
"	6	90	1,150	35	33	6.4	8	5	90	850	33	41	6.4	1.24
25	9	60	1,050	36	30	5.6	7	9	60	900	35	36	5.6	–
"	10	90	1,300	36	28	6.4	8	7	90	1,000	34	38	6.4	1.64
32	16	60	1,100	37	19	5.6	4	12	60	1,000	35	30	5.6	–
"	14	70	1,450	36	25	7.9	9	9	60	1,100	35	33	6.4	2.14
38	19	60	1,100	37	16	5.6	4	15	60	1,050	36	23	5.6	–
"	16	70	1,600	37	23	7.9	11	11	60	1,300	35	25	6.4	2.92

the heat-affected zone, which prevents hardening and increases ductility. In particular, the cooling time below 300°C is extended. This promotes the diffusion and escape of hydrogen from the welded metal, thus preventing cold cracking. (Table 1-11) In the case of submerged arc-welding, preheating is often omitted due to the large amount of heat input with that welding method.

3) Inspection of welding (Table 1-12)

An outline of completion tests is given below.

Table 1-12. Inspection of welding

- Weld test method
  - Process tests — Inspection before and during welding (Inspection of fabrication planning, welding tests, welding operations and the like)
  - Completion tests
    - Appearance test of weld bead (Including appearance of stud weld)
    - Radiographic test (JIS Z3104)
    - Ultrasonic manual testing (JIS Z3060)
    - Magnetic flaw detecting test
    - Permeation flaw detecting test

(1) Appearance test of weld bead (Inspection Standards for Weld Bead—Draft, from *Handbook of Steel Road Bridge Execution*)

(1)-1 Inspection of welding cracks

There should be no cracks on or around the weld bead under any circumstances. Microscopic testing of cracks shall be carried out, and in case of doubt, magnetic flaw detection or permeating liquor flaw detection may be applied.

(1)-2 Weld reinforcement in groove welding

In the case of groove welding, weld reinforcement within a range given in the following table may be left as le 1-11. Standard preheating temperatures

Classification of standards		Type of steel: JIS	Type of steel: ASTM	Plate thickness (mm)	Welding methods: Shielded metal arc-welding using electrodes other than low-hydrogen type	Welding methods: Shielded metal arc-welding using low-hydrogen type electrodes
I (Mainly 40-kg steels)	Steel highway bridge	SS41 SM41		t < 25 25 ⩽ t < 38 38 ⩽ t ⩽ 50	No preheating 40 ~ 60° C —*	No preheating No preheating 40 ~ 60° C
	AWS AISS		A36, A53 Gr B A375 A500, A501 A570 Gr D E	t ⩽ 19 19 < t ⩽ 38 38 < t ⩽ 63.5 63.5 < t	No preheating ⩾ 65° C ⩾ 110° C ⩾ 150° C	No preheating ⩾ 20° C ⩾ 65° C ⩾ 110° C
II (Mainly 50-kg steels)	Steel highway bridge	SM50 SMA41		t < 25 25 ⩽ t < 38 38 ⩽ t ⩽ 50	— — —	No preheating 40 ~ 60° C 80 ~ 100° C
	AWS AISC		A242, A441 A572 Gr 42, 45, 50 A588	t ⩽ 19 19 < t ⩽ 38 38 < t ⩽ 63.5 63.5 < t	— — — —	No preheating ⩾ 20° C ⩾ 65° C ⩾ 110° C
	AASHO		A36, A242 A441, A572 A588	t ⩽ 19 19 < t ⩽ 38 38 < t ⩽ 63.5 63.5 < t	— — — —	⩾ 10° C ⩾ 20° C ⩾ 65° C ⩾ 110° C
III (Mainly 55-kg and 60-kg steels)	Steel highway bridge	SM50Y SM53 SMA50 SM58 SM58		t < 25 25 ⩽ t < 38 38 ⩽ t ⩽ 50	— — —	40 ~ 60° C 80 ~ 100° C 80 ~ 100° C
	AWS AISC		A572 Gr 55, 60, 65	t ⩽ 19 19 < t ⩽ 38 38 < t ⩽ 63.5 63.5 < t	— — — —	⩾ 20° C ⩾ 65° C ⩾ 110° C ⩾ 150° C

* Use of low-hydrogen type electrodes is regarded as standard.

welded. If weld reinforcement exceeds the values given in the table, the shape of the bead, especially that of the toe of the weld, must be smoothly finished.

Weld Reinforcement in Groove Welding

Width of bead (B)	Height of reinforcement (h)
B < 15 mm	$h \leqq 3$ mm
15 mm $\leqq$ B < 25 mm	$h \leqq 4$ mm
2.5 mm $\leqq$ B	$h \leqq \frac{4B}{25}$ mm

(1)-3 Pits in face of weld bead

With regard to butt joints of main materials, T joints comprising cross sections and corner joints, there should be no pits in the face of the bead. With regard to other fillet welding and partial-penetration groove welding, three pits per joint or up to three pits per meter of joint length are allowed. Where pit size is less than 1 mm, three pits are counted as one pit.

(1)-4 Unevenness of bead surface

Unevenness of bead surface is measured as the difference in height along a 25-mm length of bead. There should be no unevenness greater than 3 mm.

(1)-5 Undercut

The depth of undercut should not exceed the values given below.

Position of unoercut	Tolerance
Bead stop end part crossing perpedicularly the primary stress applied to main material	0.3 mm
Bead stop end part parallel to the primary stress applied to main material	0.5 mm
Bead stop end part of secondary material	0.8 mm

(1)-6 Overlap

There should be no overlap.

(1)-7 Size of fillet weld

Expansion of fillet weld and throat depth should not be less than the specified fillet size and equivalent throat depth. But on one weld line, excluding 50 mm at each end, a tolerance of -1.0 mm is allowed within a range of 10% of the length.

(2) Radiographic testing

The readiness with which x-rays can pass through materials is utilized for material inspection by radiographic testing.

In this method, x-rays are beamed through the material and onto industrial x-ray film on the opposite side of the material being inspected. The film is sensitized in varying degrees, according to the amount of radiation that passes through each part of the material and this reveals the internal state of the material (Fig. 1-9).

The negative film is usually not printed out, but is checked directly. Internal flaws in the material show up as darker portions on the film (Fig. 1-10).

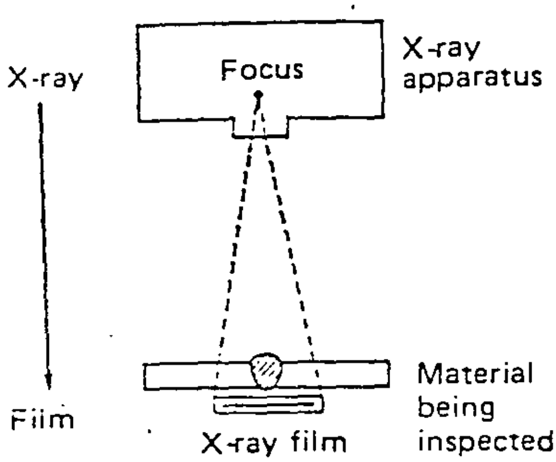


Fig. 1-9. Principle of X-ray permeation inspection

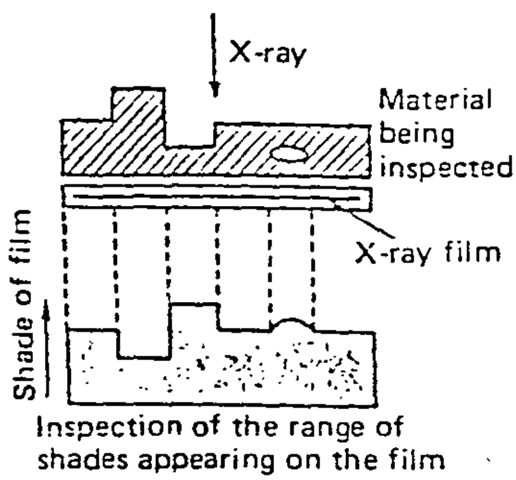


Fig. 1-10. Principle of flaw detection

There are three methods of radiographic testing in use: direct photographing, indirect photographing and permeation. Direct photographing, the most popular method, provides a nearly full-size permeated picture of a test piece, thus allowing the highest level of flaw detection.

For more details, refer to JIS Z3104 specifications.

(3) Ultrasonic testing

An ultrasonic wave is the vibration of particles, like an ordinary sound wave, but having a frequency of vibration greater than the limit (about 20 kHz) of human hearing ability. Ultrasonic waves can be utilized as a means of flaw detection.

There are several methods of ultrasonic testing, differing in principle of flaw detection or indication of flaw patterns used, ultrasonic frequency used and the like. The method most widely employed is the pulse echo technique, in which an ultrasonic pulse is directed into a test piece from a probe that can both send and receive sonic waves. Internal defects, if present, reflect the ultrasonic waves that reach them, and these reflected waves are detected by the probe.

Table 1-13 shows the ultrasonic flaw detection methods and their major applications. Vertical flaw detection, shown in Fig. 1-11(a), features easy detection of defects and their depth since a large bottom echo is usually produced in addition to the defect echo. Oblique flaw detection, shown in Fig. 1-11(b), is said to be rather difficult to apply. This is because the ultrasonic wave is directed obliquely into the test piece and no bottom echo is produced. The position of defects is computed by a trigonometric function, using both the angle of refraction predetermined for the test piece and the distance of sound propagation (distance of beam) to the flaws detected by a cathode-ray tube, as shown in Fig. 1-12.

For more details, refer to JIS Z3060 specifications.

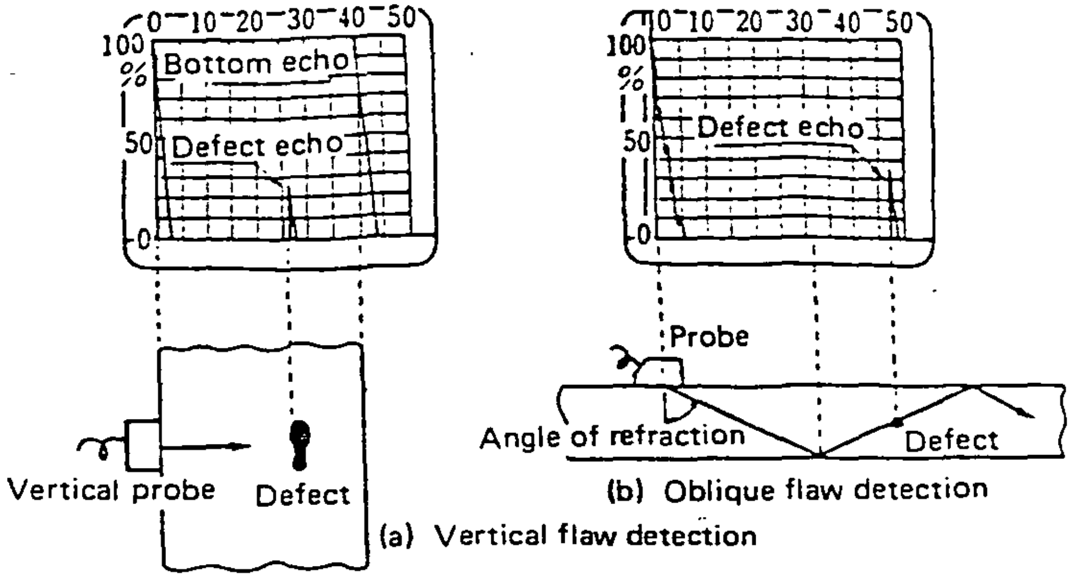


Fig. 1-11. Vertical and oblique flaw detection

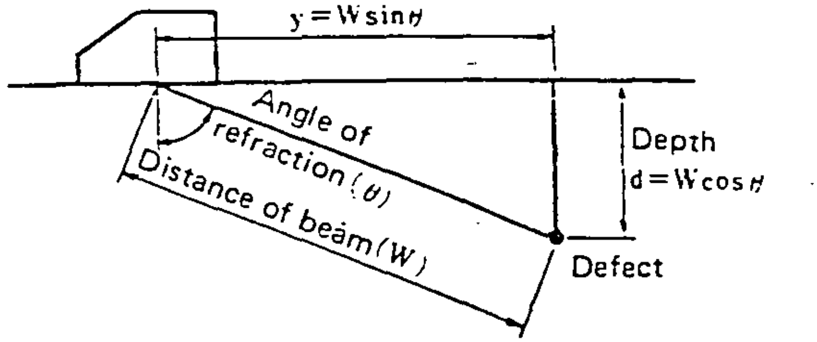


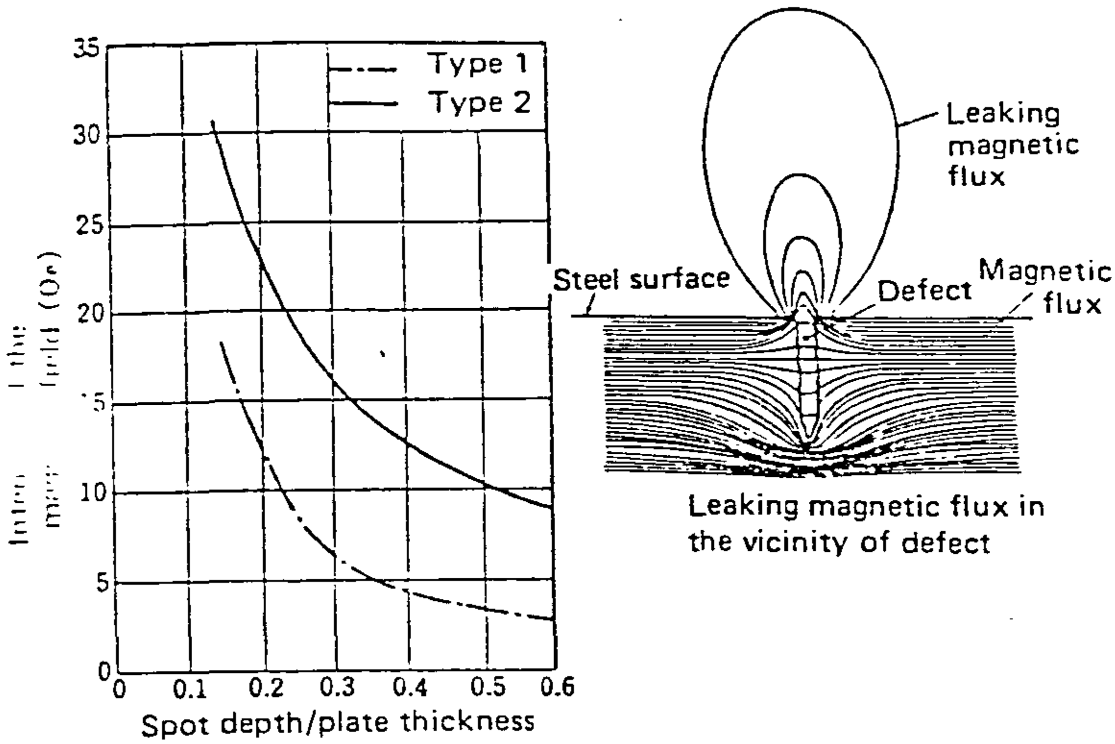
Fig. 1-12. Method of determining the position of defect in oblique flaw detection

Table 1-13. Methods of ultrasonic testing and their applications

Flaw detecting methods	Type of waves used	Application
Vertical detection	Vertical wave	Plates, forgings, casting sections, thickness measurement
Oblique detection	Lateral wave	Welding, pipe and tube forgings
Flat wave detection	Flat wave	Thin-gauge sheet
Surface wave detection	Surface wave	Surface defects

(4) Magnetic flaw detection test

When a material that is attracted by a magnet is magnetized, any defects such as cracks that it may contain show a high resistance to magnetic flux. Hence the magnetic flux detours around the defect and part of it leaks into the air. These leaks attract iron powder, which may thus be used to locate the defect visually. (Fig. 1-13) This test is highly sensitive to surface defects, but cannot easily locate internal defects.



The intensity of the magnetic field acting when clear magnetic powder patterns begin to develop on an A-shaped standard test piece.

Fig. 1-13. Magnetic flaw detection test

(.) Permeation flaw detection test

Permeation flaw detection testing is carried out by first applying a permeating liquor to the face of the test piece. It soaks into the defect. The liquor is then removed from the test-piece surface and a developing agent consisting of a fine white powder is lightly coated onto the surface. It draws out the residual liquor in the defect, allowing an enlarged pattern of the defect to be observed. Compared with magnetic flaw detection, permeation flaw detection has these advantages:

- ☐ Flaws can be detected on non-magnetic materials.
- ☐ Results are unaffected by shape of test piece or direction of defect.

But permeation flaw detection also has drawbacks:

- ☐ Only flaws that have an opening to the surface and a gap can be detected.
- ☐ Surface roughness of the test piece lowers the sensitivity of flaw detection.

(Table 1-14)

Table 1-14. Classification of test methods by type of permeation liquor

Description	Method
Fluorescent light permeation flaw Detection test	Method using washable fluorescent permeation liquor
	Method using post-emulsifying fluorescent light permeating liquor
	Method using solvent-removable fluorescent light permeating liquor
Dyeing color permeation flaw detection test	Method using washable dyeing color permeating liquor
	Method using post-emulsifying dyeing color permeating liquor
	Method using solvent-removable dyeing color permeating liquor

1.2.8 Machining Finish

For use in such structures as bridges, the faces of structural member edges are generally machined to required dimensional precision by exact gas-cutting. If higher precision is required or if shrinkage from welding cannot be assumed, mechanical planing is generally used. In most cases, a rotary-edge planer or edge planer is employed for such finishing.

1.2.9 Drilling

Most field joints are made with high-tension bolts, and the precision of bolt-hole drilling determines the precision of field-joining work.

1) Timing of drilling

☐ Drilling of framework members before assembly

☐ Drilling of framework members after assembly and welding

2) Size of drilling

☐ Full-size drilling

☐ Sub-size drilling and reaming at time of shop assembly

3) Method of drilling

☐ Drill is aligned on center punch of marking-off.

☐ Bushing is attached to steel template and punching is done thereon.

☐ Drilling is done by numerical control system without marking-off.

☐ Only partial stitching hole is made and drilled through from the plate on the front.
(Fig. 1-14)

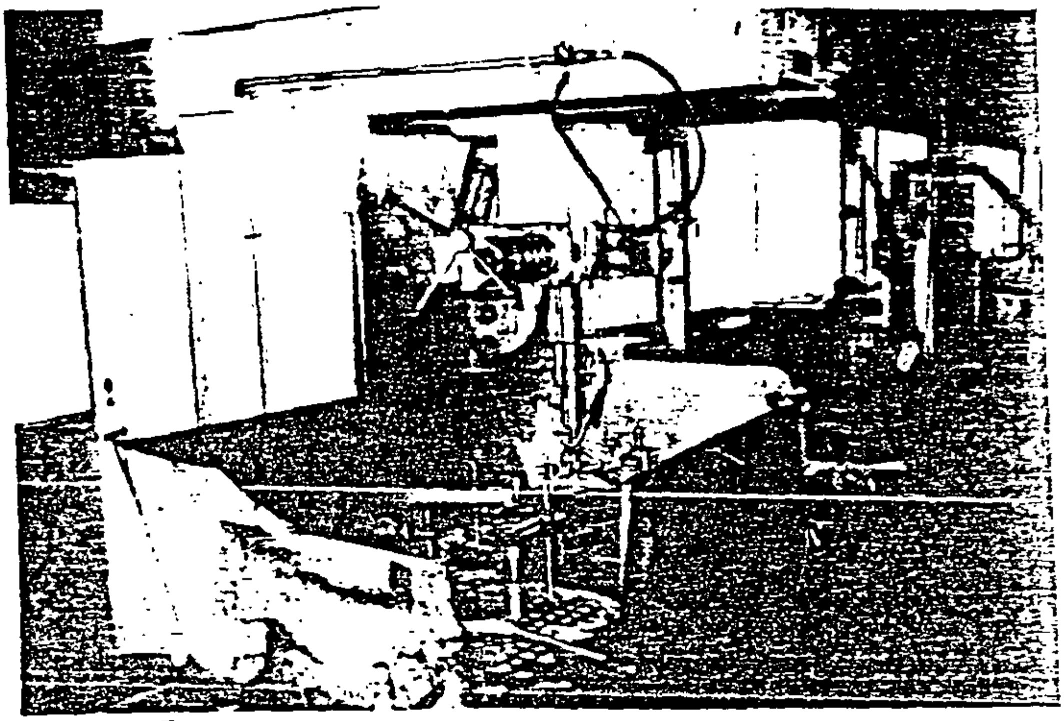


Fig. 1-14

Drilling accuracy for a road bridge is as follows:

☐ Diameter of high-tension bolt d $\leqq$ +2.5 mm

☐ Tolerance of hole d $\leqq$ +0.5 mm

(But for 20% of a set of 1 bolt, tolerance $\leqq$ +1.0 mm is allowed.)

1.2.10 Shop Assembly

After fabrication is completed, the members of the framework are assembled, either for the whole structure or parts of it, in the factory yard.

1) Objectives of inspection of shop assembly

- ☐ To confirm that framework members are correctly fabricated
- ☐ To confirm that members join properly and that the whole assembly is of specified shape and size
- ☐ To measure strain as required

2) Methods of shop assembly

(Table 1-15)

Table 1-15. Shop assembly system

Shop assembly	Range of shop assembly	Whole shop assembly
		Divided shop assembly
		Partial shop assembly
	Method of shop assembly	Normal (vertical) assembly
		Reversed assembly
		Lateral assembly
		Flat assembly

3) **Main points of shop assembly (general)**

- ☐ Shop assembly must be done on a rigid stand at a certain elevation from the ground to avoid stresses.
- ☐ For special holes, an adequate number of bolts and driftpins must be used. It is usually sufficient to join about 30% of the total number of bolt-holes, but for structures with a higher stress-to-deadweight ratio, such as steel-plate floor girders, it is desirable to join more than 50% of holes during shop assembly.
- ☐ The shop assembly site must be organized so as to obtain exact measurements.
- ☐ The bearing point of a bridge body is where the stiffener is applied, and it must be supported by the face. If bearing must occur at a point where no stiffener is attached, some protective device must be applied.
- ☐ Camber in the non-stressed conditions of shop fabrication must be considered during shop assembly. The bearing stand must be designed to bear the load evenly, without excessive local loading.
- ☐ If it is necessary to measure deflection due to the steel's dead weight by removing the middle support table, load-bearing capacity of the support table of bearing members must be fully checked to prevent uneven settling. Where a girder bridge becomes level after placement of dead load and camber is comparatively large, it is better to measure deflection by dead weight of steel in advance by removing the middle supporting stand after shop assembly under non-stressed conditions.

4) Inspection of shop assembly

Since inspection of shop assembly is the final inspection conducted at the completion of shop fabrication, it must be carried out only after all shop assembly work has been completed. Joint inspection with the customer is then carried out, preferably with the participation of the customer's representative in charge of the contract. (Fig. 1-15) (Table 1-16)

Confirmation of the following items may be required during an inspection of shop assembly.

☐ Dimensional inspection
- *Total length and span length
- *Height of girders, truss, etc.
- *Center distance of girders, truss, etc.
- *Length of frame, accuracy of meeting angle of girder, truss, etc.
- *Fabrication camber (no loaded camber)
- *Verticality of girders and truss
- *Length, cross section and bending of framework members
- *Rectilinearity of web and flange
- *Flatness of plate
- *Others

☐ Inspection of joining surfaces
- *Condition of drilling (misalignment of holes, hole diameter, conditions around hole)
- *Irregularities, gaps, etc. on field-joining surfaces
- *Shape and size of bevel for field-welding

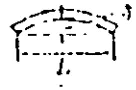
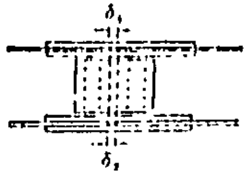
☐ Inspection of appearance

☐ Inspection of stud dowel

☐ Radiation testing

☐ Inspection of connection part

☐ Inspection of parts
- *expansion joint
- *balustrade
- *bearing
- *water drainage system
- *nameplate

	Item	Plate girder, box girder, steel floor p	Truss, arch, rigid frame	Remarks
ccuracy of members	Height of members	$H \leq 2$ (m) ± 4 (cm) $H > 2$ (m) Addition of 1 mm for every 2 m or fraction thereof	$H \leq 1$ (m) ± 2 (mm) $H > 1$ (m) ± 3 (mm)	The relative errors for field joints shall be half the values shown at left.
	Flange width	$W \leq 1$ (m) ± 2 (mm) $W > 1$ (m) ± 4 (mm)	The same as at left	
	Length of members	$L \leq 10$ (m) ± 3 (mm) $L > 10$ (m) ± 4 (mm)	$L \leq 10$ (m) ± 2 (mm) $L > 10$ (m) ± 3 (mm)	
	Curvature of compression members	—	$\delta < \frac{1}{1000}$	
	Flatness of plate	Web plate for girder: h/250 (mm) h: Height of web plate (mm) Box girder flange and floor plate decking: W/150 (mm) W: Rib spacing or web plate spacing Flange squareness: 1/100	Flange and web plate: W/150 (mm) W: Weld line space	The relative errors for field joints shall be half the values shown at left.
ccuracy in hop assembly	Total length and span	$\pm (10 + \frac{L}{10})$ (mm) L: Total length or span (m)		
	Center-to-center distance of girder and truss	$\pm [4 + (B - 2) \times 0.5]$ (mm) B: Designed center-to-center distance		
	Clearance between field joints	$\delta \leq 3$ (mm) δ: δ_1 or δ_2, whichever larger, in the figure		
	Camber	$L \leq 20$: ± 5 mm $20 < L \leq 40$: $-5 \sim +10$ mm $40 < L \geqq 80$: $-5 \sim +15$ mm $80 < L \leq 200$: $-5 \sim +25$ mm L: Span length (m)		
	Expansion joint	$L \leq 10$: Effective width $+ 10 \sim 5$ (mm) $L < 10$: Effective width $+ 10 + (L - 10) \times 0.5$ (mm) L : Span (m) Difference in height of jointly used expansion device : ± 4 (mm) Finger difference: Not more than 2 (mm)		

1.2.11 Painting

The thickness of paint applied is selected so as to avoid coating film defects such as shrinkage, drip, unevenness, transparency and the like. The quantity of paint required for such coverage is specified as the standard amount of paint.

1) **Preparation of surface for painting**
(Tables 1-17 and 18)(Fig. 1-16)

2) **Standard amount of paint**
(Tables 1-19 and 20)

Table 1-17. Surface preparation and methods

Degree of surface preparation		Methods
Cleanliness Class 1 (Class 1 rust removing)	Remove mill scale, rust and paint coat sufficiently to prepare a clean metal surface.	Blasting
Cleanliness Class 2 (Class 2 rust removing)	Remove rust and paint coat to expose the steel surface. However, rust and paint coat remain in recessed or narrow parts.	Manual tools and motor-driven tools such as a disc sander.
Cleanliness Class 3 (Class 3 rust removing)	Remove rust and deteriorated paint coat to expose the steel surface, but leave effective paint coat.	Ditto
Cleanliness Class 4 (Class 4 rust removing)	Remove powdered and adhering substances, but leave effective paint coat.	Ditto

Table 1-18. Comparison of blasting methods

Blasting method / Item	Base sheet blasting	Product blasting
Efficiency and cost	Highly efficient and less expensive since an automatic blasting machine is used.	Low in efficiency and high in cost since this method is mainly performed manually, though it can be partially automated.
Workability	Good, since it is mechanized.	Care should be taken about safety and hygiene of workers.
Relation with welding, etc.	1) Base primer may have effects on welding. 2) By thermal cutting, welding or stress relieving, base prim..r may be burnt and rust develops. Or alkalis remaining at welds may adversely affect the paint coat. So the surface must be prepared again to obtain cleanliness Class 2.	None

Blasting methods are classified by the type of material used for blasting as follows.

(1) Shot blasting (steel balls)

(2) Grit blasting (steel grit)

(3) Cut wire blasting (cut wire)

(4) Sand blasting (river sand, silica sand, slag)

Currently, shot blasting is more often used than sand blasting.

Table 1-19. Standard amounts of paint for use in general external painting

Painting process		Pretreatment: Surface preparation	Pretreatment: Base primer	Inter-val	Shop painting: Undercoat	Inter-val	Shop painting: Undercoat	Inter-val	Shop painting: Undercoat	Inter-val	Field painting: Undercoat	Inter-val	Field painting: Middle coat	Inter-val	Field painting: Top coat
A	1	Before base sheet blasting, apply base primer. For product blasting, primer may be dispensed with.	Long-exposure type etching primer 130 g/m²	1 day ~ 3 months	Lead base rust-preventive paint Class 1 250 g/m²	2 ~ 10 days	Lead-base rust-preventive paint Class 2 220 g/m²			Within 6 months	—	—	Long-oil phthalic resin middle coat paint 120 g/m²	1 ~ 10 days	Long-oil phthalic resin top coat paint 110 g/m²
	2				Lead-base rust-preventive paint Class 1 170 g/m²		Red lead rust-preventive paint Class 1 170 g/m²			Within 12 months	Red lead rust-preventive paint Class 1 140 g/m²	2 ~ 10 days			
	3				Lead-base rust-preventive paint Class 1 170 g/m²	2 ~ 10 days	Lead-base rust-preventive paint Class 1 170 g/m²	2 ~ 10 days	Phenol MIO paint 300 g/m²		—	—	Long-oil phthalic resin middle coat paint 120 g/m²		
B	1												Chlorinated rubber undercoat paint 170 g/m²	1 ~ 10 days	Chlorinated rubber top coat paint 150 g/m²
	2		Zinc-rich primer 200 g/m²	2 days ~ 6 months	Zinc-rich primer 200 g/m²	2 ~ 10 days	Chlorinated rubber undercoat paint 250 g/m²	2 ~ 10 days	Chlorinated rubber undercoat paint 250 g/m²						

1. Application
   A: Areas with no strong corrosive factors in environment.
   B: Areas with strong corrosive factors in environment such as coastal areas and heavy and chemical industrial districts.
2. Steps to be taken when the specified intervals of shop painting and field painting are exceeded
   (A−1) (within 12 months): field painting is changed to (A−2).
   (A−1)
   (A−3) (in excess of 12 months) : field painting is performed according to the method of recoating.
   (B−1)
   (B−2)

Remarks:
Amounts of paint used:
Shop painting: denotes spray painting.
Field painting: denotes brush painting.

2. MIO paint coat has a coarse surface and absorbs repeatedly applied paint, and its thickness will become somewhat thinner.
3. The shortest values of recoating intervals indicate those at an air temperature of 20°C. If the temperature is low, it is necessary to check the degree of drying of paint by the feel of a finger and then to decide the recoating intervals.
4. The recoating intervals may be longer than the longest values specified in the table, but it is desirable to recoat within the specified intervals as much as practicable.
5. Red lead rust-preventive paint should be Class 1, which is high in rustproofing and adhering capabilities, for the first coat and Class 2, which is quick to dry, for the second coat.
6. When primer is used in (B−2), product blasting should be performed for parts damaged durii processing.

Table 1-20. Standard amounts of paint for use in external painting processes for long-term rust prevention

Painting process		Pretreatment			Shop painting								Inter-val	Field painting		
		Surface preparation	Base primer	Inter-val	Undercoat	Inter-val	Undercoat	Inter-val	Undercoat	Inter-val	Undercoat			Middle coat	Inter-val	Top coat
C	1	For base sheet blasting, apply base primer. For product blasting, base primer may be dispensed with.	Zinc-rich primer 200 g/m²	2 days ~ 6 months	Thick-film type zinc-rich paint 700 g/m²	2 ~ 10 days	Mist coat 160 g/m²	1 day	Chlorinated rubber undercoat paint 250 g/m²	1 ~ 10 days	Chlorinated rubber undercoat paint 250 g/m²		Within 12 months	Chlorinated rubber middle coat paint 170 g/m²	1 ~ 10 days	Chlorinated rubber top coat paint 150 g/m²
	2						Mist coat 160 g/m²	1 day	Epoxy resin undercoat paint 250 g/m²	2 ~ 10 days	Epoxy MIO paint 300 g/m²			Middle coat paint for polyurethane resin paint 140 g/m²	1 ~ 10 days	Polyurethane resin top coat paint 120 g/m²
	3						Short-exposure type etching primer 130 g/m²	4 ~ 12 hours	Phenol zinc chromate undercoat paint 150 g/m²	1 ~ 10 days	Phenol MIO middle coat paint 300 g/m²			Chlorinated rubber middle coat paint 170 g/m²	1 ~ 10 days	Chlorinated rubber top coat paint 150 g/m²

Remarks:

Amounts of paint used: Shop painting denotes spray painting. Field painting denotes brush painting.

. Mist coat: Since thick-film type zinc-rich paint coat is porous a mist coat is applied over it to expel air from the paint coat and to prevent bubbles being produced during application of the next coat. A 50% dilute solution of paint for the next coat is used. Short-exposure type etching primer of (C–3) is used for the same purpose.

3. Epoxy resin paint is low in weatherability. So epoxy MIO paint is used for the topmost coat of the undercoat.

4. When primer is used, product blasting is performed at parts damaged during processing.

Fig. 1-17. Flowchart of quality control

Examination of drawings

↓ Examined by
person in charge in Design Section, person in charge of full-size drawing in Production Section, and chief charged with full-size drawing

Preparation of manufacturing manual

↓ Persons in charge in respective Sections

Preparation of full-size drawings

↓ (Full-size drawing operation standard)

Checked by
chief charged with full-size drawing

Cutting out of rulers and templates

↓ Checked by
chief charged with full-size drawing

Inspection of full-size drawings

↓ (Full-size drawing inspection standard)

Full-size drawing inspection sheet & report

Witnessed inspection by
person in charge in Inspection Section, person in charge of full-size drawing in Production Section, and chief charged with full-size drawing

Material acceptance inspection

Material procurement

↓

Material inspection

(Steel products application control standard)

(Material inspection standard)

Material inspection sheet & report

Witnessed inspection by
person in charge in Inspection Section and person in charge in Materials Section

↓

Shot blasting

For treating base sheets (including primer coating)
(Steel surface treatment standard)

Checked by
chief charged with blasting

↓

Cutting

(Automatic gas cutting operation standard)

(Mechanical cutting operation standard)

Steel sheet cutting inspection sheet
(Internal subcontractor)

↓ Inspected by
person in charge in Materials Section

Material rectification

↓ Checked by
chief charged with press operation

Marking-off of material (Marking-off operation standard)

↓ Checked by
chief charged with marking-off

Cutting, bending, drilling, etc. (Automatic gas cutting operation standard)
(Mechanical cutting operation standard)
(Drilling operation standard)

↓ Checked by
respective chief in charge

Sheet joining welding (Butt welding)

(Automatic welding operation standard)
(Butt welding operation standard)
(Welding rod storage standard)
(Manual welding operation standard)
(Welding operation environment control standard)
(Welding defects treatment standard)

↓ Checked by
chief welding engineer and
chief charged with welding

Bead finishing

Checked by
person in charge in Inspection Section

X-ray inspection — (Radiographic inspection standard)

Radiographic inspection report

Inspected by
person in charge in Inspection Section

Strain relieving rectification — Press

Checked by
chief charged with press operation

Marking-off of members — Primary post marking-off
(Prestraining operation standard)
(Marking-off operation standard)

Checked by
chief charged with marking-off

Member subassembly

I-shape, box-shape, crossframe, etc.
(Tack welding operation standard)
(Member assembling operation standard)
(General assembling operation standard)

Checked by
chief charged with member assembly

Welding

(Automatic welding operation standard)
(Manual welding operation standard)
(Welding rod storage standard)
(Welding operation environment control standard)
(Welding defects treatment standard)
(Weld appearance inspection standard)

Checked by
chief welding engineer and
foreman charged with welding

Strain relieving rectification

(Gas rectification operation standard)
Thermal rectification

Checked by
chief charged with rectification

↓

Marking-off of members

Secondary post marking-off
(Marking-off operation standard)
(Beam-end finishing lines, temporary drilling of joint section, etc.)

Checked by
chief charged with marking-off

↓

Cutting finish

Cutting machine
(Mechanical finish operation standard)

Checked by
chief charged with machine finish

↓

Drilling

(Drilling operation standard)
Temporary drilling

Checked by
chief charged with drilling

↓

Member inspection

(Members & parts inspection standard)

Witnessed inspection
Member inspection report

Inspected by
persons charged with inspection and production, chief welding engineer, and respective foreman and chief in charge

↓

Shop assembly

(Shop assembling operation standard)

Checked by
foreman and chief charged with shop assembly

↓

Hole enlargement

(Drilling operation standard)
Carried up to regular drilling

Inspected by
person in charge of inspection

↓

Shop assembly in-company inspection

(Shop assembly inspection standard)

Shop assembly inspection sheet

Witnessed inspection by
persons charged with inspection and production and respective foreman and chief in charge

↓

Shop assembly witnessed inspection

Shop assembly inspection report

↓

Disassembly

Pretreatment for painting

(Steel products surface treatment standard) Sand blasting, shot blasting, surface treatment

Checked by
person charged with inspection, and foreman and chief in charge of the product

Shop painting

(Painting standard)

Checked by
foreman and chief in charge of the product

Painting inspection

(Painting and completion inspection standard)

Witnessed inspection
Painting inspection report

Inspected by
persons charged with inspection and production, and foreman in charge of the product

Matchmarks for assembly

Checked by
foreman and chief in charge of the product

Packing

(Packing operation standard)

Shoe and accessories
(high-strength bolts, field rivets)

Checked by
persons charged with shipment and foreman and chief in charge of the product

Shipment

Purchased products

Witnessed inspection

(Members and parts inspection standard)

Product inspection report

Inspected by
persons charged with inspection

Receipt of products

(Product acceptance inspection standard)

Checked by
persons charged with materials and warehousing

SECTION 1.3 JOINING

1.3.1 Types of Joining

Types of joining and their applications are given in Table 1-21.

Table 1-21. Types of joining and their applications

- Joining
  - Welding
    - Shop welding
      - Butt welding
      - Fillet welding
      - Corner joint welding
    - Field welding
      - Butt welding
      - Fillet welding
      - Others
  - High-strength bolting
    - Bearing type high-strength bolts
    - Friction type high-strength bolts
      - Torque-shear type
      - Ordinary type
  - Ordinary bolting
  - Riveting

Welding is extensively used in shop fabrication. For field-joining, high-strength bolts are generally used. Under some work conditions, however, field-welded joints are also employed. High-strength bolts are generally favored in field work due to their reliability. Field-welding is mostly limited to use under good work conditions, being applied to structures in which high-tension bolt connections would detract from appearance or where the joint has a complex structure.

Table 1-22. Types and characteristics of welding rods used largely for steel structures

JIS standards	Coating types	Pertinent steels	Characteristics and scope of application
D 4301	Ilumenite	SS 41 SM 41	The most popular, applicable up to some 25 mm in plate thickness.
D 4303	Lime-titania	SS 41 SM 41	Assures better bead shapes, suitable for horizontal fillet welding and vertical and overhead welding.
D 4316	Low-hydrogen	SS 41 SM 41	Highest toughness and resistance to breaking, suitable for welding heavier thick plates.
D 4327	Iron-powder, iron oxide	SS 41 SM 41	Suited for high-efficiency flat and horizontal fillet welding, assuring better bead shapes.
D 5016	Low-hydrogen	SM50 SM50Y	Features generation of low levels of hydrogen that affects breaking, thus the most popular for welding high-strength steels.
D 5316	Low-hydrogen	SM53	
D 5816	Low-hydrogen	SM53	
D 5026	Iron powder, low-hydrogen	SM50 SM50Y	
D 5326	Iron powder, low-hydrogen	SM53	Suited for high-efficiency flat and horizontal fillet welding, assuring better bead shapes than with low-hydrogen electrodes.

D 50 16
- 16 — Classification of coatings
- 50 — Lower limit of tensile strength

Specialized welding rods

JIS standards	Exclusive names	Characteristics and application
D 4316, D 5016	Vertical down electrode	Downward fillet welding in the vertical position is possible with high efficiency.
D 5016	Extra low-hydrogen type	The generation of hydrogen is designed to be lower than that of ordinary low-hydrogen types, so it is suited for welding highly bound steel plates.
D 4316, D 5016, D 5316	Fume improved type	Improvement is made in the generation of noxious fumes associated with low-hydrogen electrodes, so it is suited for use in ill-ventilated environments.

1.3.2 Welding

1) Types and characteristics of welding rod

(1) Manual welding

Manual welding, the most widely used of all welding methods, is applicable to all joints of steel structures and all welding postures. Rod diameters are 3.2 mm, 4.0 mm, 5.0 mm and 6.0 mm, the choice being made according to size of bevel and leg length. Welding rods are also classified by usage, one type for mild steel and another for high-strength steel. (Table 1-22)

(2) Welding methods and their scope of application (Table 1-23)

Table 1-23. Application of various welding methods for steel structures

Welding methods	Types of joints
Submerged arc-welding	Various butt joints T joints and corner joints for assembly of joint sections T joints of stiffeners (in case of long weld length)
Consumable nozzle electroslag welding	Gathering points of plates, such as corners of rigid framework or diaphragm attachments (welding of all sections)
Semiautomatic CO_2 gas arc-welding	Butt joints with short weld length T joints and corner joints for assembly of sections Attachment of stiffeners, dowels and the like
Semiautomatic non-gas arc-welding	Usual joints in out door and field welding
Gravity or inclination welding with electrode set at lower angle	T joints of secondary members T joints of stiffeners
Manual welding	Welding in general

1.3.3 Joining by High-strength Bolts

1) Allowable stress of high-strength bolts (Table 1-24)

$Pa = \tau a \cdot Ao$

where

Pa: Allowable shearing (kg)

τa: Unit stress (kg/cm^2)

Ao: Sectional area calculated on the basis of outer diameter of bolt screws (cm^2)

Table 1-24.

Unit: kg

Grade of bolts		F8T		F10T		F11T	
τa (kg/cm^2)		1000		1250		1300	
Nominal bolt diameter	Ao (cm^2)	Single thread	Double thread	Single thread	Double thread	Single thread	Double thread
M16	2.011	2,011	4,022	2,514	5,028	2,614	5,228
M20	3.142	3,142	6,284	3,928	7,856	4,085	8,170
M22	3.801	3,801	7,602	4,751	9,502	4,941	9,882
M24	4.524	4,524	9,048	5,655	11,310	5,881	11,762
M27	5.726	5,726	11,452	7,158	14,316	7,444	4,888
M30	7.069	7,069	14,138	8,836	17,672	9,190	18,380
M33	8.553	8,553	17,106	10,691	21,382	11,119	22,238
M36	10.179	10,179	20,358	12,724	25,448	13,233	26,466

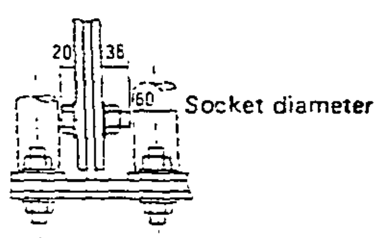
2) Minimum gap in high-strength bolt fastening (Table 1-25)

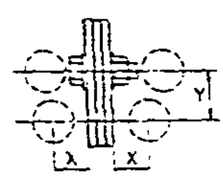
Table 1-25.

Unit: mm

Bolt head side	X	51	50	49	48	47	46	45	44	43	42	41	40	38	36	34	32	30
	Y	0~20	28	31	33	35	37	39	40	41	42	43	44	45	47	48	49	50
Nut side	X	69	68	67	66	65	64	63	61	59	57	55	52	49	46	42	38	33
	Y	0~11	19	22	24	26	28	29	32	34	36	37	41	44	46	48	50	51

Note: In case of M22 high-strength bolt and socket diameter of 60 mm.





3) Grip length of high-strength bolts (Table 1-26)

Table 1-26.

Unit: mm

Nominal diameter / Tightening length	M16	M20	M22	M24	Nominal diameter / Tightening length	M16	M20	M22	M24
6 ~ 10	40	45	50	55	71 ~ 75	105	110	115	120
11 ~ 15	45	50	55	60	76 ~ 80	110	115	120	125
16 ~ 20	50	55	60	65	81 ~ 85	115	120	125	130
21 ~ 25	55	60	65	70	86 ~ 90	120	125	130	135
26 ~ 30	60	65	70	75	91 ~ 95		130	135	140
31 ~ 35	65	70	75	80	96 ~ 100		135	140	145
36 ~ 40	70	75	80	85	101 ~ 105		140	145	150
41 ~ 45	75	80	85	90	106 ~ 110			150	155
46 ~ 50	80	85	90	95	111 ~ 115			155	160
51 ~ 55	85	90	95	100	116 ~ 120			160	165
56 ~ 60	90	95	100	105	121 ~ 125				170
61 ~ 65	95	100	105	110	126 ~ 130				175
66 ~ 70	100	105	110	115	131 ~ 135				180

Note: Grip lengths are the tightening lengths shown above plus the values in the table below, raised to a unit of 5 mm.
The maximum and minimum extra lengths in this case are as shown below.

Unit: mm

Nominal diameter	The value to be added to tightening lengths	Maximum extra length	Minimum extra length
M16	30	9	5
M20	35	10	6
M22	40	10	6
M24	45	13	9

4) Fastening control standard

(1) Processing of contact face (friction face)

- ☐ Projections around the edges of a bolt hole fully cleaned to remove floating rust, oil, black skin, etc., and must be processed to obtain a slip coefficient of over 0.45.
- ☐ Excess bolt-hole metal must be removed with a grinder.
- ☐ Fastening is done only after condition of processed contact face has been inspected.

(2) Gap of joints

- ☐ Framework members and connecting board must be in close contact by fastening.
- ☐ If misalignment occurs, joints must be tapered or the gap must be eliminated by padding.

(3) Storage and control of bolts

- ☐ Packaged bolts must be stored in a warehouse with little moisture, to protect them from rusting.
- ☐ In handling bolts, full care must be taken to avoid damaging the threads.

(4) Inspection of fastening force gauge and fastening machine

- ☐ Axial force gauges must be inspected once a year. However, when a gauge is used in a specific project, it must be inspected one month before first use and monthly thereafter.
- ☐ Torque wrenches must be inspected once a year. However, when a torque wrench is used in a specific project, it must be inspected one month before first use and monthly thereafter.
- ☐ Fastening machines must be inspected one month before use and once every two months thereafter.

(5) Determination of target torque value

(5)-1 Axial force of fastening bolt

The standard for fastening axial force is 10% above the design axial force of the bolt.

Type of steel	Nominal diameter of bolt	Design axial force	Fastening axial force
	M20	16.5 t	18.1 t
FIOT	M22	20.5 "	22.6 "
	M24	23.8 "	26.2 "

(5)-2 Classification of lots subject to inspection

Bolts are fastened by applying torque, and the fastening torque value (target torque value) of a lot subject to inspection is determined by the coefficient of torque measured in the field (field torque coefficient). Lots are classified as follows.

- ☐ For each production lot of bolts. However, when the difference in average values of torque coefficient in the inspection record is less than 5%, another production lot may be assumed to be the same as the working lot.
- ☐ Before start of work in morning and afternoon each day.
- ☐ When temperature and other weather conditions change suddenly.

(5)-3 Determination of target torque value

- ☐ Five bolts are chosen at random according to the classification of lot subject to inspection.
- ☐ They are fastened, using axial force gauge and torque wrench.
- ☐ Torque value is read when specified axial force is reached.
- ☐ Five sample bolts are fastened with axial force of −10% of standard axial force, standard axial force itself and +10% of standard axial force, in that order, and respective torque values and torque coefficients are obtained.
- ☐ Torque coefficient (K) and target torque value (T) are then determined from the average of the results of measurement for the five bolts.

Formula:

$$T = K \times d \times N \qquad K = \frac{T}{d \times N}$$

where T = fastening torque value Kgf•m; K = torque coefficient value; d = bolt diameter (mm); and N = bolt axial force (ton).

(5)-4 For bolts of short length neck (70 mm and under)

- Calibration described in (5)-3 above is omitted and torque coefficient value is determined by mill sheet or bolt inspection record, and target torque value is then determined.
- If bolts of lengths over 70 mm are used in the same work and the difference in average values of torque coefficient in the bolt manufacturer's inspection record is less than 5%, calibration is carried out on the assumption that the bolts belong to the same working lot.

(6) Adjustment of fastening machine

A nut runner (NR) and torque wrench (preset type) are used to adjust the fastening machine.

- The fastening machine is adjusted daily, before start of work.
- It is generally adjusted for each lot subject to inspection as specified in (5)-2.
- The fastening machine must be adjusted so that the average value of fastening axial force for more than five sample bolts comes within the range given below.

Type of steel	Nominal bolt diameter	Design axial force (t)	Fastening axial force (t)	More than 5 bolts	
				Lower limit	Upper limit
F10T	M20	16.5	18.1	17.2	19.0
	M22	20.5	22.6	21.5	23.7
	M24	23.8	26.2	24.9	27.5

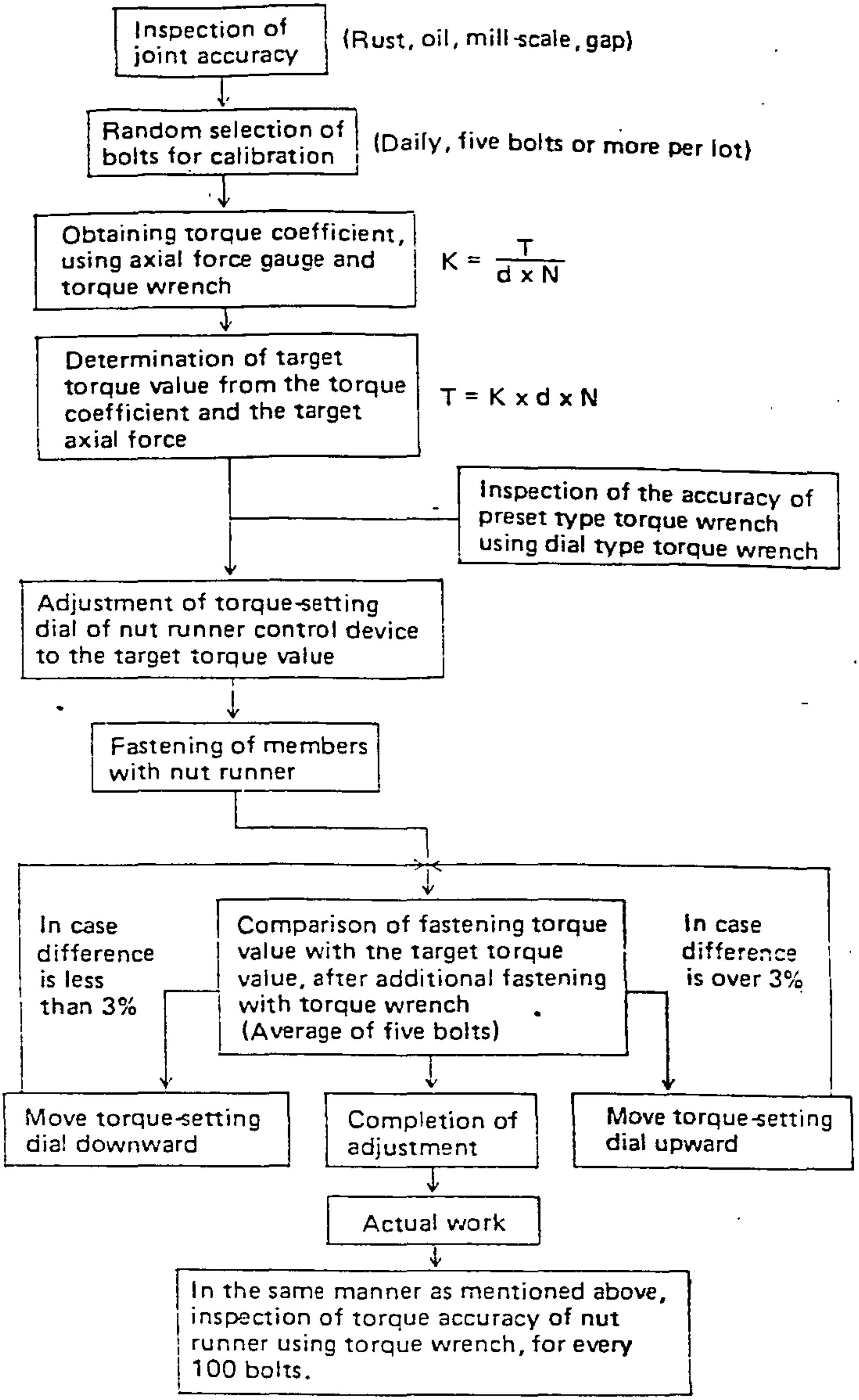
- Adjustment standards for fastening machine (Fig. 1-18)

(7) Fastening method

- Order of fastening is from the center toward the end in a set of bolts, and care must be taken that previously fastened bolts do not come loose.
- Fastening must generally be done in three stages:
  - * Hand fastening: with a spanner (wrench)
  - * Intermediate fastening: with an impact wrench or the like, within 60% to 70% of bolt axial force.
  - * Final fastening: with a nut runner, preset-type torque wrench or the like, up to 100% of bolt axial force
- To confirm that bolt and nut do not turn together in final fastening, bolts, nuts, washers and base metal are marked with paint or other means after intermediate fastening.
- As a rule, a bolt is fastened by turning the nut. If the bolt head must be turned, a corrected torque value must first be obtained from a test piece.

Fig. 1-18.

Adjustment standards for fastening machine



(8) Fastening inspection

- ☐ After fastening is completed, more than 10% of the bolts emplaced are chosen at random from each group of bolts during the same day, unfastened with a torque wrench and inspected.
- ☐ Average difference between target torque value and field fastening torque value must be less than ±4%.

$$-4\% \leqslant \frac{T_1 - T_0}{T_0} \times 100 \leqslant 4\%$$

where T_1.....Average field torque value
T_0.....Target torque value

- ☐ Difference between field fastening torque value of individual bolts and target torque value must be within ±7%.
- ☐ Bolts that have been excessively tightened (more than 7% greater than target torque value) must be replaced by new ones, and bolts inadequately tightened (more than 7% smaller than target torque value) must be retightened.
- ☐ If the difference between average field fastening torque value and target torque value exceeds ±4%, inspection must be repeated, using the same number of bolts as were subjected to the preceding inspection. If two successive inspections are failed, all bolts must be inspected.

(9) Points of note about fastening work

- ☐ Fastening of bolts must not be carried out in rainy weather.
- ☐ If an electric fastener is used, a reaction absorber must also be used. Note the lowering of voltage, and never work more than 100 m away from power source.
- ☐ Make certain that there is no residual current in the electric fastener before fastening the next bolt.
- ☐ Excessive fastening of bolts is more dangerous than short fastening. Full caution must be exercised.

SECTION 1.4 CORROSION AND PROTECTION MEASURES FOR STEEL PRODUCTS

1.4.1 Causes of Corrosion

Iron and steel products are made by reducing iron ore (iron oxide) — a process of forcibly removing the oxygen from the iron. It is a natural tendency for iron and steel to try to revert to the more stable form of iron oxide (rust). This reversion from metal to oxide in a conductive environment is called corrosion. There are two general types: "wet corrosion", which occurs in the presence of moisture; and "dry corrosion", which occurs without moisture. For steel products used in civil engineering works, dry corrosion may be ignored since these structures are exposed to water in the environment: fresh water, sea water, water in the air and the soil.

Wet corrosion occurs through the influence of neutral water and oxygen. The effect of this water-and-oxygen combination varies with the environment, and the speed and extent of corrosion vary accordingly. But the same chemical reaction is always involved:

$$Fe + H_2O + 1/2O_2 \rightarrow Fe(OH)_2$$

That is, iron (Fe) is attached by water (H_2O) and oxygen (O_2), producing ferrous hydroxide. This compound reacts further with oxygen, together with addition or separation of water, to form "rust" — stable oxides such as $FeOOH$, Fe_3O_4 and noncrystalline substances.

The turning of iron and steel into "rust" is an electrochemical reaction, operating in the same manner as a galvanic cell. On the metal surface, many tiny voltage differences may be set up for various reasons, such as the presence of a minute speck of impurity. Moisture on the metal surface, containing air or dissolved chemicals, acts as a conductor between these microscopic anodes and cathodes, allowing minute currents to flow. At the site of each anode, the current causes iron to oxidize. As corrosion progresses, the tiny anodes and cathodes shift their positions on the metal surface, so that iron and steel usually corrode evenly.

1) **Effects of environmental factors on corrosion**

Since the rate of corrosion in neutral environments such as fresh water, sea water, air and soil is determined by the rate at which oxygen reaches the steel surface by diffusion, any factor that increases oxygen diffusion promotes corrosion. The main factors include concentration of residual oxygen in water, flow rate, extent of mixing and temperature.

If the amount of residual oxygen dissolved in water is large, the amount of diffused oxygen will increase. At any given residual oxygen concentration, as flow rate increases the amount of oxygen reaching the exposed steel surface will increase. A rise in temperature will increase the rate of diffusion.

Rust developing on a steel surface hinders somewhat the diffusion of oxygen and thus retards further corrosion to a degree. This protective action of rust varies widely with service environment, even for the same kind of steel. It is comparatively small for rust developed in water and soil, but is substantial for rust developed in the open air. For steel immersed in water, when the water flow rate is high it tends to remove surface rust mechanically, expediting further corrosion. This effect is accelerated when the flowing water contains solids such as suspended sand near the ocean bottom.

The mechanism of corrosion is not generally affected by the presence of neutral salts, such as table salt, in water. But their presence does change the concentration of dissolved oxygen and also affects the protective nature of surface rust in hindering the diffusion of oxygen.

2) **Effects of the metal's properties on corrosion**

As noted above, corrosion of steel in neutral environments such as fresh or sea water, air and soil is governed by the diffusion of oxygen. Among low-alloy steels, differences in metallic structure have little effect on the ability of the surface rust layer to hinder the diffusion of oxygen. Nor do differences in stress and plastic deformation of the steel.

This is true, however, only when the item of steel is uniform in metallic structure and is subjected to uniform stress and plastic deformation. When there are local differences in these factors, the electrical potential in water may vary, causing one part to act as the anode of a corrosion cell and resulting in concentrated corrosion. Also, when two members made of different materials are in contact, one may act as the anode of a corrosion cell. In these cases, the corrosive effect is strongest near the contact point. It extends beyond the contact point in proportion to the difference of electrical potential between the two members and the conductivity of the environment.

Mill scale on a steel surface provides some protection, but it always contains cracks and pinholes where corrosion can begin and then spread to surrounding parts of the surface. Mill scale has a negative electric potential against the base metal, making the scale the cathode of a local galvanic cells and thus accelerating the corrosion of the base steel.

When low-alloy steels begin to corrode, the alloying elements they contain become part of the rust. This affects corrosion resistance by changing the nature of the rust layer. For service in the air or in sea water, proper formulation of alloying elements such as phosphorus, copper and chromium in a steel provides corrosion resistance two to three times that of ordinary steel. For service in fresh water or in soil, however, the corrosion resistance of low-alloy steel is little different from that of ordinary steel.

1.4.2 Relationship of Rust Thickness to Loss in Steel Thickness

A substantial part of the Fe content lost by corrosion of a steel surface (generally more than half) remains in the rust layer that forms on the steel surface. Fe is also lost in rust that is dislodged from the steel, and as Fe in ionic form which is washed away without becoming part of the rust.

The proportions of Fe content lost in these ways vary with the service environment The percentage of Fe remaining in the form of a rust layer on the steel surface is usually high in soil and comparatively low in the air. It also varies with flow rate when the service environment is water.

Assuming that 100% of the Fe content lost by corrosion remains in the rust layer on the steel surface, the relationship between the thickness of the rust layer and the loss in thickness of the steel is given by formula below:

$$y = \frac{D \cdot S \cdot x}{100\mathrm{A}}$$

where
- y = loss in thickness of steel (mm)
- D = apparent specific gravity of rust
- A = specific gravity of FE (7.85)
- x = thickness of rust layer (mm)
- S = Fe content of rust layer (%)

The following example is calculated from data obtained from the corrosion of steel pipe piles in a marine environment (atmospheric zone ~ submerged zone). Here, D = 2.01 and S = 52.4% in formula above.

$$y = \frac{2.01 \times 52.4}{100 \times 7.85} x = 0.134x \qquad x/y = 7.46$$

In this example, it is thus found that the thickness of the rust layer is about 7.5 times the loss of thickness of the steel. This result clearly shows the steel. Even where only about half the Fe lost by corrosion remains in the rust layer, the thickness of the rust layer will be substantially greater (about 3 ~ 4 times) than the thickness lost by the original steel.

Jumping to the conclusion that thickness of rust layer = thickness lost by steel is therefore an error. Fig. 1-19 shows the actual relationship between the two.

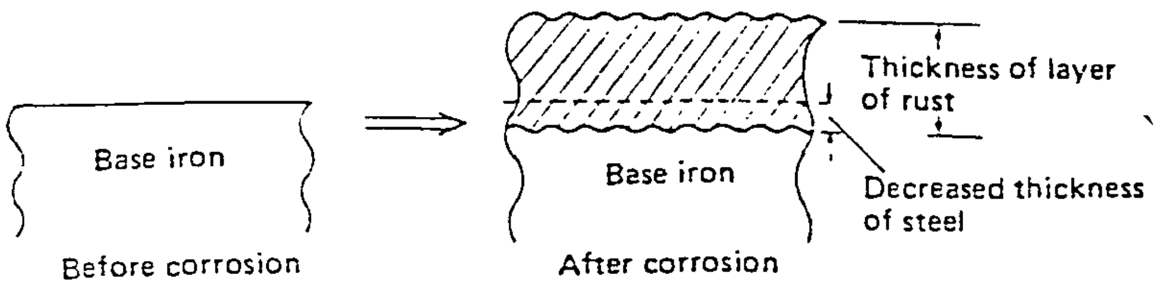


Fig. 1-19. Relationship of rust thickness to loss in steel thickness

1.4.3 Service Environment and Corrosion

1) Corrosion in sea water

The corrosive properties of sea water relative to steel are basically no different from those of fresh water. Because of its higher content of salts, however, sea water has greater electrical conductivity and tends to cause local corrosion due to its high ion content. The corrosion rate of steel in sea water (less than 0.1 mm/yr), in fact, averages twice the rate in fresh water.

Since offshore steel structures have many long vertical members extending from underwater to the surface and above, corrosion of those parts of the structure in the splash zone (just above the mean high water level) is the greatest problem. This is clear from Fig. 1-20, which shows the distribution of corrosion along such members.

In the tidal zone, where the surface of the sea rises and falls regularly with the tide, it would seem that corrosion would be greater due to the frequent alternation of wetting and drying. Yet it is less severe than in the splash zone. This is because a galvanic cell is formed between the part of the structure in the tidal zone and the part just below the surface of the water, due to the difference in oxygen supply. The part in the tidal zone is thereby protected electrically from corrosion, while corrosion of the part below the surface is accelerated. The corrosion peak shown immediately below the surface in Fig. 1-20 reflects this situation.

For steel structures designed for ports and harbors in Japan, values of one-side corrosion rate as given in Table 1-27 are used.

2) Corrosion in the atmosphere

Atmospheric corrosion of steel proceeds by the same mechanism as does corrosion in water, but there are several features peculiar to atmospheric corrosion. It occurs when the steel surface is wet, from rainfall or dew, and the relative humidity is high (over 70%). An invisible thin film of water then covers the steel surface. Repeated wetting and drying with changes of weather build up a layer of rust on the steel surface. Unlike the rust layer formed in water, however, that formed in the air offers greater protection against further corrosion. Once a layer of rust develops, the rate of rusting diminishes.

In addition, the degree of rusting in the air is strongly influenced by the presence of various rust-promoting substances in the atmosphere. Typical of the substances that can worsen rusting if present in the atmosphere are sulfur dioxide and granular sea salt (NaCl).

Fig. 1-21 is an example of exposure test results for steel in an industrial zone (Tokyo area).

The protective effect of a rust layer, though true of carbon steel, is more notable for atmospheric corrosion-resistant steel containing small amounts of copper, chromium, phosphorus, nickel and molybdenum. Such steel rusts similarly to carbon steel at first, but with the passage of time corrosion slows and after about five years it ceases entirely. This is because part of the rust that forms gradually becomes a stable layer that adheres strongly to the metal surface, creating a protective film that halts the further advance of corrosion.

Unpainted carbon steel is rarely used in the open air, but unpainted atmospheric corrosion-resistant steel gives excellent service and in addition takes on a beautiful surface appearance, ranging in color from dark brown to dark blue.

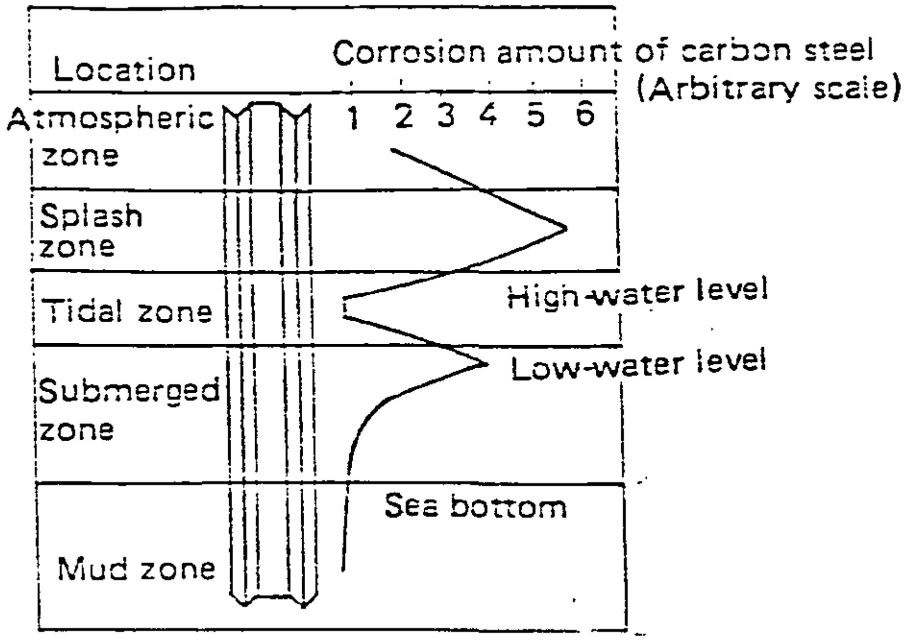
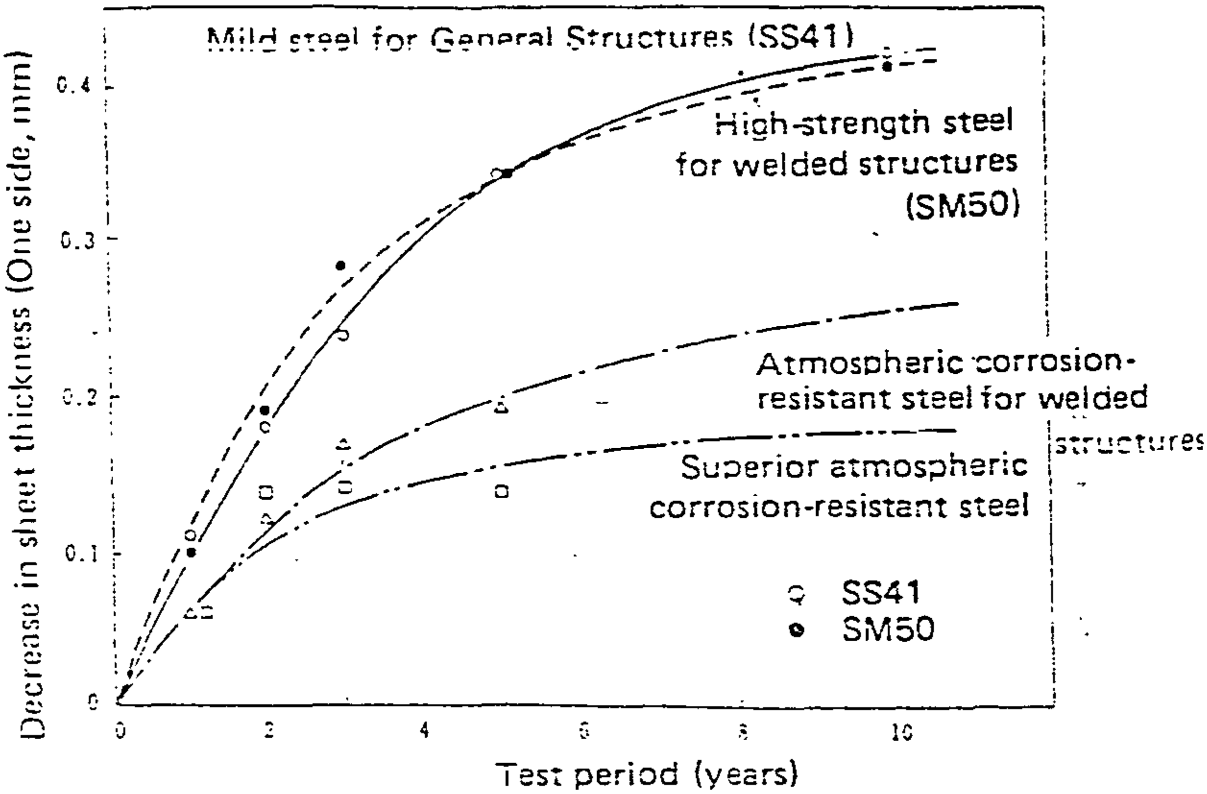


Fig. 1-20. Corrosion progress in different marine environments

Table 1-27. Standard corrosion amount of steel

	Corrosive environment	Corrosion amount (mm/yr)
Sea area	Above high-water level	0.3
	Between high-water level and sea bottom	0.1
	In the seabed mud stratum	0.03
Land area	Above the ground (In the atmosphere)	0.1
	Under the ground (Above the residual water level)	0.03
	Under the ground (Under the residual water level)	0.02



Note: Corrosion amounts in atmospheric exposure vary widely depending upon the environment. The data above shows the results of an exposure test made in an industrial zone in Tokyo.

Fig. 1-21. Example of corrosion rates of atmospheric corrosion-resistant steel and carbon steel

3) **Corrosion in the soil**

The corrosive effect of soil on steel is determined by specific electrical resistance, air permeability (porosity), moisture content, acidity, dissolved salts and other factors. The most important of these is relative resistance, and its value practically determines the corrosive effect of a given soil. (Table 1-28)

Except in highly acidic soil, corrosion occurs by the action of oxygen and water. Since many factors affecting corrosion act at the same time, however, corrosion is not necessarily greater in high-porosity soil. In fact, the reverse true in many cases.

The corrosive effect of soil varies greatly from one location to another. As an example, the results of corrosion testing for steel in the United States for 12 years are given in Table 1-29. As can be seen in the table, average corrosion values are at most only 0.06 mm/yr. But corrosion of steel in soil is likely to be localized, with deep pit corrosion occurring in most cases. This is because soil in contact with a steel surface is less uniform in its qualities than is fresh water or the open air.

The rate of pit corrosion tends to decrease with time:

$$P = kt^n$$

where P = maximum pit depth at time t, and k and n (n = 0.1 ~ 0.9) are constants. For low-porosity soil, n has a high value, and there is thus little slowing of pit corrosion with time.

The features of corrosion discussed above apply to cases where the soil is excavated for pipelines or similar installations. But for steel pile driving, where the soil is less disturbed, less corrosion is noted.

Table 1-28. Relationship between relative resistance and corrosive effect

Corrosive effect	Relative resistance (ohm — cm)
Extremely great	0 ~ 1,000
Greater	1,000 ~ 2,000
Medium	2,000 ~ 5,000
Less	5,000 ~ 10,000
Extremely less	> 10,000

Table 1-29. Ranges of corrosive effect of soil (carbon steel)

	Average corrosion value (mm/y)	Max. pitting corrosion (mm)
Max	0.06	> 5.4
Min	0.003	0.4
Average	0.02	1.8

Long-term anticorrosive painting, use of atmospheric corrosion-resistant steel and use of steel protected by hot-dip galvanizing or spraying of zinc or aluminum are mostly used. Combinations of these measures can ensure protection of onshore structures for long periods.

(1) Long-term anticorrosive painting

MIO paint with high adhesion is used between base and middle coats of anticorrosive paint, and high-performance chlorinated rubber resin is used as the finishing coat. For some structures, zinc-rich paint is used as the anticorrosive coating and thick-film epoxy resin or polyurethane resin paint for finishing.

(2) Atmospheric corrosion-resistant steel

Protection of this steel against corrosion is provided by the formation of a stable, strongly-adhering surface layer of rust, rather than by an applied coating of paint. The steel contains alloying elements such as copper and phosphorus. The dense, inactive rust layer that forms upon initial exposure to the atmosphere is a protective film, effective against further corrosion. Its effects are shown in Fig. 1-21.

(3) Hot-dip galvanizing

When steel is dipped in molten zinc and withdrawn, a coating of zinc remains on the steel surface and solidifies. Because this coating method is relatively simple and the zinc coating provides good protection against atmospheric corrosion, it is widely used for such structures as steel towers and bridges, and for roofing materials.

The coating comprises a pure zinc layer and an iron-zinc alloy layer. When zinc is exposed to the air, a film having anticorrosive properties is formed. The protection it provides depends in good part on the film's density and thickness. For a given service environment, useful life of a hot-dip galvanized item is directly related to the thickness of the zinc coating: the thicker the coating, the longer the useful life.

Fig. 1-26 shows the useful life of hot-dip zinc coatings according to service environment.

There are more than 200 zinc-coated bridges in Japan. A survey of three such bridges was carried out in November 1979, comparing the thickness of the zinc coating with values in Fig. 1-26. Useful life was then determined up to the point when 10% of the zinc coating would remain. Results are shown in Table 1-31.

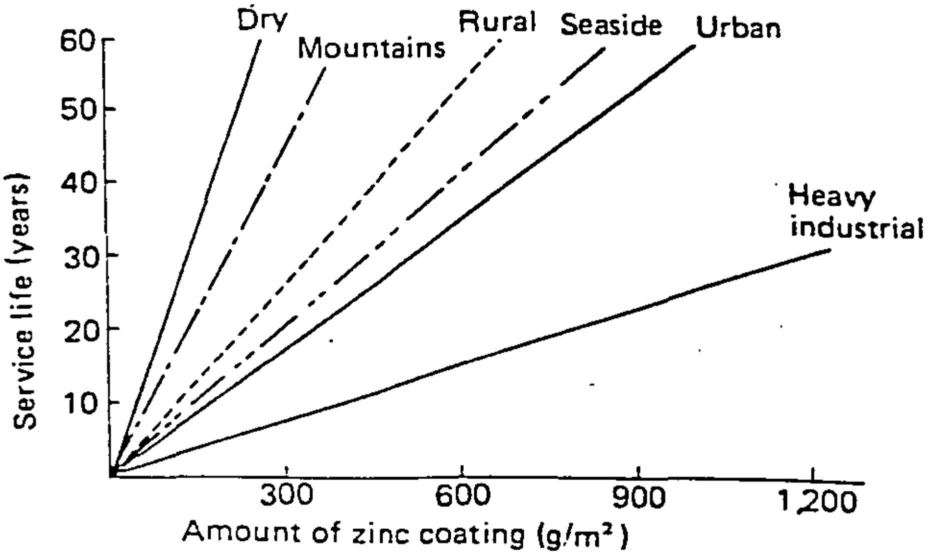


Fig. 1-26. Zinc coating amount and service life in different environments

Table 1-31. Results of research

	Minimum coating thickness measured (μ)	Amount of zinc coating (g/m^2)	Corrosion amount (g/m^2/year)	Estimated future service life (years)
Shin Nukuigawa Bridge	111	795	10	72
Yomoyori Crossing Highway Bridge	118	845	10	76
Adachi Elevated Highway Bridge	127	909	15	55

(4) Melt-spraying of zinc and aluminum

Molten metal, sprayed by gas or air, covers the steel surface and solidifies in this method of protecting steel structural members. It has the advantage of being usable for on-site treatment, including the spraying of large structures, and its use has gradually been increasing. Tables 1-32 and 33 show the relationship between thickness of sprayed coating and useful life, as estimated from actual test results.

In industrial and marine atmospheres, aluminum coatings provide much better corrosion protection than do zinc coatings. A combination of melt-sprayed coating and painting achieves far superior protection, and is used on long-span bridges.

Table 1-32. Average thickness of melt-sprayed zinc coating (mm) and estimated service life

Estimated service life / Environment	10 ~ 20 years	20 ~ 40 years	40 years or over
Ordinary atmosphere	0.1 ~ 0.15	0.15 ~ 0.30	0.30 and over
Saltish atmosphere	0.25 ~ 0.30	0.30 ~ 0.37	0.37 and over

Table 1-33. Average thickness of melt-sprayed aluminum coating (mm) and estimated service life

Estimated service life / Environment	5~10 years	10~20 years	20~40 years	40 years or over
Industrial atmosphere	0.12~0.15	0.15~0.20	0.25~0.30	0.30~0.37
Saltish atmosphere	0.15~0.20	0.20~0.25	0.25~0.30	0.30~0.37

Source: J.E. Wakefield: Iron Age, 165 No. 2 (1950)

1.4.5 Maintenance of Painting

As mentioned above, paint does not provide permanent corrosion protection because it gradually deteriorates. Therefore, the condition of painted steel surfaces must be constantly checked and repainting carried out when needed. The timing of repainting is generally determined by visual observation of the extent to which the paint film has deteriorated, and is undertaken when the extent of corrosion has reached a preset standard. The durability of the paint film is higher, however, if repainting is done at an early stage, when rusting is not so advanced. A shorter cycle of repainting increases the cost of maintenance, but it lowers the cost of depreciation since more frequent repainting extends the useful life of the structure. The optimal repainting cycle is generally determined as that which minimizes the sum of maintenance and depreciation costs. This requires several repetitions of the calculation.

The latest trend is to adopt a painting system that provides a highly durable paint film and then to extend the repainting cycle so as to minimize maintenance cost. Such systems include the use of atmospheric corrosion-resistant steel as base metal, or use of either a zinc-rich primer with good anticorrosive properties or melt-sprayed aluminum as primer. Although initial costs are high for special steel and extra painting, their use is considered more economical in the long term since maintenance costs are lower.

Underground structures

Steel pilings and underground pipelines are the most common underground steel structures.

(1) Corrosion protection of steel piles

Table 1-29 shows average corrosion rates for steel in soil. Japan's Soil Engineering Association surveyed the extent of corrosion of underground steel structures at 10 locations having different ground conditions, over a 10-year period. The results showed an average corrosion rate (both faces) of 0.0106 mm/yr.

Steel piles, due to their function, do not require measures against pit corrosion. Assuming a corrosion rate of 0.02 mm/yr will ensure an adequate service life, as can be seen from Table 1-29 and the results of the survey mentioned above. Also, since the rate of corrosion is very low for steel structures in the soil, such protective measures as coating are unnecessary for steel piles. Taking a corrosion allowance is more widely used. The useful life of onshore structures in Japan is about 80 years, and about 2 mm corrosion allowance is generally provided for these structures.

(2) Corrosion protection of underground pipelines

Underground pipelines, having a function different from that of steel piles, do require corrosion protection. Differences in ventilation, electric current leaks and pit corrosion may cause corrosion of pipelines. Protective measures include use of asphalt jute wrapping, polyethylene coating and coal tar enamel glass cloth covering. For important installations such as major long-distance pipelines, electric protection is also used.

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DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MONTAJE DE ESTRUCTURAS PARA EDIFICIOS

SUBTEMA

REQUISITOS MÍNIMOS DE CONOCIMIENTO DEL COMPORTAMIENTO DEL ACERO Y SU APLICACIÓN A ESTRUCTURAS DE EDIFICIOS

a).- PRODUCTOS DE ACERO
b).- COMPORTAMIENTO ESTRUCTURAL DEL ACERO

ING. VÍCTOR SÁEZ DE OCARIZ ALBISÚA
PALACIO DE MINERÍA
SEPTIEMBRE / OCTUBRE DE 1998

SECTION 1.1 STEEL PRODUCTS AS STRUCTURAL MEMBERS

Steel structural members can be small in sectional area, because steel is far stronger than wood or concrete. The dead load of the structure (its own weight) is therefore relatively small, and the proportion of dead load is also small. This means, however, that it is necessary to estimate the external load acting on a steel structural member as precisely as possible. To enclose a large space such as a gymnasium, for example, a steel truss structure is frequently employed since it has a low dead load though it is large, and is thus an economical choice.

For steel members, with their small sectional areas, buckling and other kinds of deformation must be prevented. Otherwise, such members may be subject to bending, buckling or torsional deformation even though simple calculations indicate a sufficient yield strength. In designing a steel-frame building, deformation must be considered in addition to structural calculations. An important part of the study of deformation of steel structures and their members is the methods of joining members and column bases, and the design conditions of the joints. Neglect of these considerations may result in deformation of the steel structure, meaning failure of the steel structure to meet the design requirements.

Nevertheless, the low dead load of the steel structure offers advantages. Regarding the building foundation, for example, on soil of low bearing capacity where a reinforced concrete building would require a pile foundation, a direct foundation is sufficient for a steel-frame building. Such a foundation is less expensive and is completed more quickly. In an ultra-high-rise building, steel structure is used because, in addition to the higher toughness of a steel frame, its dead load is lower than is possible for reinforced concrete. Thus the steel members are smaller in sectional area, leaving more of the interior space in the building free for use.

SECTION 1.2 MECHANICAL PROPERTIES OF STEEL

The mechanical properties of steel are yield point, tensile strength, percentage elongation and other properties measurable by tensile testing. The behavior of steel during tensile testing, in which a steel test-piece is subjected to a stretching load (tensile stress), is described below.

The relation between the tensile stress σ and the strain ϵ caused by the stress in a steel test-piece of standard cross section is shown in Fig. 1-1. As a steadily greater tensile stress is applied, stress and strain are at first proportional, as represented by segment OA of the stress/strain curve. (If, for example, a certain stress causes the test-piece to lengthen by 0.1 mm, twice that stress will cause it to lengthen by 0.2 mm.) This relation, known as Hooke's law, can be expressed as:

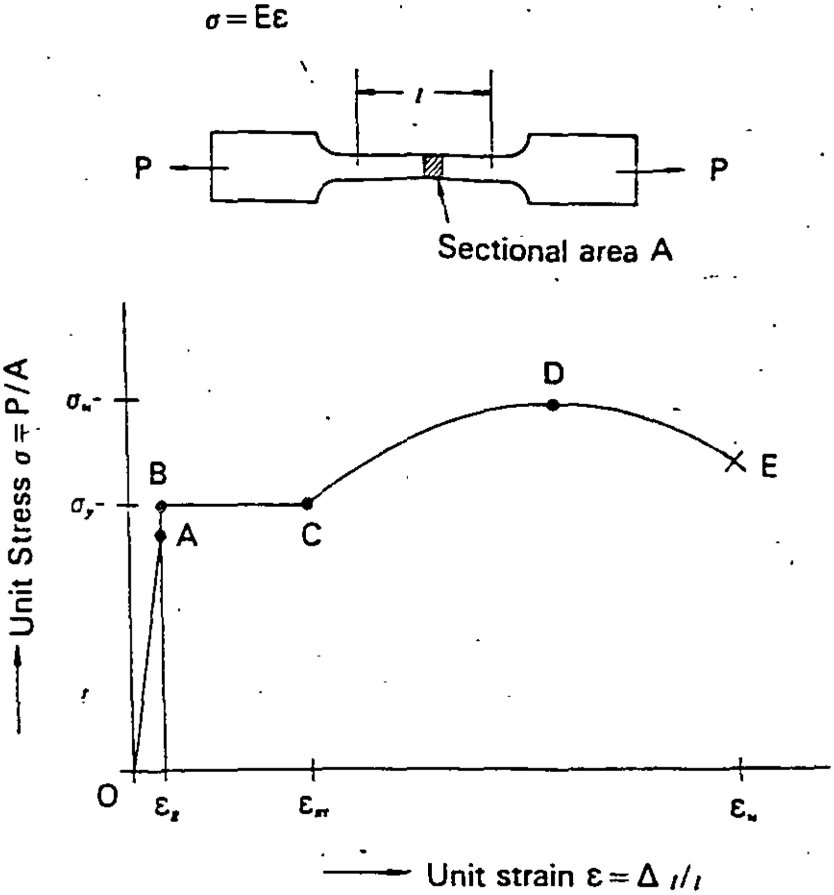


Fig. 1-1. Stress vs strain in a steel test-piece

where the proportionality constant E is called the modulus of elasticity, or Young's modulus, for the material being tested. If the load is removed from the test-piece during the initial part of a tensile test (represented by segment OA of the curve), the strain disappears and the test-piece returns to its original dimensions.

As the load is increased beyond point A, stress and strain gradually lose the strict proportionality of Hooke's law, and the stress/strain curve no longer follows the extension of the straight segment OA. Point A is thus called the proportional limit. If the load is removed from the test-piece at some point between A and B on the curve, the test-piece will not return completely to its original dimensions; a little residual strain will remain.

When point B is reached, strain continues to increase with no additional increase of stress. This phenomenon is called "yield". Segment BC is called the yield shelf. The unit stress corresponding to B on the curve is the yield stress or yield point, and is expressed as σ_y. Since points A and B are rather close together, for the steels ordinarily used in building construction it may be considered in general that Hooke's law holds true up to point B, at which yielding begins.

When strain in the test-piece reaches ε_{st} (point C), stress resumes its increase. This phenomenon is called "strain hardening". As the load is further increased, the relation between stress and strain changes steadily, as shown by segment CD and beyond on the curve. Change in stress corresponding to a given change in strain is much greater than in the early stage of the test represented by segment OA. Unit stress reaches a maximum value at point D, after which it declines while strain continues to increase. The unit stress at point D is called the maximum unit stress or tensile strength, and is expressed as σ_u. Finally, when point E is reached, the test-piece breaks. In addition to tensile strength, the tensile test also measures the mechanical property called elongation, which is simply the final elongation of the test-piece, at point E.

Segment OB of the curve, where the stress/strain relationship obeys Hooke's law or nearly so, is called the "elastic zone". Segment BE, where Hooke's law does not apply, is the "plastic zone". The ratio σ_y/σ_u (yield point/tensile strength) is called the yield ratio — another mechanical property that is used as a criterion for steel application.

The strength of various steels is generally expressed in terms of tensile strength, by which steels are grouped into the 40-kg class (σ_u = 40 kg/mm^2), 50-kg class, 60-kg class and so on. Steels with tensile strengths of up to about 160 kg/mm^2 are in use as structural steels.

Yield point generally rises with increasing tensile strength, but at a faster rate. That is, the yield ratio increases with increasing strength. Also, elongation is generally less at higher tensile strength, resulting in a shorter yield shelf. For steels having a tensile strength greater than 70 kg/mm^2 and steels that have undergone plastic working, the yield shelf is almost lost. That is, they show almost no yielding.

The stress/strain curves for steels of various tensile strength ratings are compared in Fig. 1-2. The straight segments of these curves, representing the elastic zone, overlap. That is because the modulus of elasticity (E) of all steel materials is the same, regardless of tensile strength. For steels that have high tensile strength or that have undergone plastic working, the 0.2% offset proof stress is used instead of the yield point. This is the unit stress that causes a residual strain of 0.2%, as shown in Fig. 1-3, and it is expressed as $\sigma_{0.2}$ in the Japanese Industrial Standard (JIS).

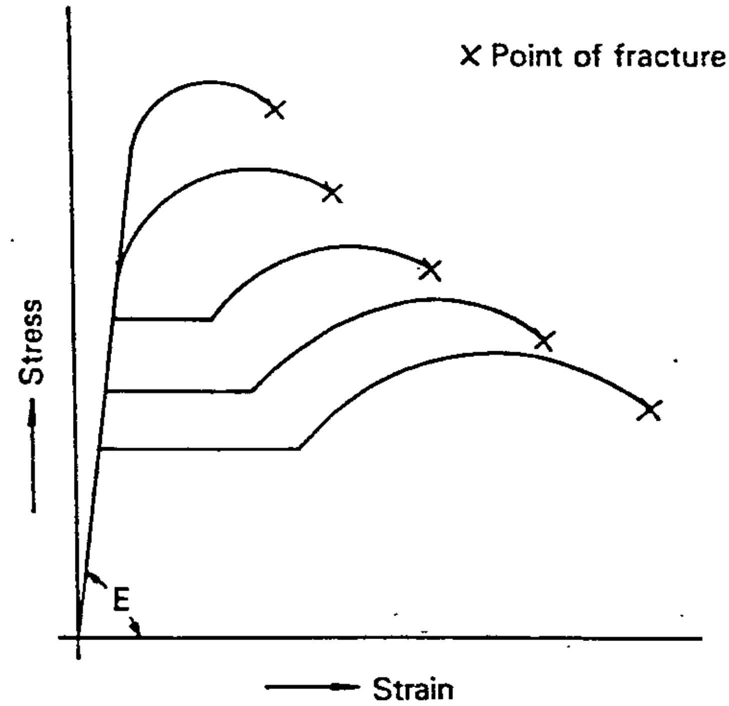


Fig. 1-2. Stress/strain curves for steels of various tensile strengths

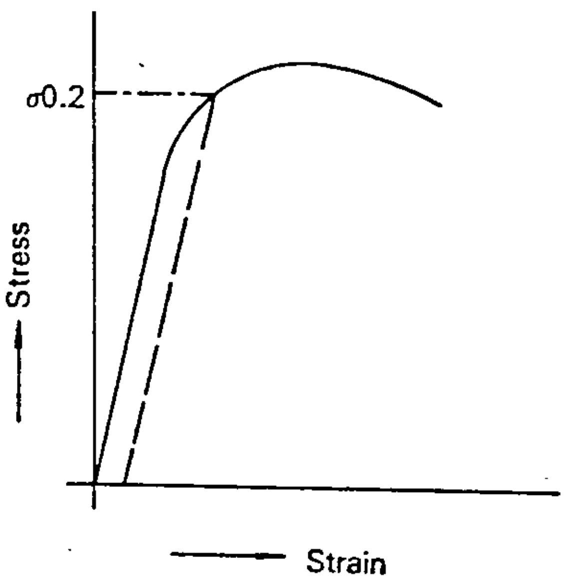
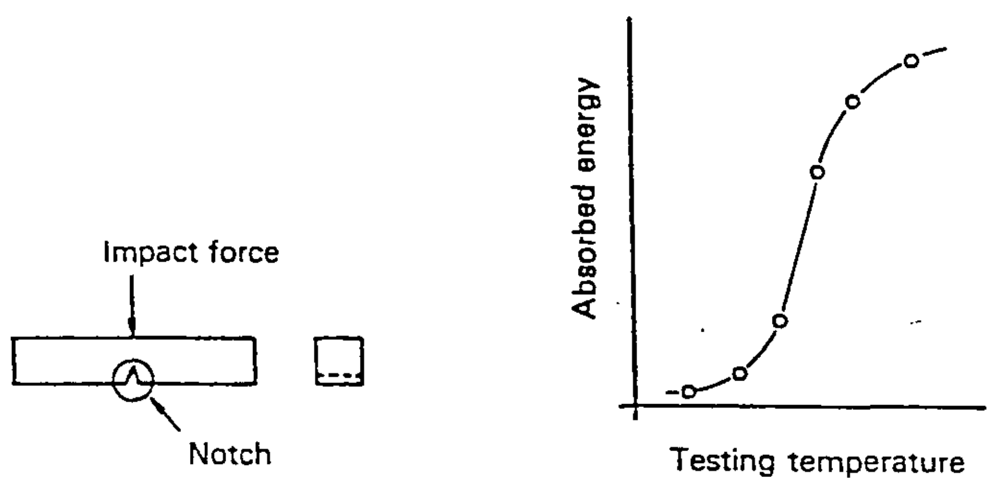


Fig. 1-3. 0.2% offset proof stress

Certain other important mechanical properties of steel, such as impact value and hardness, must be measured by means other than tensile testing. The impact value expresses the notch toughness of a steel, and is often taken into account for welded structures. It is the amount of energy absorbed by a V-notched test-piece of square section (Fig. 1-4 (a)) when it is broken by the impact of a heavy pendulum swinging downward from a specified height. Impact values are useful for comparison between materials: the larger the impact value, the tougher the material. But they cannot be correlated to material properties as closely as can tensile test results. For example, impact value for the same material varies greatly with testing temperature. For accuracy, therefore, it is necessary to obtain the relation between temperature and impact curve by testing at several temperatures. This relation, called the transition curve, is shown for one material in Fig. 1-4 (b). The impact value at 0°C is often used a representative value for a given material.



(a) Charpy impact test-piece

(b) Results of impact test

Fig. 1-4. Impact test-piece and test results

SECTION 1.3 METALLURGICAL PROPERTIES OF STEEL

Steels used for steel-frame building construction include those in the 40-kg class, which are usually called carbon steel, and high-strength steels in the 50-kg class or above. High-strength steels range in tensile strength from 50 kg/mm^2 to more than 100 kg/mm^2. High-strength steels rated at 50 - 60 kg/mm^2 are produced by adding carefully controlled amounts of suitable alloying elements to the steel. For high-strength steels in the 60-kg class and above, addition of alloying elements is supplemented by heat treatments such as hardening and tempering. They are called tempered steels, while high-strength steels produced by addition of alloying elements and application of rolling techniques alone are called non-tempered steels.

In most cases, steel materials are welded during the course of fabrication. Therefore, steels must not only possess high strength but must also be suitable for welding. For good weldability, a steel should not show high hardness in welded parts, but should have adequate elongation and notch toughness even in the heat-affected zone adjacent to a weld. Since weldability is affected by the kinds and amounts of alloying elements present in the steel, it is important to restrict both to the extent possible.

The influence on mechanical properties and weldability of the main chemical components in steel is summarized below.

(1) Carbon (C): the primary chemical component that determines the properties of a steel. As carbon content increases, the tensile strength, yield point and hadness increase but the elongation decreases, making the steel more brittle. Carbon has the greatest influence on weldability.

(2) Manganese (Mn): increases the strength and hardness of a steel and decreases elongation slightly, but causes less loss of toughness than does carbon. Manganese also prevents brittleness due to sulfur.

(3) Silicon (Si): raises yield point, but causes brittleness if too much is added (2% or more).

(4) Phosphorus (P) and sulfur (S): increase the brittleness of steel as their content rises. Both tend to separate out (segregate) in steel.

A major factor in weldability is the carbon equivalent, C_{eq}, of the chemical components in a steel. The smaller this value, the better for weldability. Carbon equivalent may be calculated by an equation such as that shown below, in which the unit is weight %:

$$C_{eq} = C + \frac{1}{6}Mn + \frac{1}{24}Si + \frac{1}{40}Ni + \frac{1}{5}Cr + \frac{1}{4}Mo + \frac{1}{14}V$$

High-strength steels tend to have a high carbon equivalent. When it exceeds a certain limit ($C_{eq} = 0.39 - 0.43$), the loss of weldability is compensated by preheating or postheating of the weld zone.

SECTION 2.1 STRUCTURAL STEEL MEMBERS

2.1.1 Single Members and Assembled Members

Steel sections can be classified according to methods of application into products used as single members and products used mainly in combination to form assembled members. Classified by type of section, the main products are flat bars, angles, cut-tees and round bars, as shown in Fig. 2-1. Steel products of these types are rarely used singly as columns or beams, but are combined in a lattice girder or truss to form members such as those shown in Fig. 2-2. The lattice girder, also called the Vierendeel girder, is an unstable structure unless the joints are rigid. It also has little shearing strength compared to the trussed girder, and is liable to deflect greatly. This point should be noted with care.

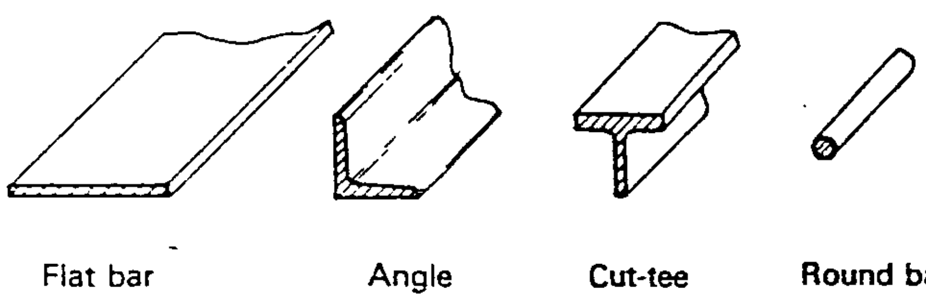


Fig. 2-1. Examples of assembly sections

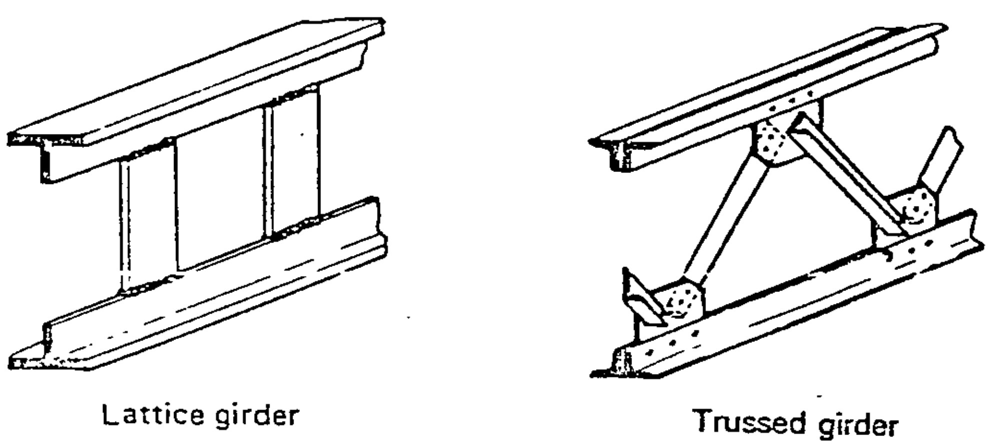


Fig. 2-2. Examples of assembled members

2.1.2 Wide-flange Beams

Wide-flange beams are similar in appearance to I-beams. Fig. 2-3 shows the differences in their section geometries.

(1) The flanges of wide-flange beams have inner and outer surfaces in parallel, while the flanges of I-beams are tapered.

(2) The inside dimensions of wide-flange beams of the same series are fixed, whereas for I-beams the outside dimensions are fixed.

(3) For wide-flange beams there are three series of sections, having width-to-height ratios (B/H ratios) of about 0.5, 0.75 and 1.0, respectively, whereas there is no I-beam section having a B/H ratio of 1.0.

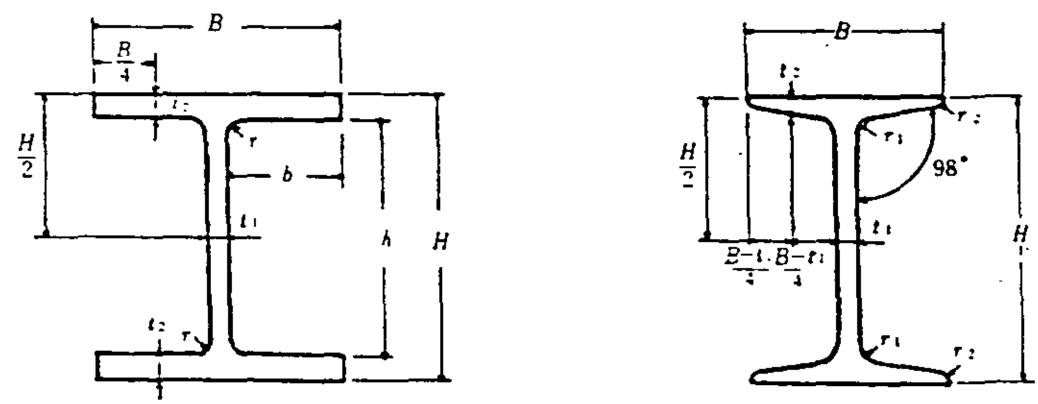


Fig. 2-3. Cross sections of wide-flange beam and I-beam

The size of wide-flange beams is expressed in terms of their sectional dimensions by the following formula:

$$\mathrm{H}-(H)\times(B)\times(t_1)\times(t_2) \qquad (2\text{-}1)$$

where H = height, B = width, t_1 = web thickness and t_2 = flange thickness of the section in mm, as shown in Fig. 2-3. The table of standards for wide-flange beams includes various sections that differ slightly in values of H, B, t_1 and t_2 although the sections are of about the same height. The types of wide-flange beam sections are far more numerous than are those given in the standards for I-beams.

It may seem that there is no regularity in standard dimensions of wide-flange beams. It should be noted, however, that the inside dimension h of the section is invariable for wide-flange beams of a series having almost the same height:

$$h=H-2\times t_2 \qquad (2\text{-}2)$$

For example, the value of h for both $H-294\times 302\times 12\times 12$ and $H-300\times 300\times 10\times 15$ is 270 mm.

The reason why inside flange dimensions are fixed for wide-flange beams of the same series is that the width h of the rolls used in final rolling on universal mills is the same regardless of the shape of the rolled steel. When joining

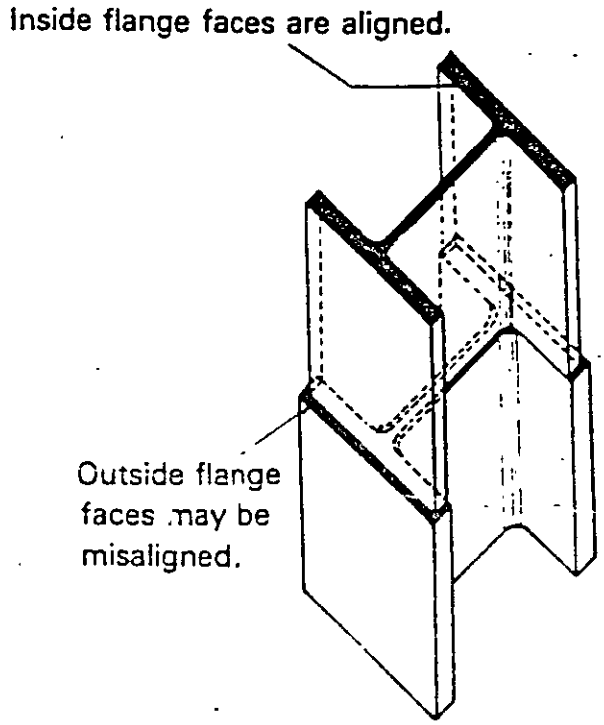


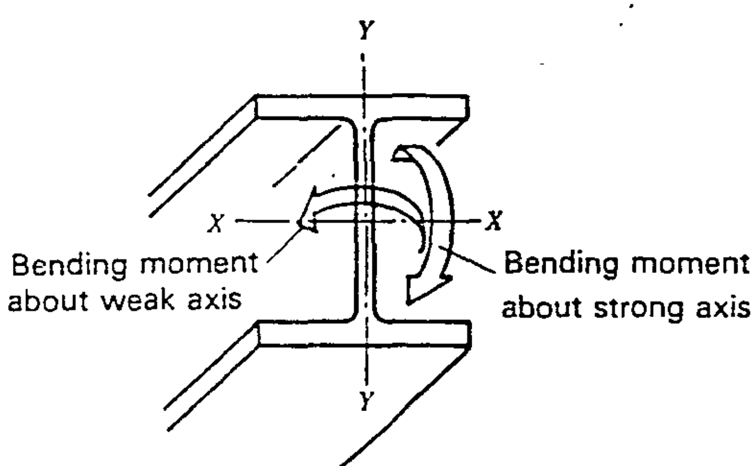
Fig. 2-4. Misalignment in wide-flange beams of the same series

wide-flange beams of the same series, therefore, it should be understood that the outside faces of the flanges may not be aligned even though the inside faces are aligned. This is illustrated in Fig. 2-4.

Anisotropy of wide-flange beams

Wide-flange beams and I-beams of narrow-width series have sections suitable for use as structural members such as beams that are subject to a large bending moment. Wide-width wide-flange beams were developed for use as column members as well. What should be emphasized is that steel products such as wide-flange beams are anisotropic in their resistance to bending — the resistance is not the same in all bending directions.

Fig. 2-5 shows the strong axis and weak axis of the wide-flange beam section. Generally, the ratio of bending rigidity of strong axis to weak axis is approximately 3:1 for the wide-width series and 15:1 for the narrow-width series. Since the



Strong axis = X axis
Weak axis = Y axis

$I_X > I_Y$

I_X : Geometrical moment of inertia about strong axis

I_Y : Geometrical moment of inertia about weak axis

Fig. 2-5. Strong and weak axes of wide-flange beam

beams in a building are normally subject to bending about one axis only, wide-flange beams should be positioned so that they are subject to bending about the strong axis.

In a column, however, there is a bending moment in both the X and Y directions. When wide-flange beams are used as columns, the anisotropy of their bending strength comes into question even if wide-width types are used. In general, it is good practice to position them with their strong axis parallel to the beams exerting a larger bending moment and their weak axis parallel with the beams exerting a smaller bending moment, as shown in Fig. 2-6 (a). The beams attached in parallel with the weak axis of the column are often pin-connected as in Fig. 2-6 (b), to prevent bending moment from being applied in the weak-axis direction. In that case, bracing should be employed against horizontal forces that might act in the longer-span direction. This type of column arrangement is suitable for buildings which have spans of different length in the two directions.

Where wide-flange beams are used as columns in a rigid frame structure with equal spans, other structural considerations are required. Fig. 2-6 (c) shows such a structure, in which the columns alternate in the direction of their strong axis. In some cases, as will be described later, square tubes can also be used.

The foregoing examples indicate how wide-flange beams, though anisotropic, can be used as columns that are subject to bending moments in more than one direction caused by a vertical load. Their anisotropy must also be considered when providing for horizontal forces caused by wind or earthquake.

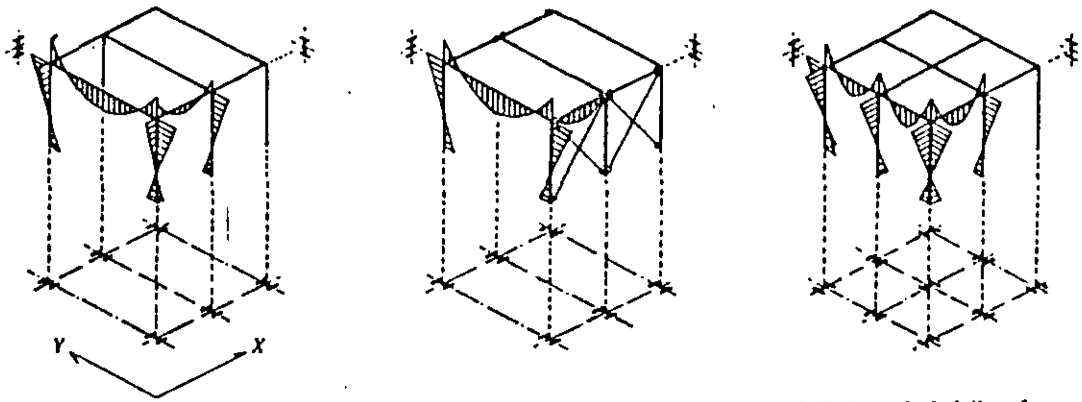
Notwithstanding their anisotropy, wide-flange beams have substantial advantages for structural use:

(1) They can be used as structural members simply by cutting rolled products.

(2) Their untapered flanges are ideal for bolt-joining.

(3) The openness of their section facilitates joint preparation and joining.

For these reasons, the wide-flange beam is the type of steel section used in largest quantities for steel-frame structures.



(a) As the span is larger in the Y direction, the bending moment working on columns is also larger in that direction than in the X direction.

(b) One-way brace frame
No bending occurs in the weak axial direction.

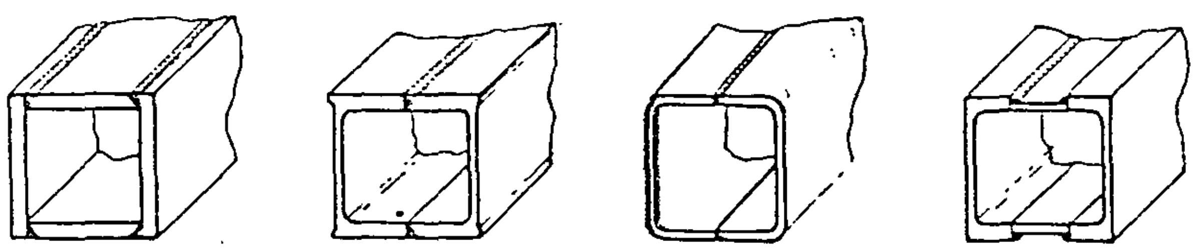
(c) Equal-rigidity frame

Fig. 2-6. Stresses in steel frames and orientation of wide-flange beams

2.1.3 Square Tubes

For the reasons mentioned above, wide-flange beams are basically more suitable for use as structural beams than as columns. Steel products that are free of anisotropy, such as cross-shaped sections and tubes, including centrifugally-cast tubes, are more commonly used today as columns. In general, steel products having a closed section, such as steel tubes, are superior in torsional rigidity and other aspects of section performance to steel products of open section, such as wide-flange beams. Steel tubes can save weight when used as columns subject to compression and bending. They have not been widely adopted for that use, however, since they are more difficult to fabricate than wide-flange beams and their connection with beams — which involves the welding of diaphragms — is also more difficult.

However, Japanese steelmakers have now made available steel tubes of various sectional shapes designed for ease of fabrication into square columns. The use of such square columns is rising for small and medium-size buildings. Examples of square tubes are shown in Fig. 2-7. They include both electrically welded tubes and tubes fabricated from shapes by welding.



(a) (b) (c) (d)

Fig. 2-7. Various types of welded square tube columns

2.1.4 Beams and Columns of Steel-Frame Buildings

1) Various types of beams

In steel-frame buildings, as-rolled wide-flange beams can be used as structural beams when the span is 6 ~ 7 m or less, which is often the case in ordinary rigid-frame structures. For a larger span, structural members having better section performance must be used.

When a bending moment is applied to a beam, as shown in Fig. 2-8, the beam deflects. This creates a compressive stress in the upper part of the beam and a tensile stress in the lower part. These two stresses in the beam balance the applied force. The bending resistance of the beam can be increased by either of two methods:

- ☐ Increase the sectional areas of the upper and lower flanges, which are subject to compressive and tensile stress respectively.
- ☐ Increase the height of the beam.

(1) Castellated (honeycomb) beam

For better beam efficiency, the castellated or honeycomb beam is sometimes used. It is fabricated by first gas-cutting the web of a wide-flange beam in a zigzag line, then welding the halves together at the peaks (Fig. 2-9). The increased height of the beam raises sectional efficiency, and the openings in its web can be utilized for piping or ducts.

(2) Trussed girder

Whereas the bending strength of a castellated beam is raised by an increase in beam height, that of a trussed girder (Fig. 2-2) is raised by use of steel angles as diagonal members. Where a trussed girder will be subject to a large bending moment, steel sections such as wide-flange beams, pipes and square tubes are used as chord members (upper and lower horizontal members) to increase the sectional area of the chords. In large trussed girders, the load may act on places other than the joints. Thus the effects of bending moment cannot be ignored, even for trussed frame members, and care must be taken to prevent buckling of members by compression (Fig. 2-10).

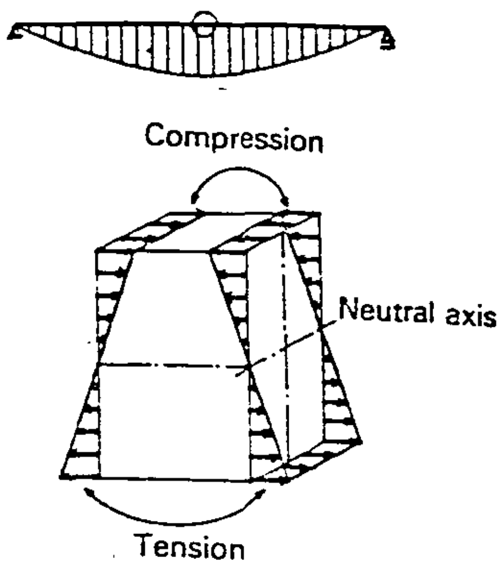


Fig. 2-8. Distribution of bending moment and stress on beam

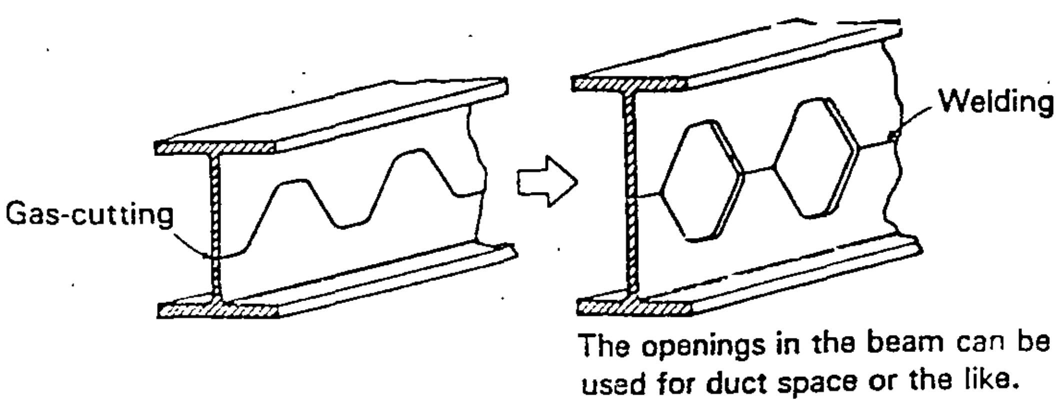
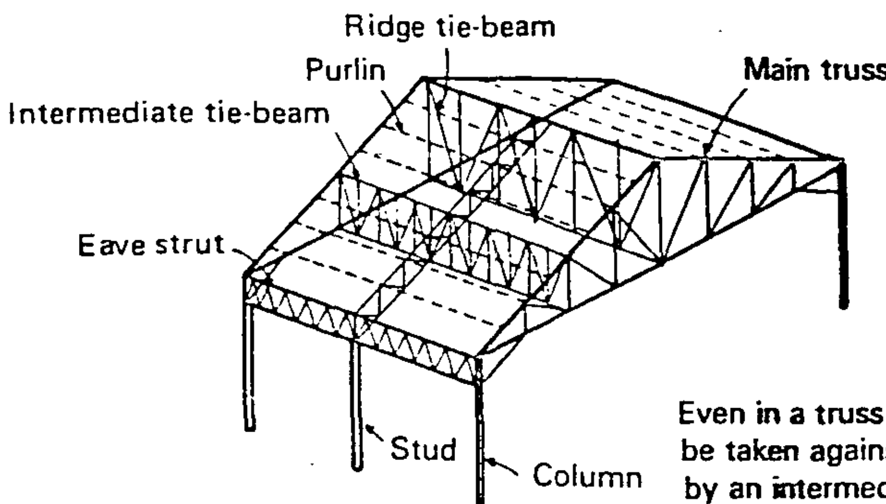
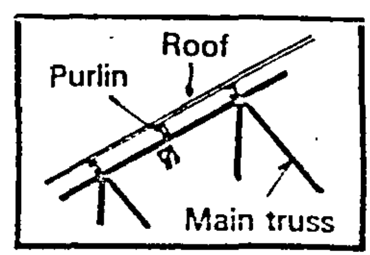


Fig. 2-9. Manufacture of castellated beam





Even in a truss structure, precautions must be taken against a bending moment caused by an intermediate load acting on the princip rafter member (upper chord member).

Fig. 2-10. Bending stress in truss chord member

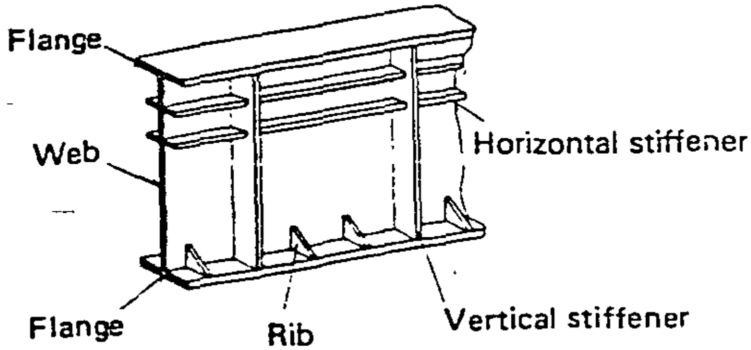


Fig. 2-11. Plate girder and stiffeners

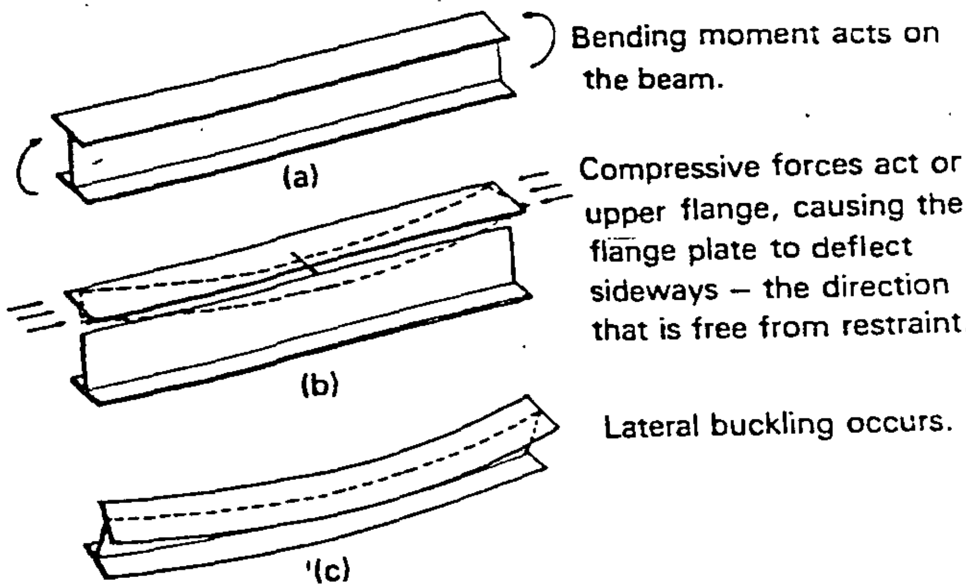
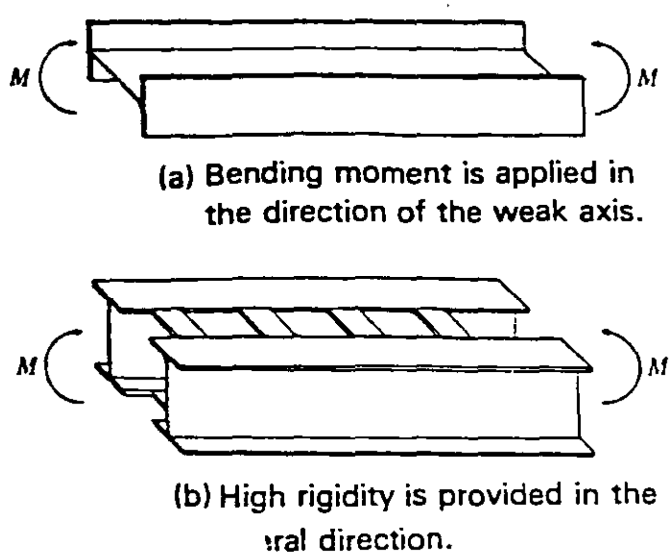


Fig. 2-12.Lateral buckling



..y. 2-13. Cases where lateral buckling of the beam is avoided

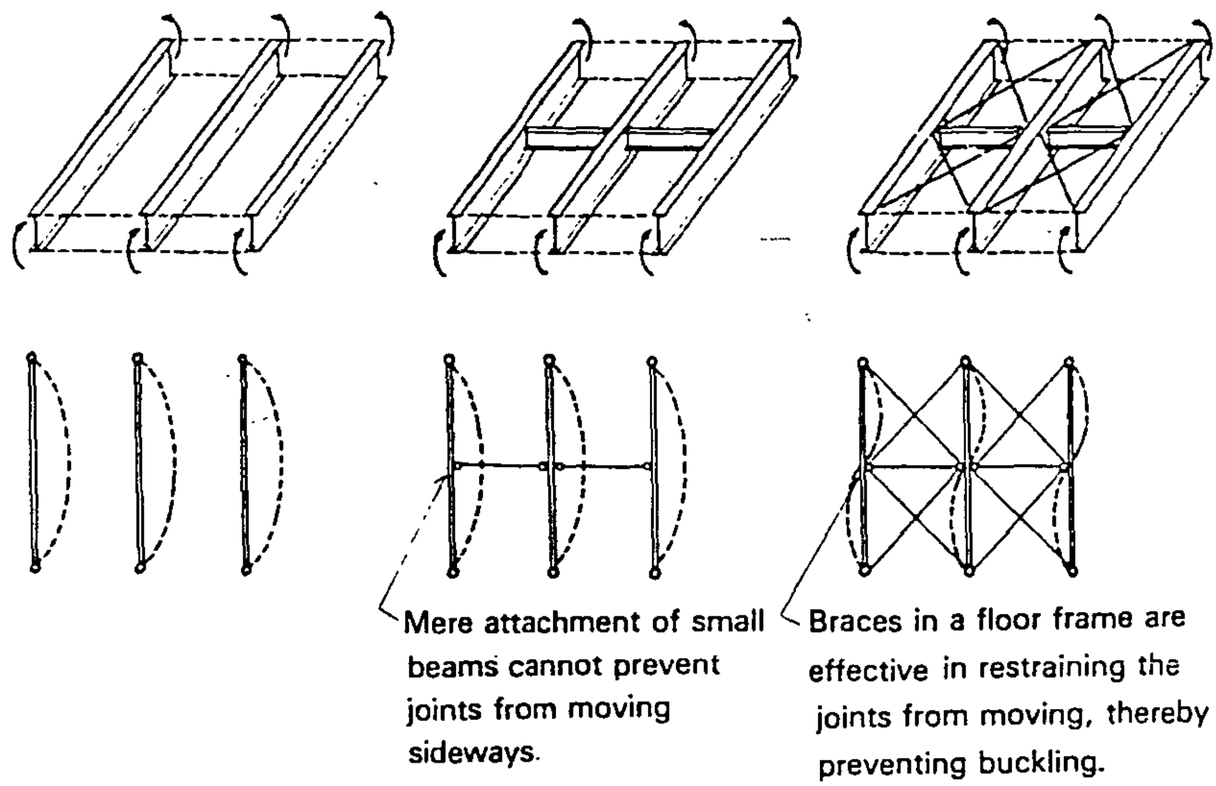


Fig. 2-14. Floor framing and beam stiffening.

(3) Plate girder

The plate girder is a type of solid-web beam that is usually fabricated by welding together steel sections of large height. The thickness of a plate girder is small compared to its section height, and buckling of the web, in particular, is likely to be a problem. This is avoided by attaching stif- ers vertically and horizontally on the web, as shown in Fig. 2-11.

2) Buckling by bending

The most critical type of failure for beams subjected to bending is lateral buckling. Normally, buckling is associated with compression members such as columns, and the expression "buckling by bending" may thus seem strange. Fig. 12 helps to make the concept clear. When a bending moment is applied to a steel section used as a beam, a compressive stress is produced in the upper flange. This flange may be regarded approximately as a kind of flat plate under compression, as indicated in Fig. 2-12 (b). An axial force proportional to the bending moment is exerted on every part of the flat plate's cross section. The flange tends to deflect sideways — in the lateral direction of the beam, where there is no restraint by the web. The beam is thus deformed as in Fig. 2-12 (c). This is lateral buckling.

It is not a problem for every beam. Lateral buckling will not occur in a wide-flange beam, for example, when the bending moment is applied to the weak axis, or when two members are connected as in Fig. 2-13 so as to increase rigidity. However, it should be considered that the bending strength of a steel section used singly as a beam is determined by the load of lateral buckling.

To prevent lateral buckling in steel-frame buildings, lateral stiffeners must be attached at adequate intervals. In floor framing, large beams are restrained from lateral deflection since they are attached to small beams between them, as shown in Fig. 2-14. Braces are also attached, to prevent the intermediate stiffening points of the large beams from moving sideways. In this way, members subject to a bending moment in one direction are designed so that the force is transmitted three-dimensionally.

3) Buckling by compression

The buckling of compression members is another type of buckling failure that must be guarded against. As the compressive load on a column increases, at a certain load P_{cr} the column suddenly buckles and cannot support a larger load (Fig. 2-15). The buckling load P_{cr} can be defined in terms of the member's rigidity and dimensions as

$$P_{cr} = \pi^2 EI / l^2 \qquad (2\text{-}3)$$

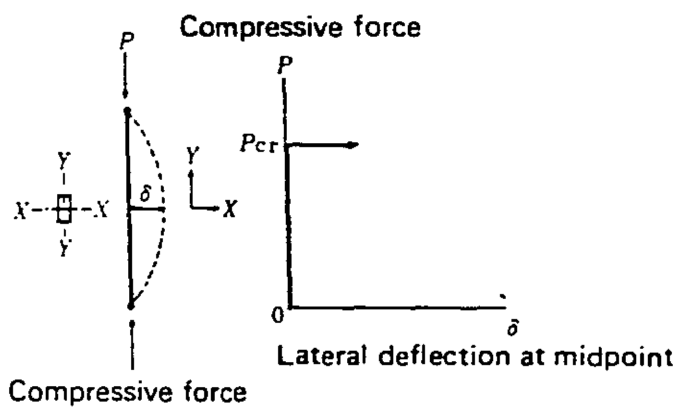


Fig. 2-15. Diagram of buckling

where EI is bending rigidity and l is the length of the member. Thus the buckling load P_{cr} is directly proportional to the member's rigidity and inversely proportional to the square of its length.

In an ordinary steel frame, however, the only members subject solely to compression, as in Fig. 2-16 (a), are special members such as the diagonals in a truss and thin braces. Column members in a rigid-frame structure are subject to a bending moment M as well as to the axial compressive force N (Fig. 2-16 (b)). They cannot be treated simply as compression members. In designing members on which both a bending moment and axial compression are imposed, both the compressive buckling load and the lateral buckling load must be considered.

Stiffening is often provided to restrain columns from buckling. In this case, the design load for stiffener ⓑ in Fig. 2-17 (a) should be 2% or more of the axial force acting on the column. This load is determined by the lateral force F required for the lateral stiffener to change the buckling wave form of the column from a half-wave to a full wave. Unless one end of the stiffener can transmit force to a member with adequate rigidity, the stiffener cannot prevent buckling, as is also the case with buckling by bending.

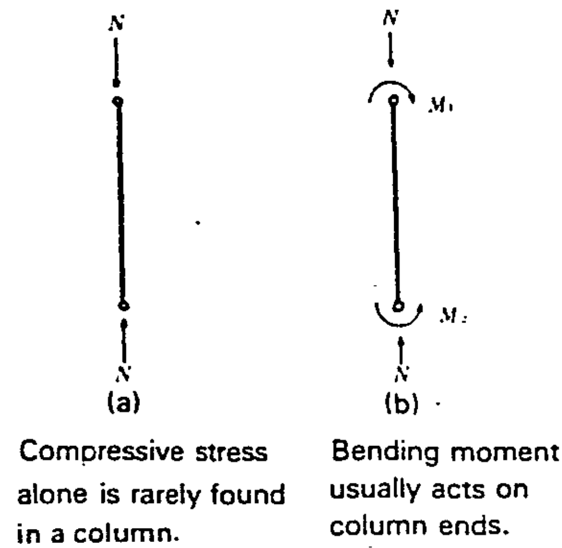


Fig. 2-16. Stress acting on a column

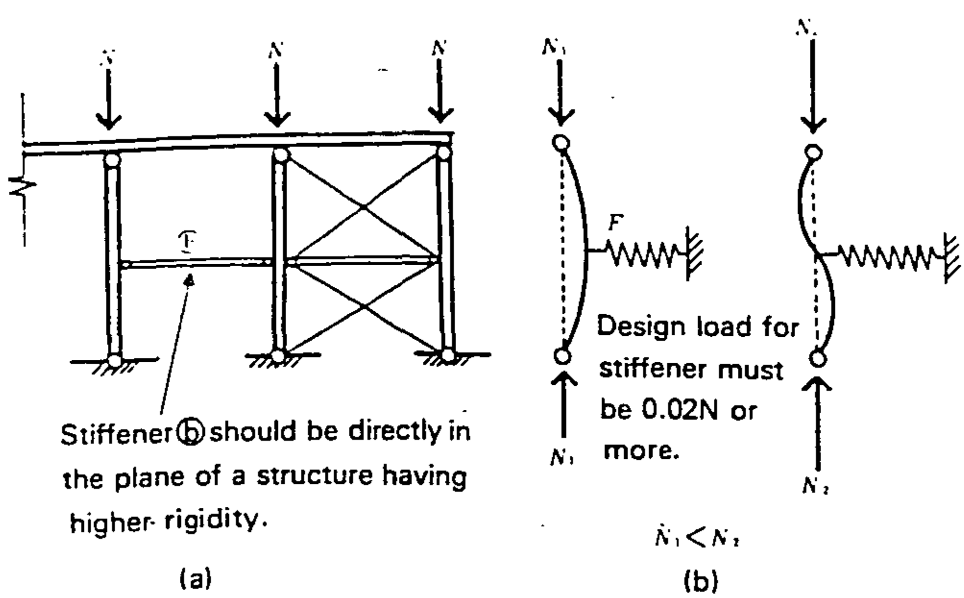


Fig. 2-17. Concept of stiffeners against buckling of columns

2.2.1 Joining by High-strength Bolts —

Methods of joining with bolts are classified into three types as shown in Fig. 2-18.

1) Shearing joint

The stress-transmission mechanism of a shearing joint is illustrated in Fig. 2-18 (a). Sectional force N acting on the joined members is transmitted by contact pressure (bearing pressure) between the side of the bolt and the bolt hole. A shearing force is thus applied to the section S of the bolt. Accordingly, the yield strength of a shearing joint should be either the shearing strength of the bolt, determined by its diameter, or the bearing strength of the joined members, whichever is smaller in value. This is only true if there is no clearance between bolt and bolt hole. In most cases, however, some clearance is unavoidable.

2) **Friction joint**

Fig. 2-18 (b) shows the concept of the friction joint. As in the shearing joint, the sectional force N is transmitted at right angles to the axis of the bolt, but its stress-transmission mechanism is completely different from that of the shearing joint. In the friction joint, a large clamping force is applied to the bolt, whereby a compressive stress is produced between the joined members. The sectional force N is thus countered by the frictional resistance of the surfaces of the joined members. Generally, the relation between the forces shown in Fig. 2-18 (d) is

$$N_s = k \cdot T \qquad (2\text{-}4)$$

where N_s = the sliding load, k = the slip coefficient and T = the axial force of the bolt.

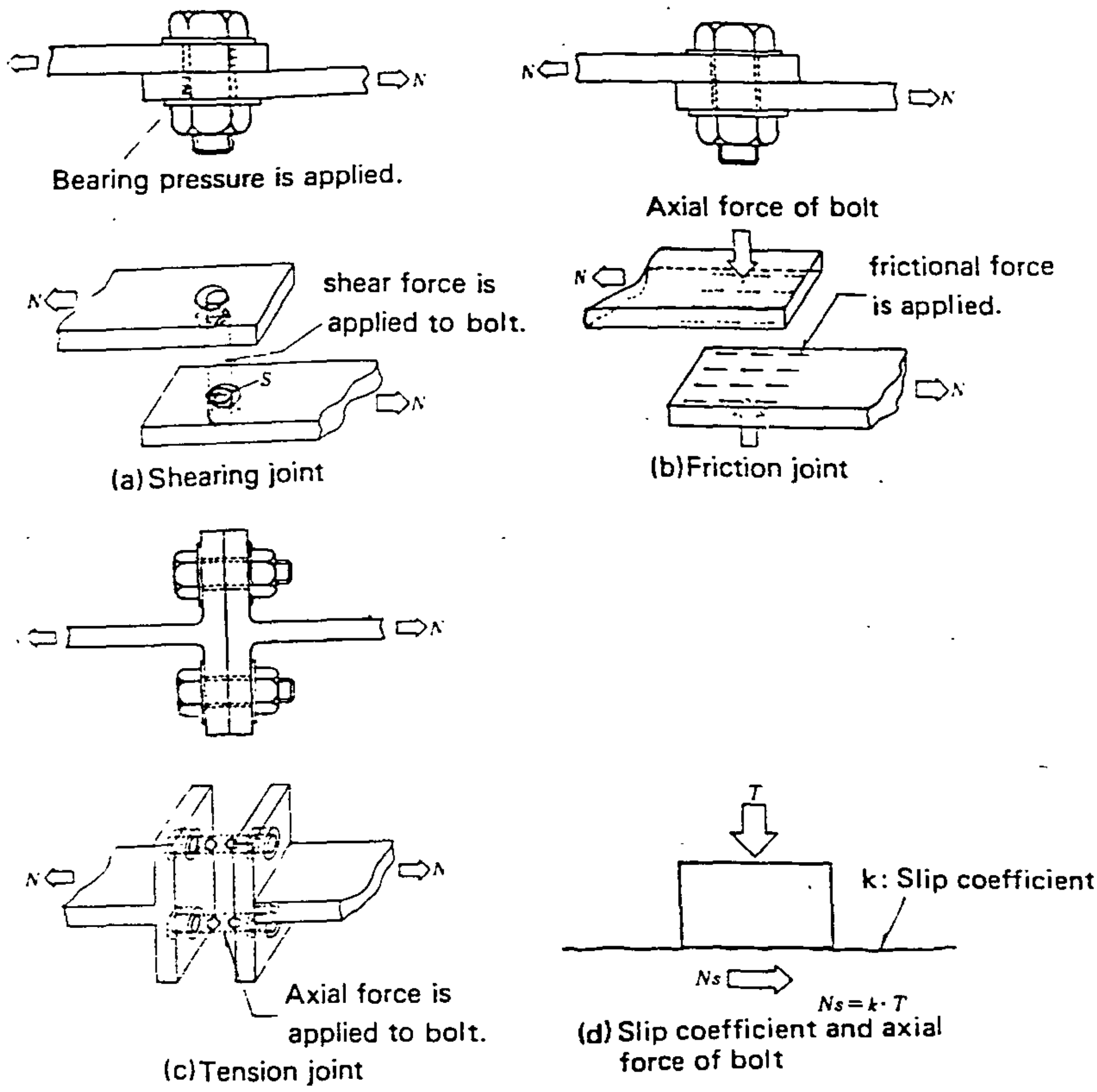


Fig. 2-18. High-strength bolt joining

Thus the strength of a friction joint depends on the axial force exerted by the bolt and the slip coefficient between the joined members. Bolts used for friction joints are heat-treated so as to withstand large axial forces. Types of joints that allow easy control of axial force T in the bolts are now coming into wider use. An example is the use of special high-strength bolts shown in Fig. 2-19, in which a notch separates the bolt shaft from an end-portion called the pintail. Breaking off of the pintail during tightening indicates that the specified axial stress has been attained. Since the slip coefficient k is also important in a friction joint, ample care should be taken to ensure a value of 0.45 or higher in the usual case. This is why the black skin of the steel surfaces being joined should be removed and red rust is allowed to develop.

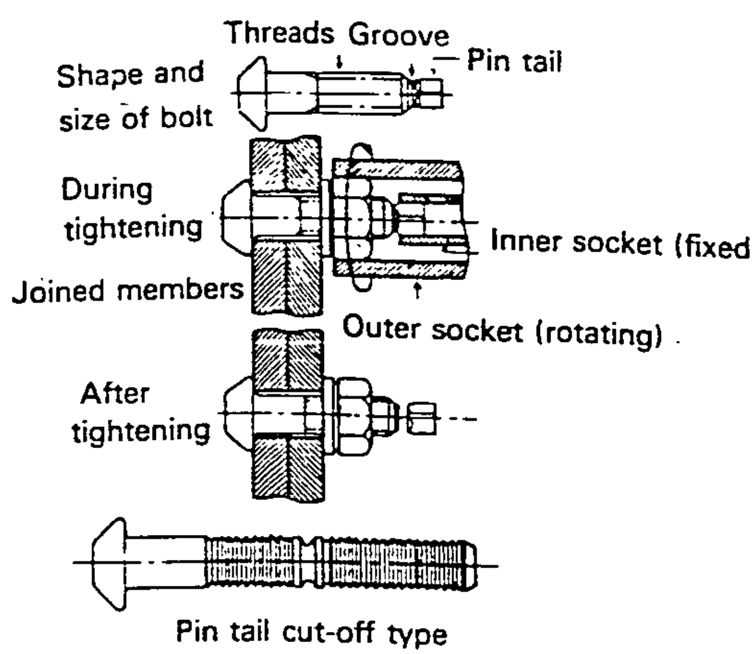


Fig. 2-19. Principle of special high-strength bolt

Although the stress-transmission mechanism of the friction joint is different from that of the shearing joint, the number of bolts required, n, can be calculated as

$$n = N / Ra \qquad (2\text{-}5)$$

where R_a = allowable shearing strength and a standard slip coefficient of 0.45 is used.

3) Tension joint

Fig. 2-18 (c) shows the concept of the tension joint, another way in which high-strength bolts can be applied. The tension joint differs from the preceding two types in that the sectional force N is transmitted through the bolts as axial stress. If a tensile stress is created in the bolt in advance, then the joined members do not separate until that tensile stress is offset by axial force N. This makes it possible to design joints having high rigidity. It should be carefully noted, however, that if the bolt arrangement and the thickness of the joined members are not adequate, the bolts are subjected to additional bending moment which greatly reduces the strength of the joint, as shown in Fig. 2-20.

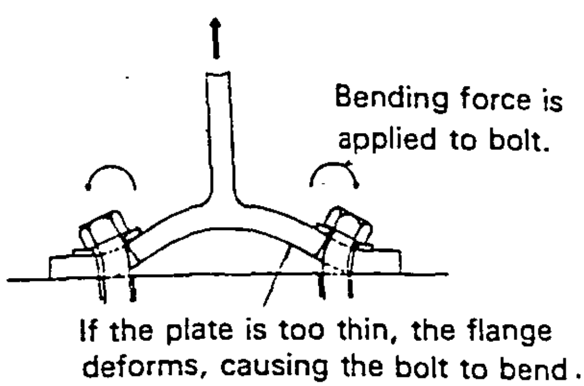


Fig. 2-20. Development of prying action force in tension joint

2.2.2 Joining by Welding

1) Basic points about welding

Brazing (hard soldering) is a joining method similar to welding. In both methods, an easily meltable metal, solder or welding rod, is filled between two metal members and allowed to harden, joining the members together. The fundamental difference is that in brazing the joined members (base metal) themselves are not melted, while in welding the base metal at the joint is melted and united with molten filler metal.

Thus welding is not a bonding operation, like the joining of glass plates with epoxy resin. Rather, it is nothing less than a process of fusing metals. Table 2-1 classifies the various welding methods used in the construction of a steel-frame building. An electric arc is generally used as the heat source for welding. Gas welding is rarely used.

In arc welding, as shown in Fig. 2-21, an electric arc is struck between the base metal and a welding rod or electrode. The heat of the arc fuses the metals. If molten metal at such high temperature is in contact with air, rapid oxidation occurs. To avoid that problem, a number of methods have been devised to shield the arc from contact with air. These methods are classified into three groups, according to the type of welding equipment used: manual arc welding, semiautomatic welding and automatic welding.

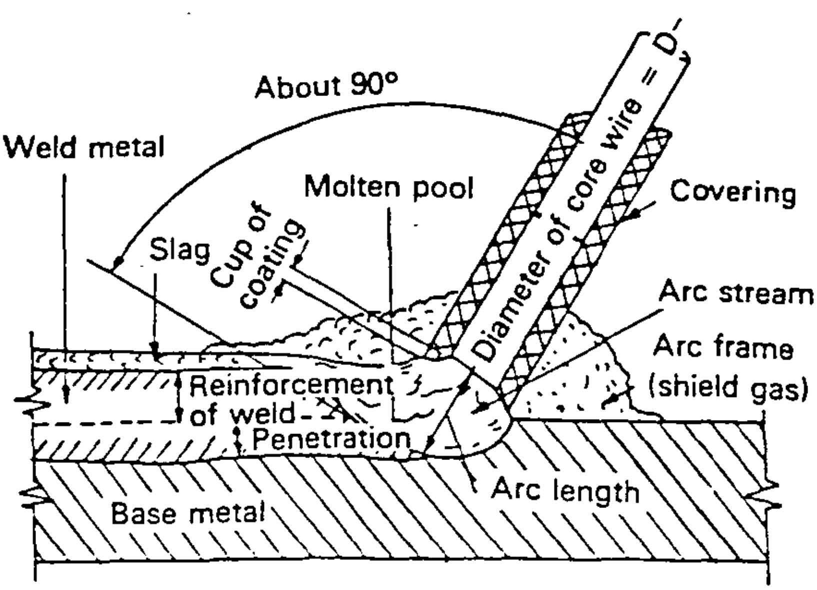
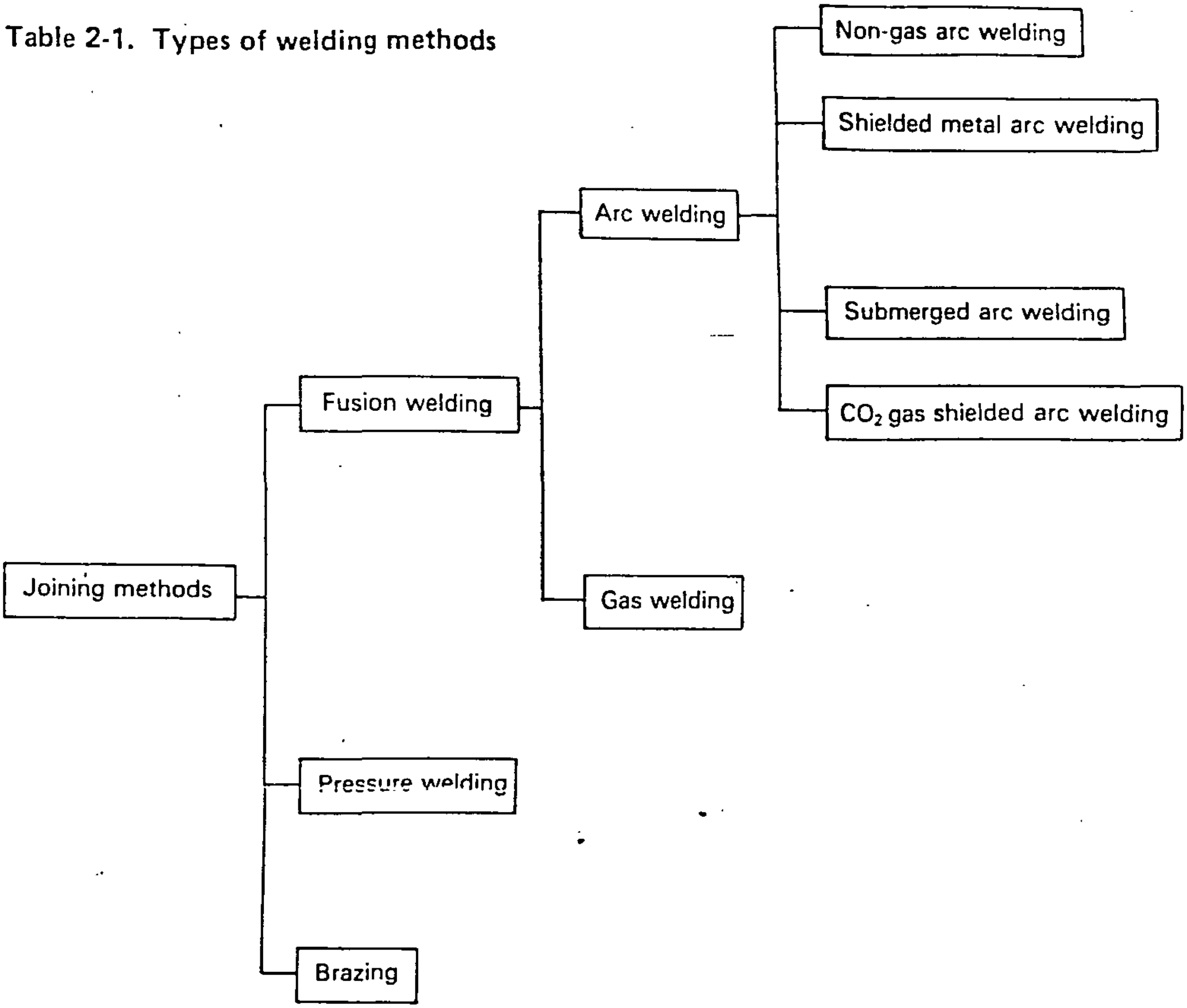


Fig. 2-21. Welding mechanism

Table 2-1. Types of welding methods



(1) Manual arc welding

A covered electrode consisting of a core wire coated with flux is used in manual arc welding. The flux coating melts under the heat of the arc and produces a shielding gas that protects the molten metal and improves its weldability. In most cases, it is necessary to change welding rods during the welding operation. This method is widely used for welding of intricate portions.

(2) Semiautomatic welding

This method involves the use of a machine equipped with an automatic device for feeding core wire in coil form to the welding torch, eliminating the need to change welding rods manually. The torch itself can be moved and operated manually. Semiautomatic welding methods are divided broadly into CO_2 gas arc welding, which uses carbon dioxide gas as a shielding gas, and non-gas arc welding, in which the shielding gas is generated by the flux on the core wire. Most factory welding today is done by CO_2 gas arc welding, while non-gas arc welding is mainly employed in field welding since it is little affected by wind.

(3) Automatic welding

Automatic welding usually means submerged arc welding. A compound similar in composition to the flux is applied to the weld area in advance, and the core wire is submerged in this compound during welding. This is generally called union melt welding. Automatic welding features high productivity and is suitable for welding thick plates. It is widely used in factory welding of long pieces such as plate girders and column members.

2) Fillet welds

The types of welded joints most commonly used in steel-frame buildings are fillet welds and butt welds. Representative examples of fillet welds are shown in Fig. 2-22. At a fillet weld, the base metals may not be united completely, and this fact must be taken into account. Fig. 2-23 indicates the mechanism of stress transmission at a fillet weld. On side A of the weld, a tensile stress acts on base metal 1 and a shearing stress acts on base metal 2 when the joint is subjected to a tensile force. On side B of the weld, however, a shearing stress acts on both base metals 1 and 2, and is transmitted by the weld.

The mechanism of stress transmission is thus fairly complicated for a fillet weld, since it varies with the direction of the weld line, and the role of shearing stress is important. When a fillet-weld joint is broken in tension fracture, the break occurs across what is called the throat, as shown in Fig. 2-24. Throat thickness is taken into consideration as a reference value in the design of fillet welds. It is calculated with reference to unit shearing stress, but not to welding direction. For a joint like that shown in Fig. 2-23, required weld length l can be calculated from the equation

$$F = t \times l \times F_s \qquad (2\text{-}6)$$

where F = force to be transmitted (tons); t = throat thickness (cm); and F_S = allowable unit shearing stress (tons/cm^2).

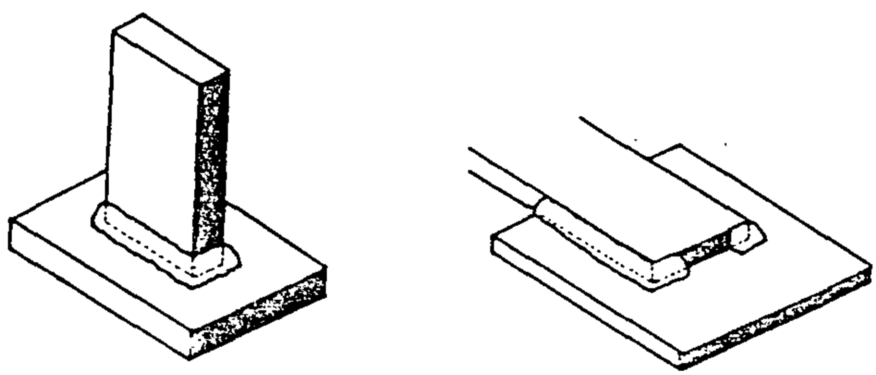


Fig. 2-22. Examples of fillet welds

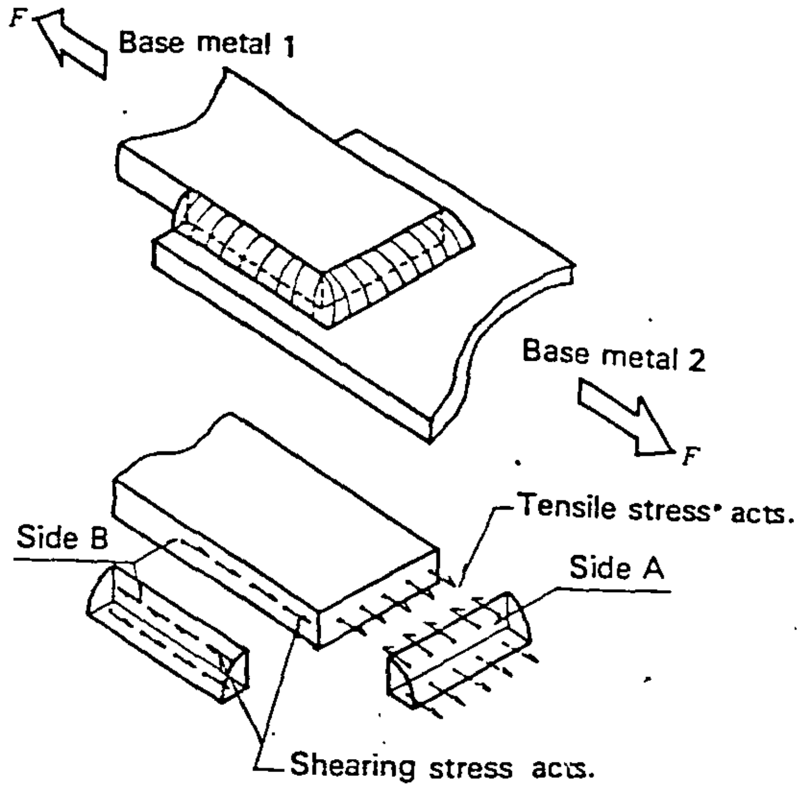


Fig. 2-23. Stress transmission mechanism at a fillet weld

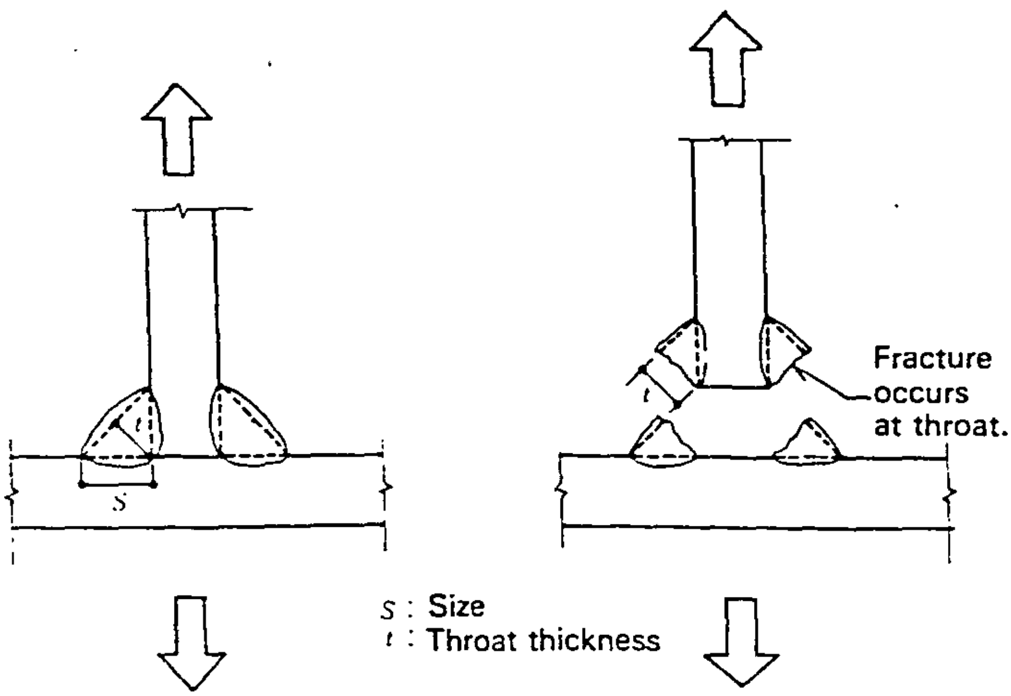


Fig. 2-24. Fillet weld breakage

3) Butt welds

When plates are to be butt welded, edge preparation called grooving is first carried out on the end faces of the plates. The result can be seen in Fig. 2-25. Edge preparation ensures full penetration of the molten metal through the cross section during welding.

Tensile stress is transmitted directly through a butt weld, as indicated in Fig. 2-26. Hence butt welds have a higher joint efficiency than do fillet welds. For this reason butt welding is generally used for the welding of main steel-frame members.

Corresponding to Equation 2-6 for fillet welds, the equation for the design of a butt welded joint is

$$F = t \times l \times F_t \quad ; \quad F_t = \sqrt{3} \times F_s \qquad (2\text{-}7)$$

where F_t = allowable unit tensile stress (tons/cm²). The design yield strength of the butt weld is 1.7 times that of the fillet weld because of the difference in mechanism of stress transmission — tensile stress in the butt weld, shearing stress in the fillet weld.

The types of grooves commonly used in edge preparation for building construction are given in Table 2-2.

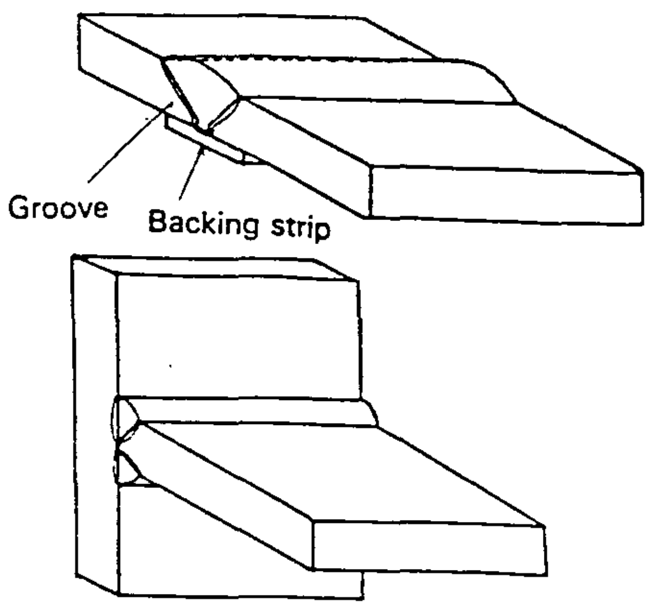


Fig. 2-25. Examples of butt welds

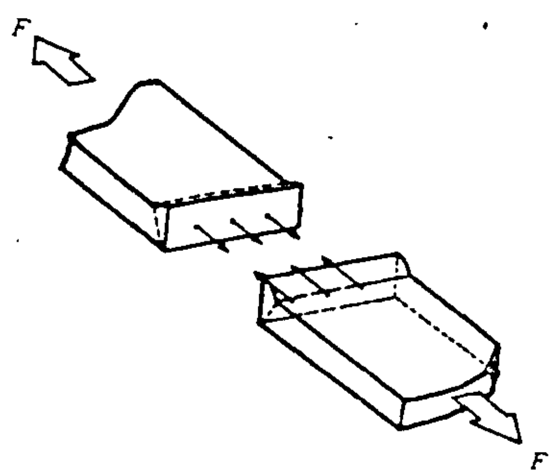
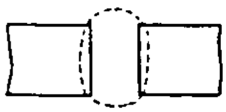
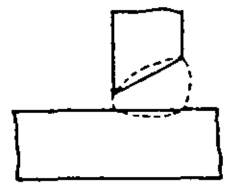
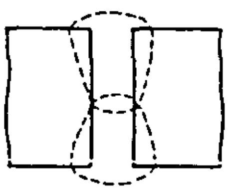
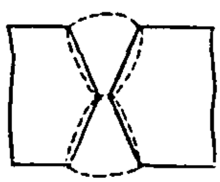
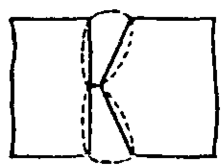
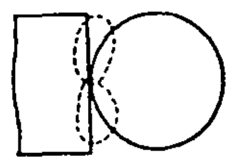


Fig. 2-26. Stress transmission mechanism at a butt weld

Table 2-2. Types of grooves

Single grooves	Shape	 Square groove	 V grove	 V groove	 V groove
	Application	Narrow-groove welding of steel sheets and plates	Plate joining, beam to beam	Plate joining, column to column	Column assembly, column to beam
Double grooves	Shape	 Square groove	 X groove	 K groove	 Flare K groove
	Application	Heavy plate joining	Heavy plate joining, beam to beam	Heavy plate joining, beam to beam, column to beam	Reinforcing bar to plate

2.2.3 Welding Details

1) End tab

Weld defects are very likely to occur near the start and end points of a weld bead, since the arc is electrically unstable near those points. Welding defects must be avoided, since they can cause serious structural failure.

Use of end tabs is a measure to prevent weld defects at these locations. The end tab is a piece of plate with the same groove shape as the base metal. When it is placed on top of the backing strip, welding can be started from the end tab, allowing the weld length to be longer than the base metal (Fig. 2-27). Welding flaws caused by an unstable arc are thus on the end tab, which is cut off after the joint is welded.

2) Scallop

Another problem to be avoided in welding is the intersection of weld lines. If two weld lines cross, heat is applied twice at the intersection, greatly changing the properties of the steel and raising the incidence of weld defects. Such an intersection is avoided by making a notch, called a scallop, in one of the base metals being joined, as shown in Fig. 2-27. When making a large scallop, care should be taken not to reduce the section area of the base metal unduly.

3) Boxing welding

For a fillet weld, an effect similar to that of the end tab for butt welds is achieved by lengthening the start and end points of the weld by more than twice the size S of the weld. This is called boxing welding, and is shown in Fig. 2-28. When this technique is used, it should be emphasized that the effective length of the fillet weld is not the actual length of the weld line, but the actual length minus $2S$ (twice the size of the weld).

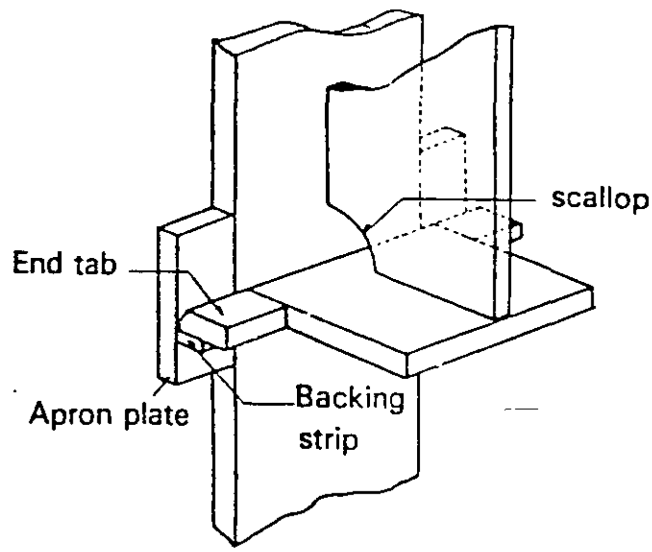


Fig. 2-27. End tab and scallop

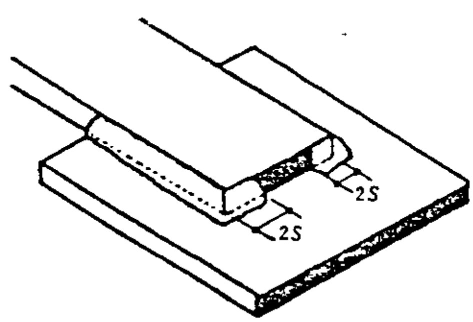


Fig. 2-28. Boxing welding

2.2.4 Quality Control of Welds

During welding, there is a high possibility that complex thermal effects will cause changes in steel properties, resulting in weld defects. It cannot be overemphasized that poor-quality welding prevents steel members from exhibiting the expected yield strength, and in some cases the result may be the collapse of the entire building. Therefore, inspection of welds is a vital part of steel-frame fabrication.

Weld defects are broadly divided into three kinds:

(1) Insufficient dimensions (weld length too short, size too small)
(2) Surface defects (pits, surface cracks)
(3) Internal defects

Surface defects and insufficient dimensions can be detected visually. For internal defects, ultrasonic flaw detection is widely used. Fig. 2-29 illustrates the principle of this method. Ultrasonic flaw detection is basically the same as sonic fish detection. When a sound wave is emitted from the bottom of a boat, it is reflected by the seabed and this "echo" can be detected aboardship. The distance L_1 between the boat and the seabed is expressed by

$$L_1 = v \cdot (t_2 - t_1)\ /2 \qquad (2\text{-}8)$$

where v = the speed of sound, t_1 = time of sound wave emission and t_2 = time of reflected sound detection.

If a shoal of fish passes at a certain level L_2 beneath the boat, a sound wave reflected by the fish is received at time t_2, earlier than t_2, and

$$L_2 = v \cdot (t_2' - t_1)\ /2 \qquad (2\text{-}9)$$

The ultrasonic flaw detection method for welds is based on exactly the same principle. The "shoal of fish" corresponds to internal defects such as cracks and the "seabed" corresponds to the bottom face of the steel plate. In the steel frame of a building, however, welded joints of complex shape such as that shown in Fig. 2-30 are used more often than simple joints of flat plates. Therefore, the skew angle-beam technique, in which the ultrasonic waves are directed diagonally rather than perpendicularly, is the standard nondestructive inspection method for welds. Fig. 2-31 shows an ultrasonic inspection in progress.

2.2.5 Welding and Residual Strain

When two plates are joined by a center weld, as shown in Fig. 2-32, the welded surface will warp due to thermal shrinkage. Ordinarily, this is avoided by allowing for such shrinkage when plates are being fabricated. Fig. 2-33 shows an example of this technique, which is called pre-strain. Another method, used after welding, is to deform the weldment forcibly in a press, or to heat it locally to correct the shape. In field welding, various methods are used to ensure dimensional accuracy, such as following the work sequence indicated in Fig. 2-34.

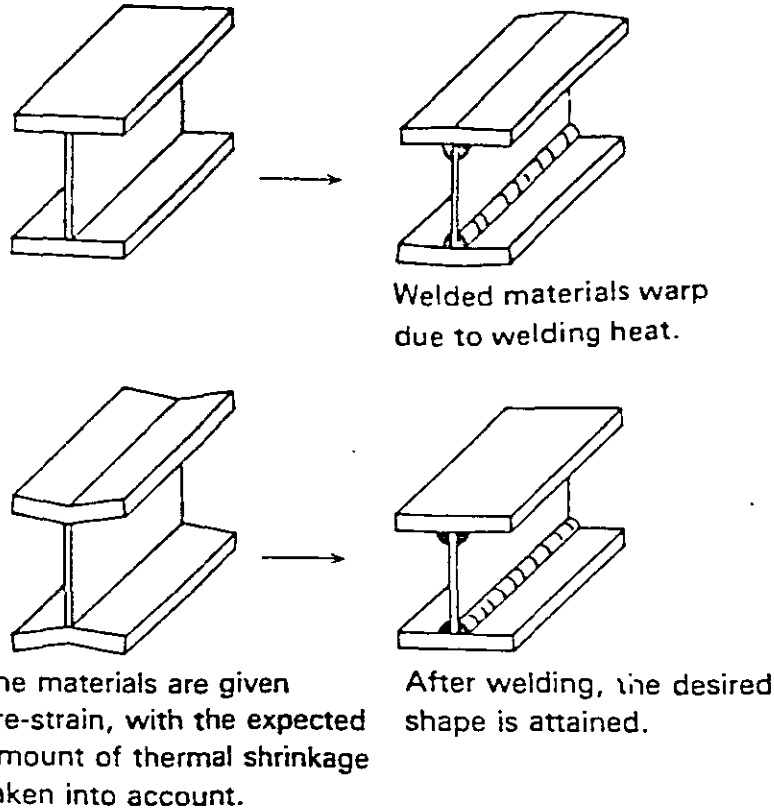
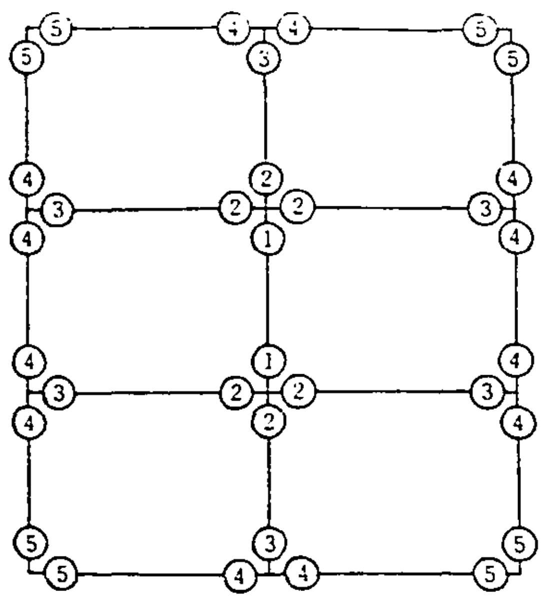


Fig. 2-33. Effects of pre-strain



In joining beams in steel-frame structure, the welding sequence follows the numbers shown in the plan. This is most effective in reducing assembling deviation to the minimum.

Fig. 2-34. Field welding sequence

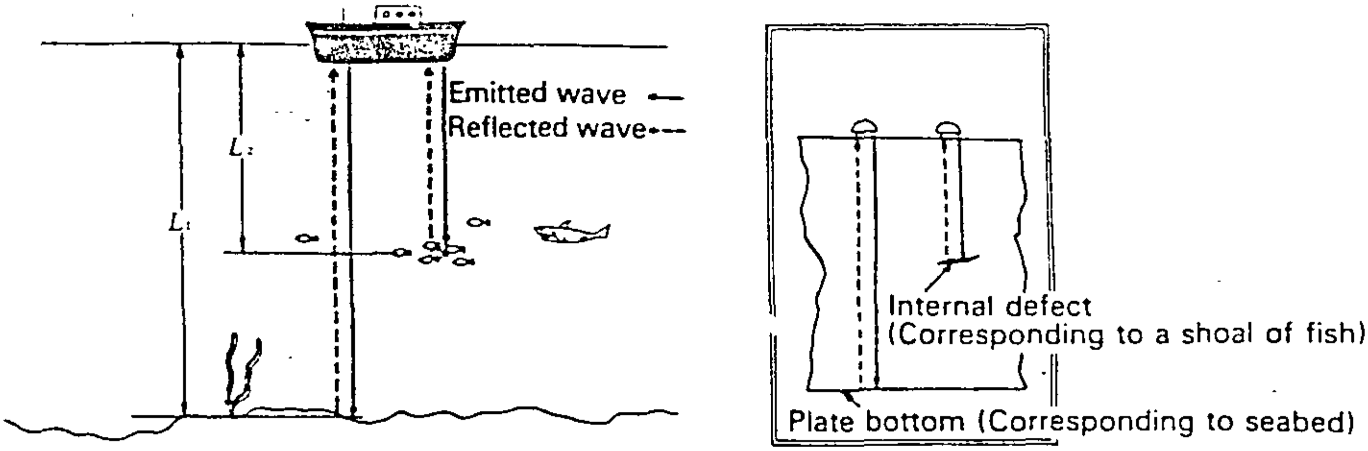


Fig. 2-29. Fish detection and ultrasonic flaw detection

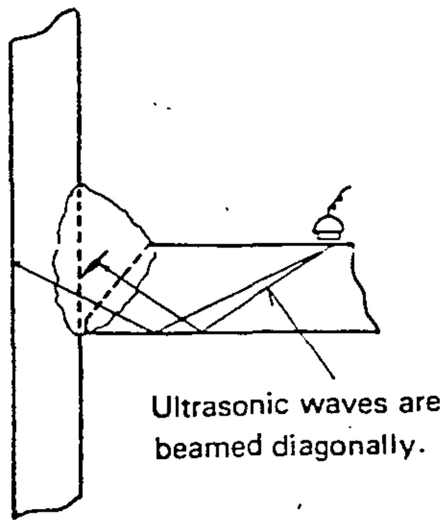


Fig. 2-30. Angle-beam inspection method

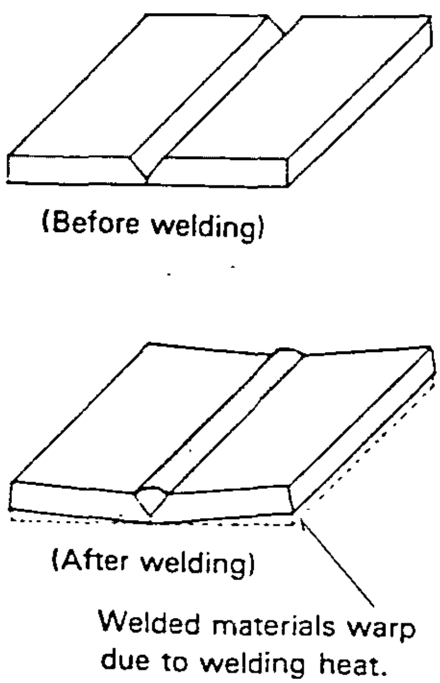


Fig. 2-32. Welding and thermal strain

SECTION 2.3 DESIGN OF CONNECTIONS

2.3.1 Basic Concepts of the Joining of Members

There are two basic concepts for designing connections between structural members: the existing-stress method and the total-strength method.

In the existing-stress method, connections are designed on the basis of the sectional forces (M, N, Q) acting on the members; these forces are obtained by stress calculation. In the total-strength method, the aim is to design connections with a yield strength equal to or greater than that of the members to be joined.

Given the loading condition shown in Fig. 2-35, let us compare the designs of a high-strength bolt friction joint based on these two methods.

1) Example by the existing-stress method

If high-strength bolts of M16, F10T are used in two-side friction, the long-term allowable shearing force R is 6.03 tons per bolt ("Design Standard for Steel Structures and Commentary", Architectural Institute of Japan). The required number of bolts n for the long-term design load N = 10 tons is determined by the following equation:

$$n = N / R = 10.0 / 6.03 = 1.66 \rightarrow 2 \qquad (2\text{-}10)$$

In this case, two bolts are required (Fig. 2-35 (a)).

2) Example by the total-strength method

If the allowable tensile stress f_t = 1.6 tons/cm² is transmitted to the effective sectional area A (actual sectional area minus the sectional area across the diameter of the bolt hole), the axial force N' for total-strength design is

$$N' = A \times f_t = 1.6 \times (8.0 - 1.75) \times 1.6 = 16.0(\mathrm{t}) \qquad (2\text{-}11)$$

The required number of bolts n' is

$$n' = N'/R = 2.65 \rightarrow 3 \qquad (2\text{-}12)$$

Thus three bolts are required (Fig. 2-35 (b)).

As is evident from these examples, the two methods lead to different design results. The key features of each method are summarized in Table 2-3. The choice between them must be made on the basis of cost and the importance of the member. An important point to remember is that transmission of existing stress alone is not sufficient. It is also necessary to provide structural continuity with respect to rigidity and yield strength. It is thus desirable to design joints with sufficient reserve capacity.

Table 2-3. Key features of existing-stress and total-strength methods

	Existing-stress design	Total-strength design
Use of steel materials	Small amount	Great amount
Standardization	Difficult	Easy
Yield strength	Inferior	Superior

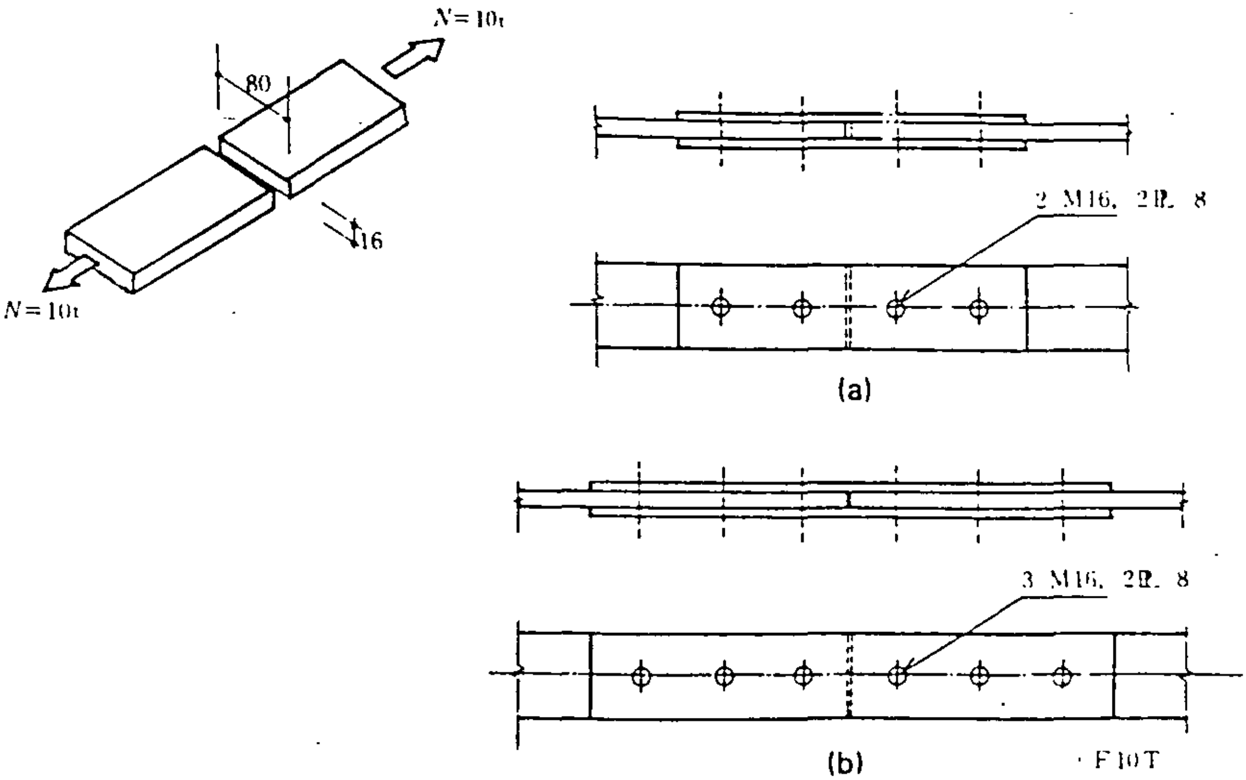


Fig. 2-35. Existing-stress design and total-strength design

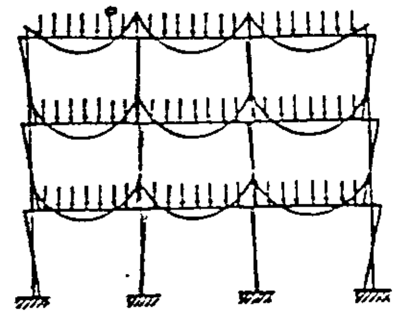
2.3.2 Joints in Columns and Beams

1) Positions of Joints

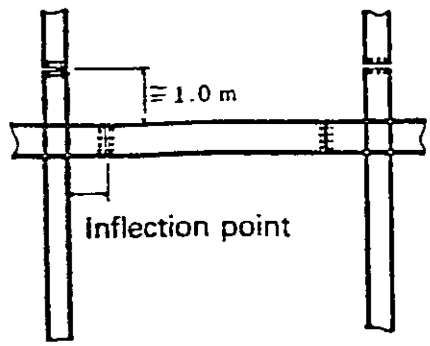
The criterion for deciding the positions of joints in columns and beams includes three conditions:

(1) Existing stress in members
(2) Accessibility
(3) Provision for transport

In principle, joints should be located at positions where existing stress is small. To this end, it is desirable to choose the inflection points where the value of the bending moment is zero in the column or beam. For columns, joints are often provided at a height of about 1 m from the upper end of the beam so as to secure better accessibility (Fig. 2-33). Conditions of transport may also impose restrictions on the length of steel-frame members. Fig. 2-37 shows examples of standard dimensions of a truck and a trailer. Members of a frame should be in lengths which can be accommodated in the bed of a truck. Therefore, joint positions may well be chosen by taking a few stories as a tier so that column members are several stories high. Care, however, should be taken since transport restrictions may vary considerably with the roads used and the site condition.



Distribution of bending moment in rigid frame



Position of joints

Fig. 2-36. Position of joints in columns and beams

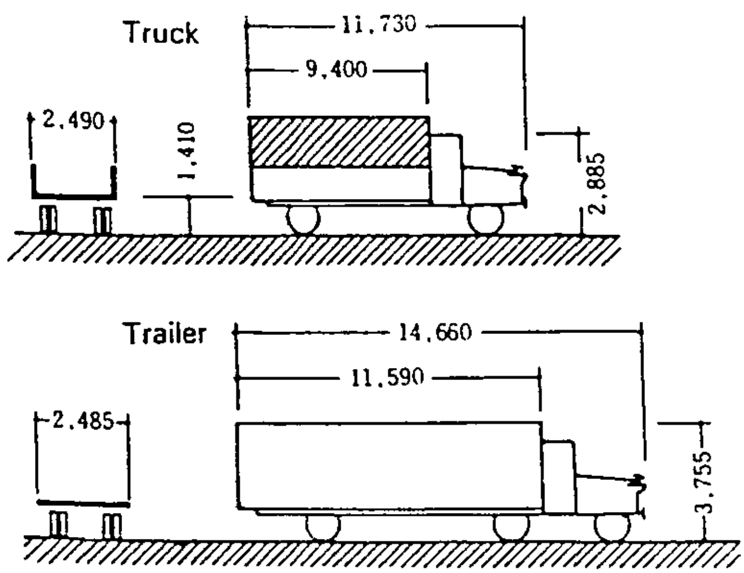


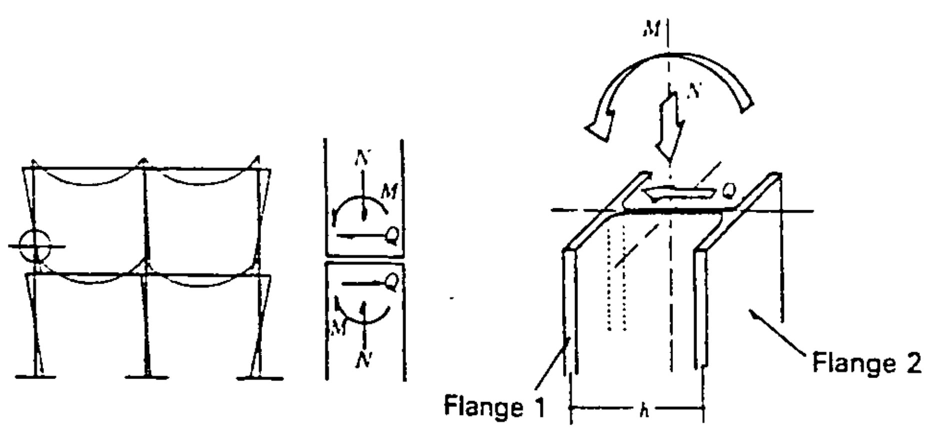
Fig. 2-37. Standard dimensions of truck and trailer (Unit : mm)

2) Column joint design by the existing-stress method

When the existing-stress method is used, joints are designed by use of the sectional forces (M, N, Q) at the joint positions as shown in Fig. 2-38 (a). In practice, the bending moment is considered to be borne by the flange, the shearing force by the web. Generally, it is also allowable to regard the axial force as proportionally distributed according to the sectional area ratio of flange and web. Therefore, the forces that act in the flange and web and their directions are as indicated in Fig. 2-38 (c).

	Flange 1	Flange 2	Web
Bending moment	: M/h (↓)	M/h (↑)	0
Shearing force	: 0	0	Q (←)
Axial force	: $A_F/A \cdot N$ (↓)	$A_F/A \cdot N$ (↓)	$A_W/A \cdot N$ (↓)
Total	$\frac{M}{h}+\frac{A_F}{A}\cdot N$ (↓)	$-\left(\frac{M}{h}\right)+\frac{A_F}{A}\cdot N$ (↓)	$\sqrt{Q^2+\left(\frac{A_W}{A}\cdot N\right)^2}$ (↙)

The resultant forces acting on the flanges and web can thus be calculated and the number of bolts required at each joint position may be determined.



(a) (b) (c)

Fig. 2-38. Design of column joints

3) Column joint design by the total-strength method

When the total-strength method is used, joints must be designed so as to transmit the allowable unit stress f_t of the section. The following equations can be used to find the number of high-strength bolts required for a frictional connection:

$$n_F R = B \times t_f \times f_t \tag{2-13}$$

$$n_w R = (H-2t) \times t_w \times f_t \tag{2-14}$$

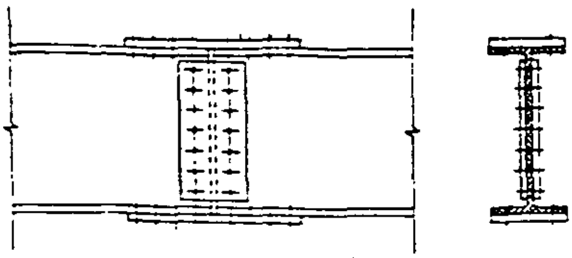
where n_F = number of bolts for flange, n_W = number of bolts for web and R = shearing force per bolt.

This approach may also be applied to the design of beam joints.

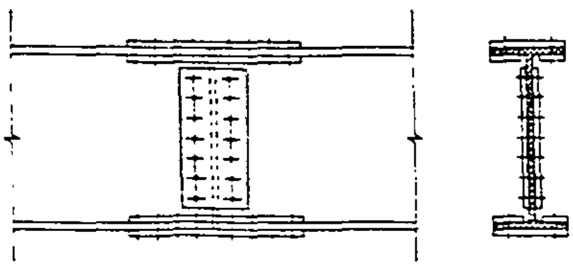
4) Examples of joint design

Fig. 2-39 (a) and (b) show beams field-joined with high-strength bolts. Bolted friction joints of flanges are of two types: the one-side friction type as in (a) and the two-side friction type as in (b). Allowable shearing force on the high-strength bolts is twice as high for the two-side type, which is therefore more widely used. When the flange width is 125 mm or less, the width for doubling on the inner face of the flange is narrow, making it difficult to secure the minimum edge distance. For such members the one-side friction joint must be used.

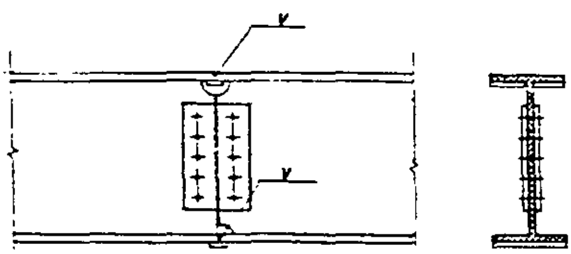
Webs may be connected by a two-side bolted friction joint in most cases. Where complete connection welding is used instead, a scallop must be provided in the web, through which a backing plate is passed. When ALC (autoclaved lightweight concrete) slabs or deck plates are used as floor members in a steel structure, welded joints are preferable since they leave no protrusions on the top flange of the beam.



(a) Flange 1-side friction joint



(b) Flange 2-side friction joint



(c) Flange welding (example of field welding)

Fig. 2-39. Examples of beam joints

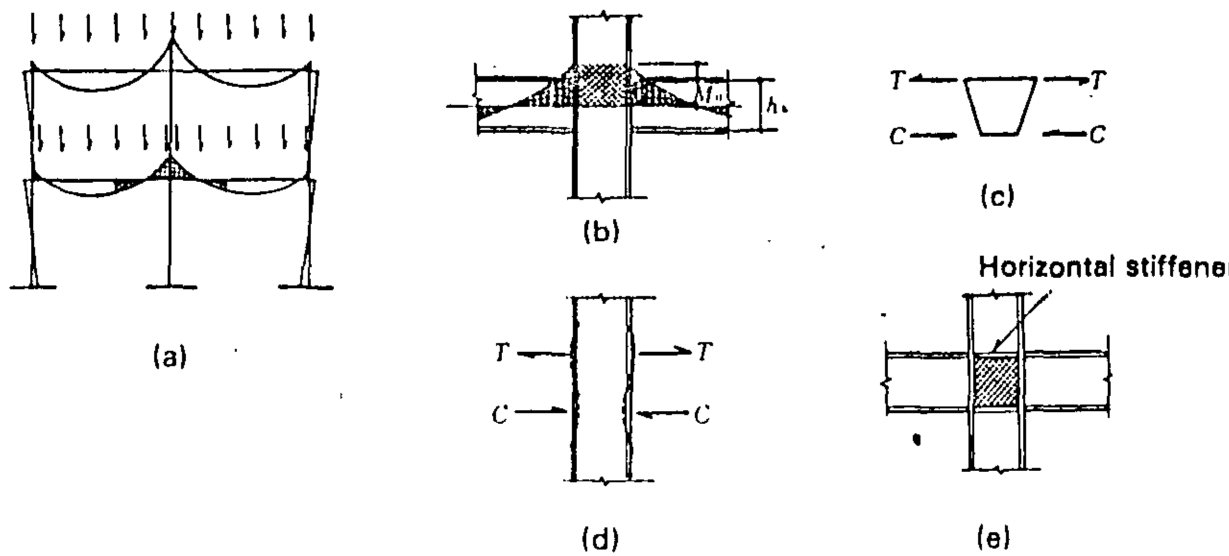
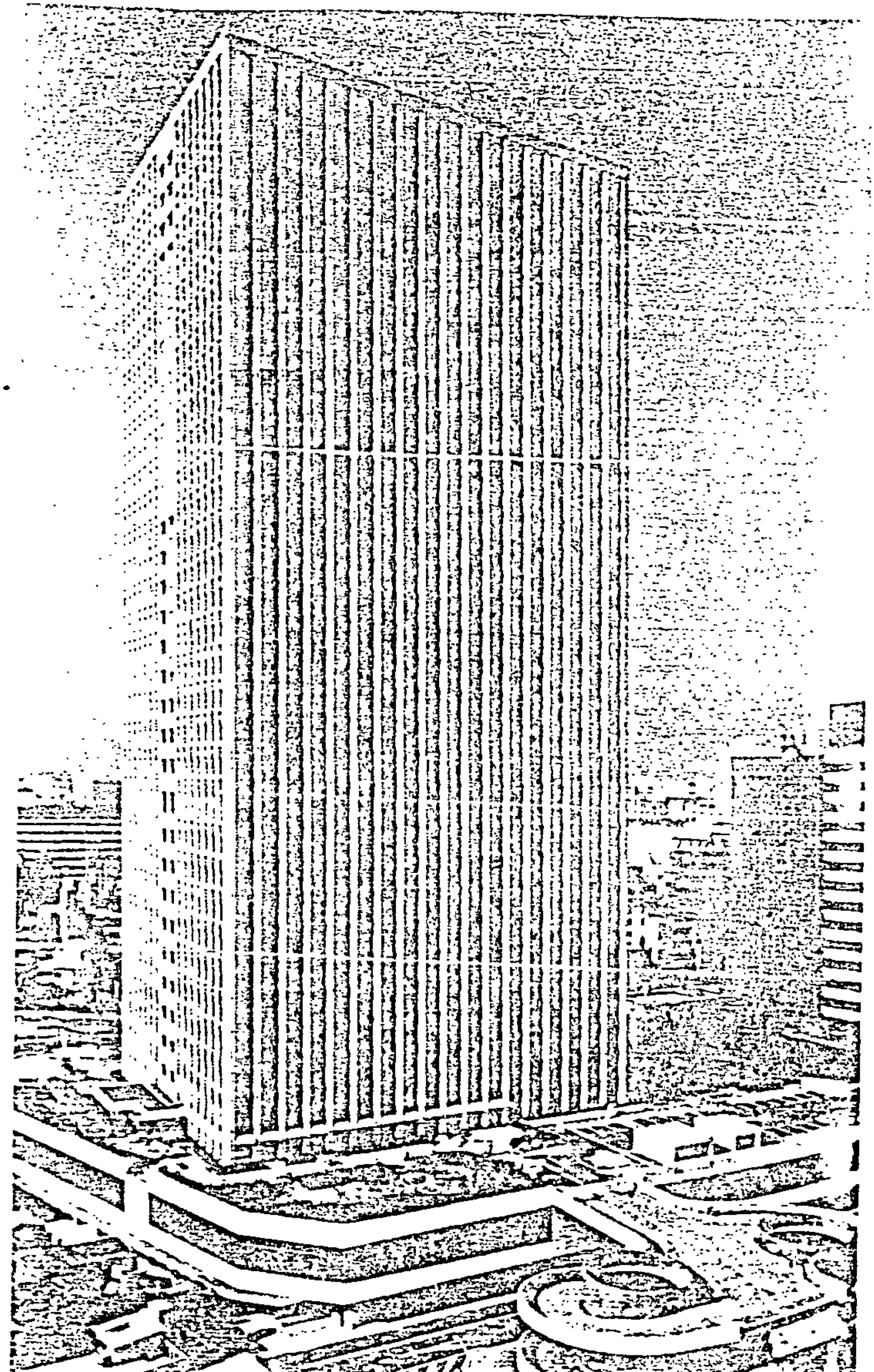
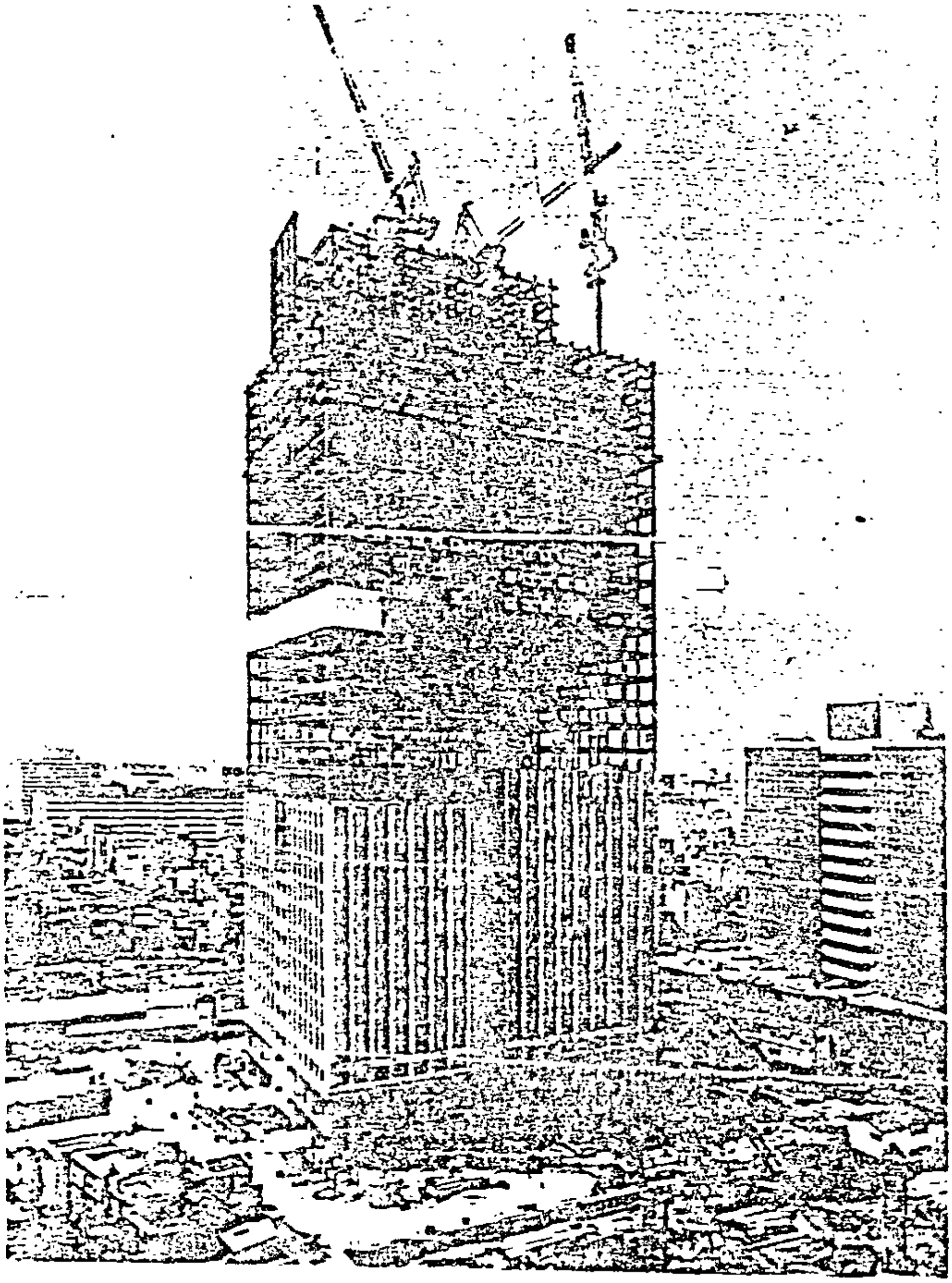


Fig. 2-40. Stress of beam-to-column connection under vertical load



(d) July 6, 1979



(c) August 5, 1978

Fig. 2-113. High-rise building under construction

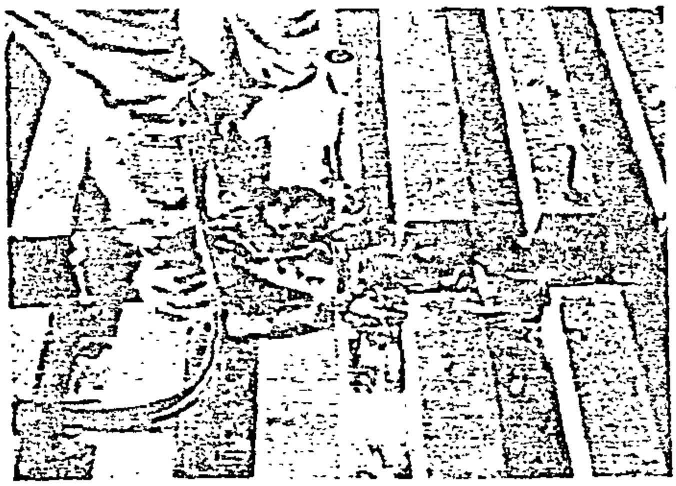


Fig. 2-109. Welding of stud bolts

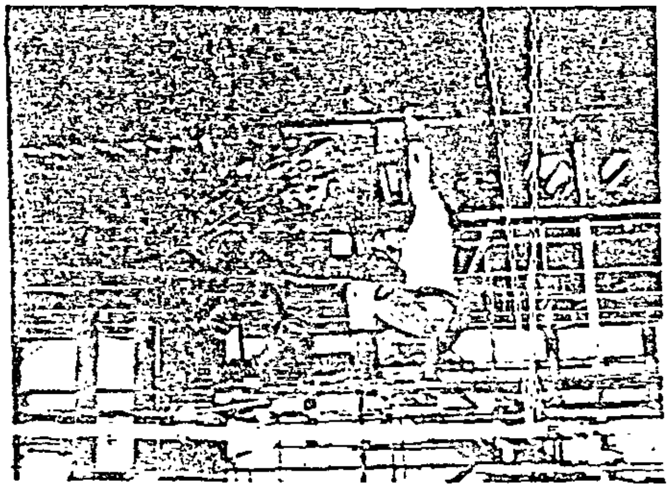


Fig. 2-110. Attachment of PC-bar braces

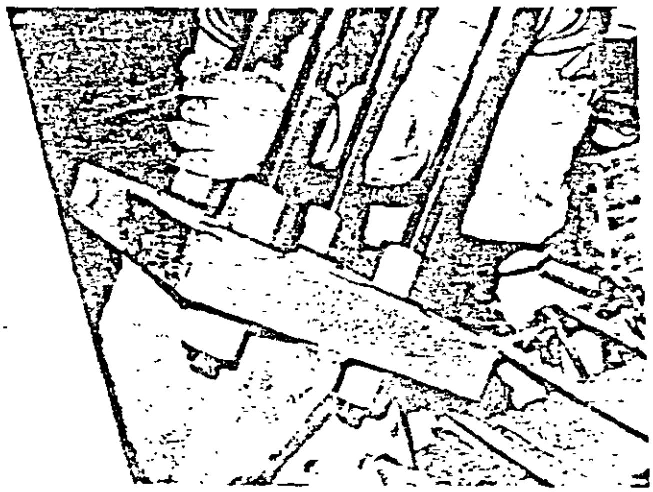
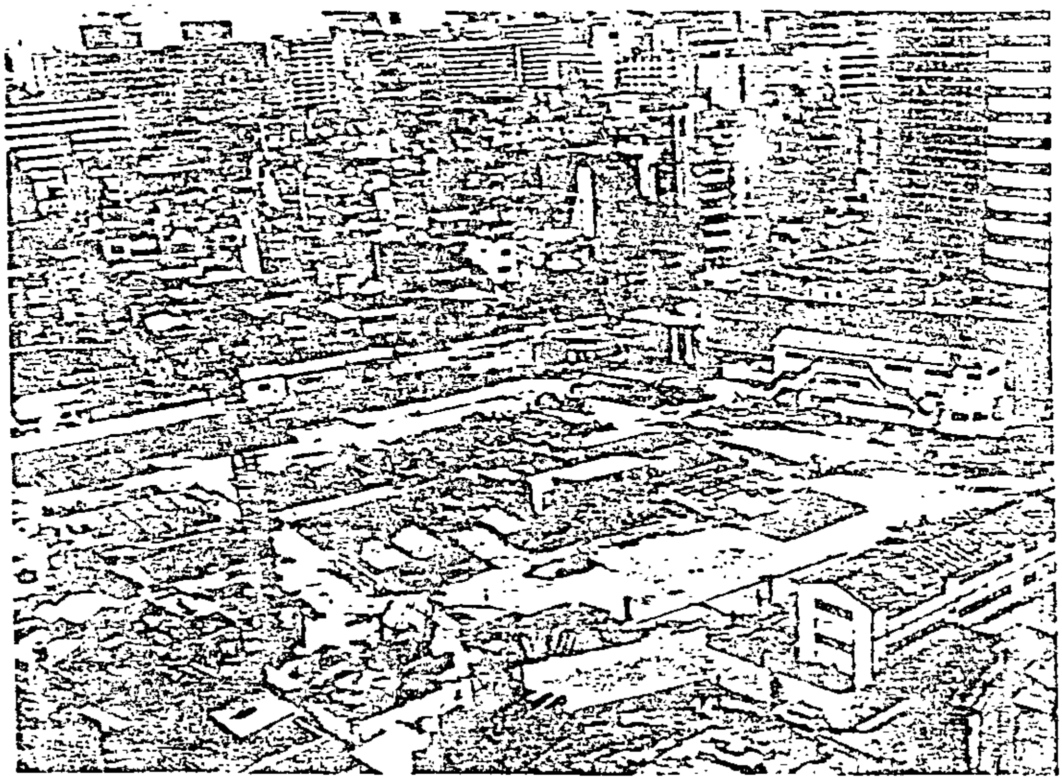
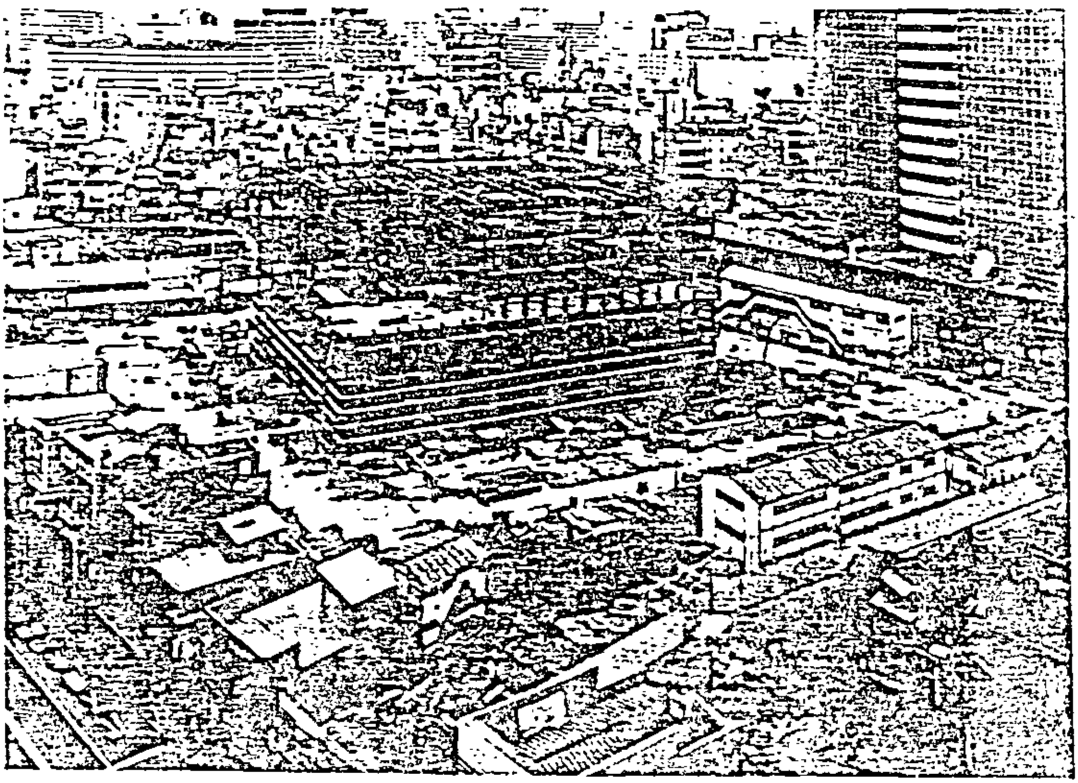


Fig. 2-111. Details of PC-bar brace joints



(a) April 8, 1977



(b) March 5, 1978

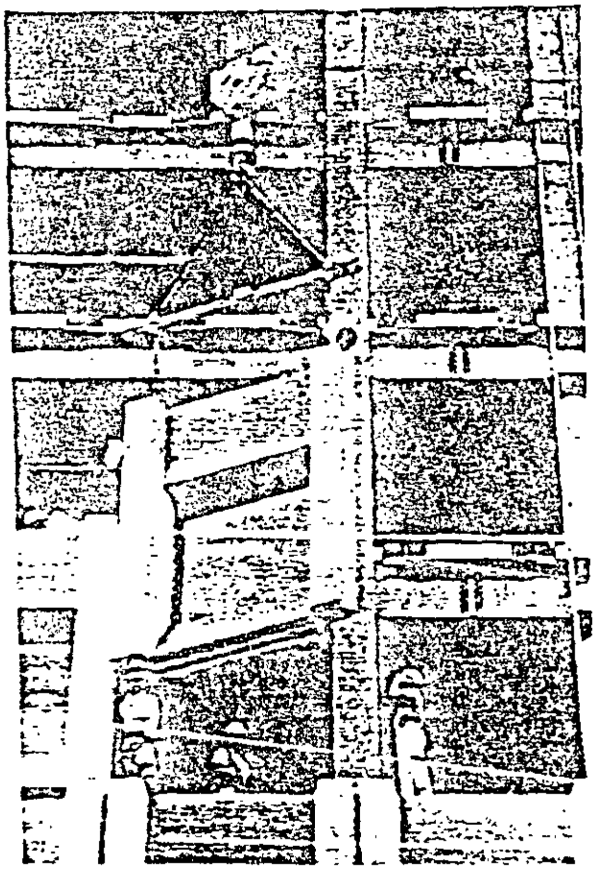
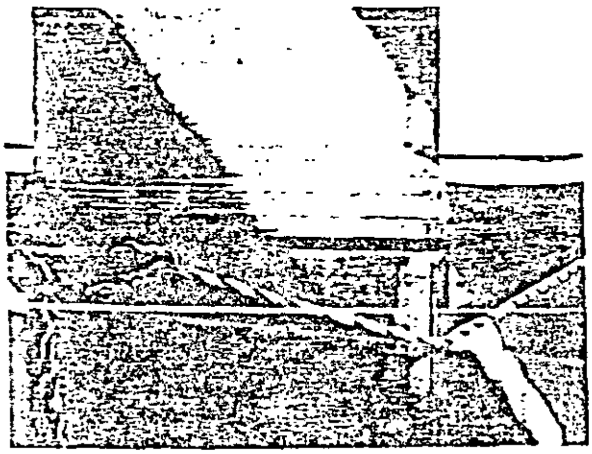


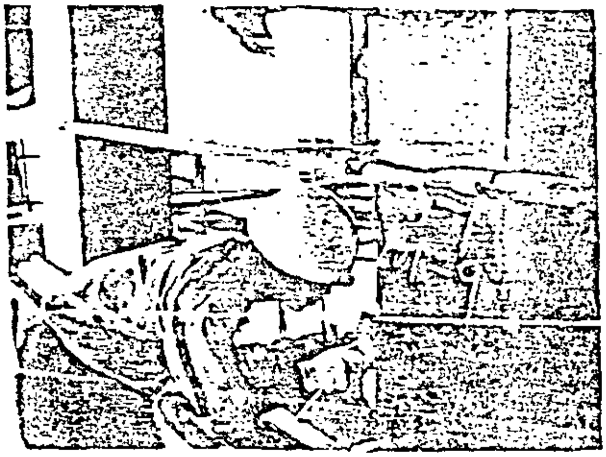
Fig. 2-112. Hanging of precast concrete slabs for curtain wall

(a) Field welding of beam flanges

(b) Field welding of column joints



(c) Outer appearance after field welding



(d) Appearance test

Fig. 2-106. Field welding of columns and beams

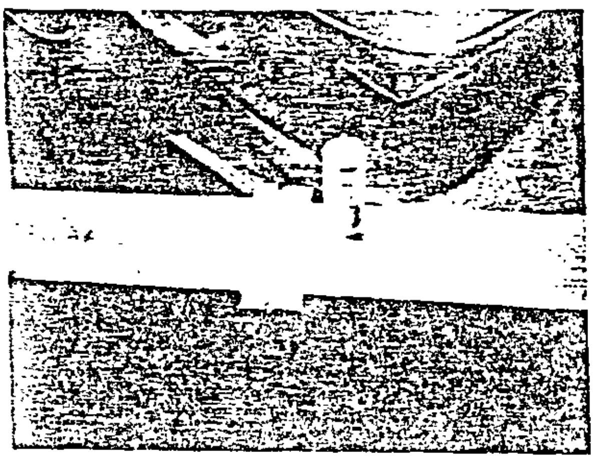


Fig. 2-107. Color check of field welds

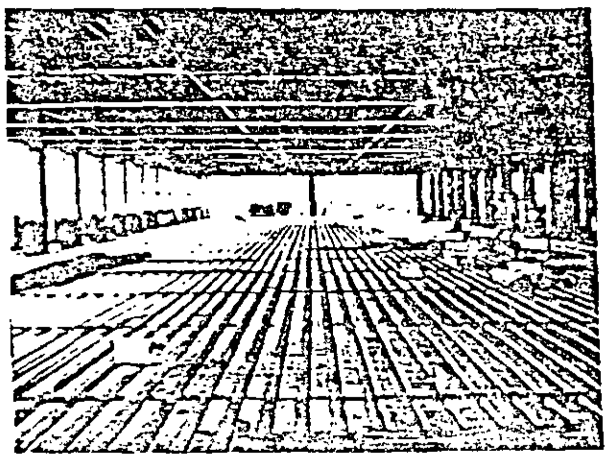
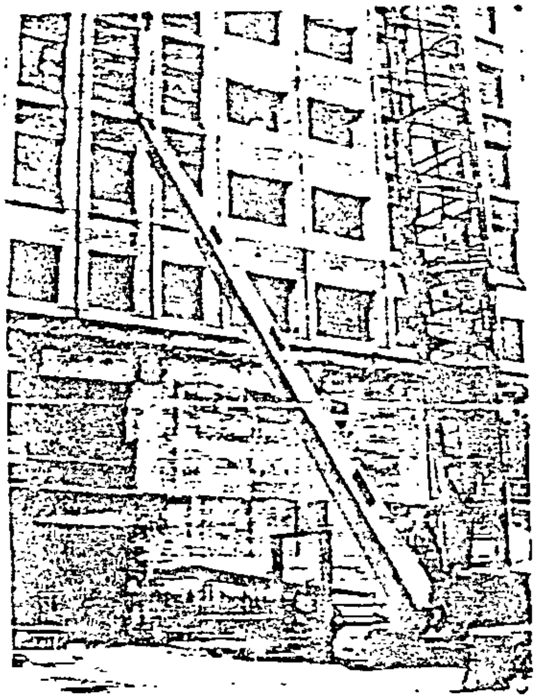
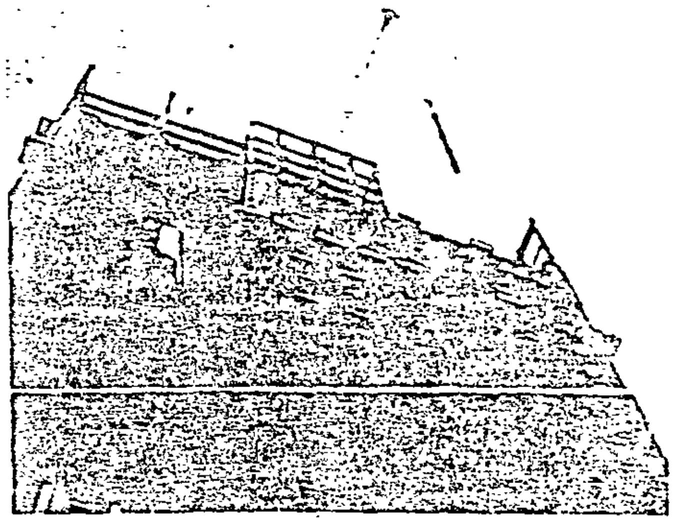


Fig. 2-108. Installation of deckplates

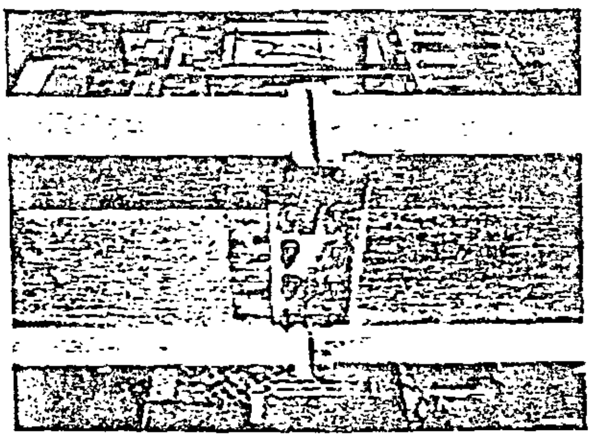


(a)

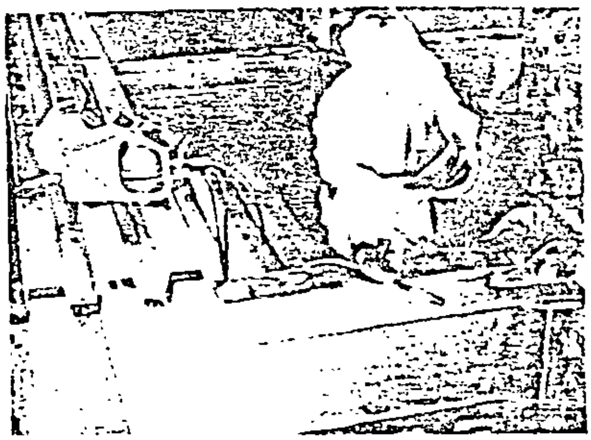


(b)

Fig. 2-103. Lifting of steel columns

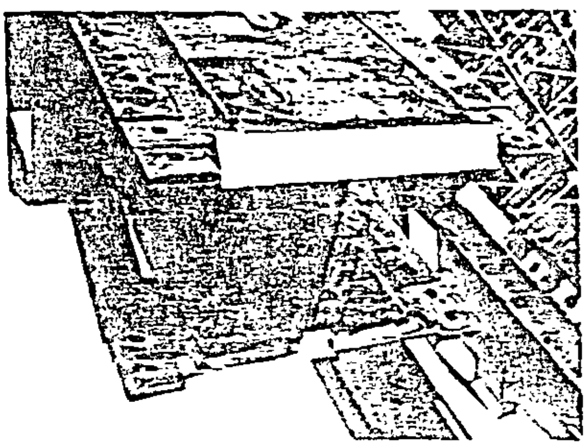


(a) Beam-to-beam joints
(before field welding)

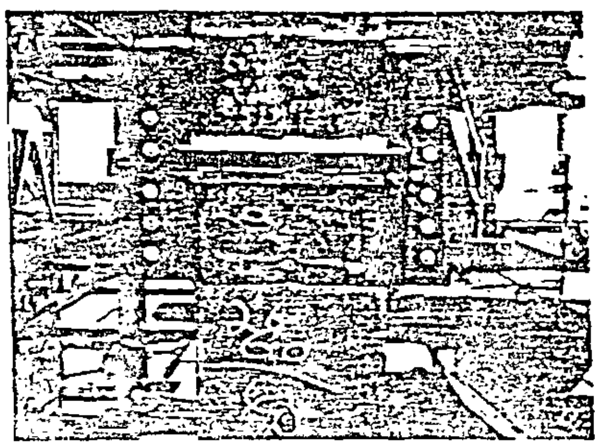


(b) Beam-to-column joints
(before field welding)

Fig. 2-104. Beam-to-beam joints and beam-to-column joints



(a) Groove shape at column joints



(b) Temporary column fastening with ordinary bolts, using an erection piece

Fig. 2-105. Setting of column joints

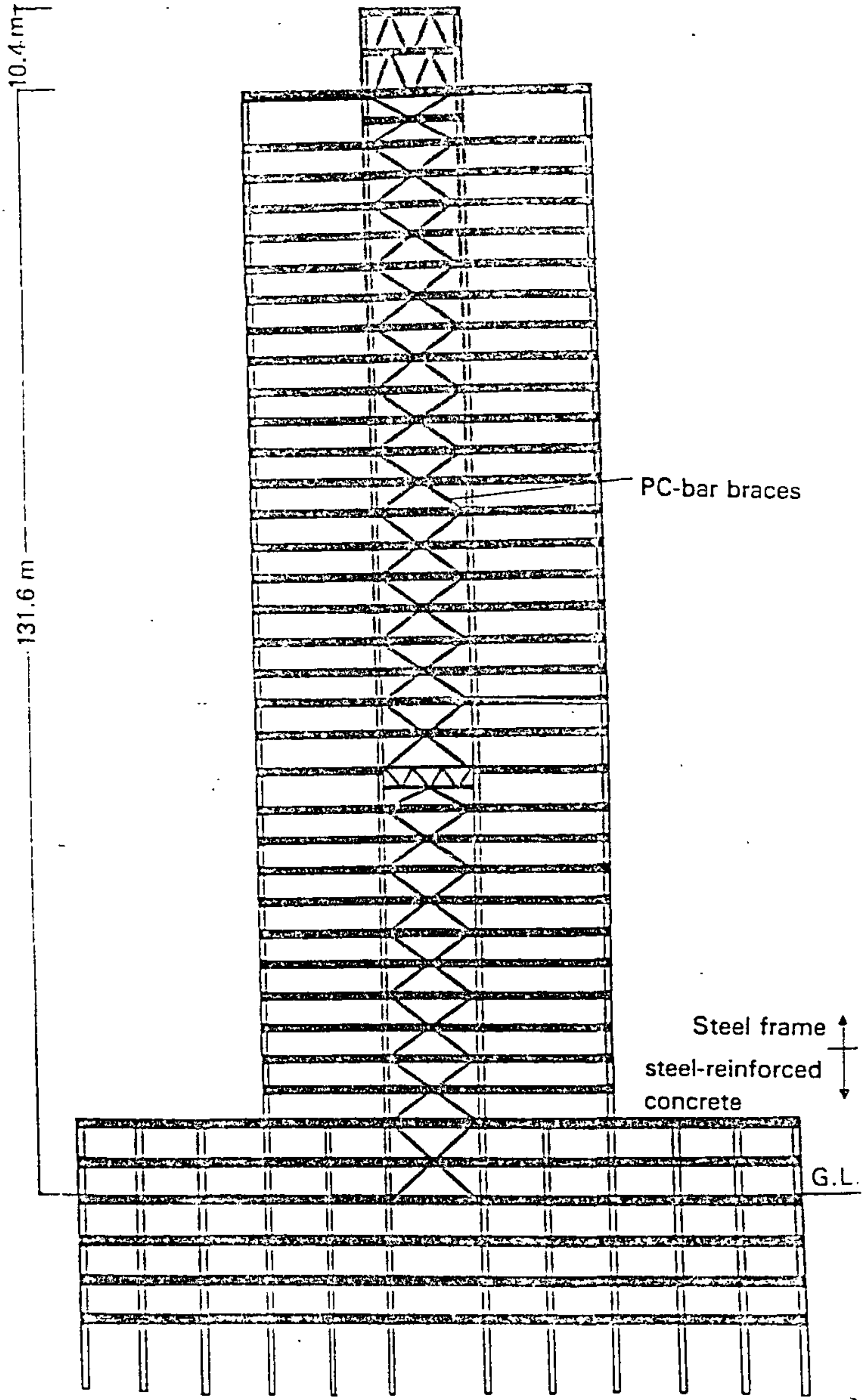


Fig. 2-102. Cross section

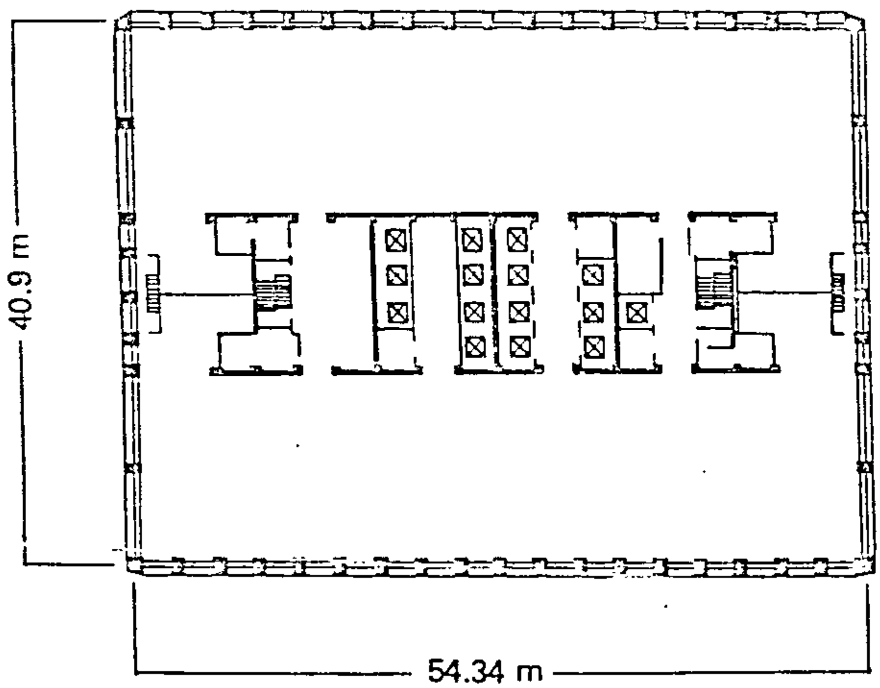


Fig. 2-101. Standard floor plan

methods used are ultrasonic flaw detection, X-ray radiographic examination and color checking. In the latter method, shown in Fig. 2-107, a penetrant, usually red, is applied to the weld surface. After rinsing, a developer is applied which brings out the color in penetrant which remains in such surface flaws as may exist. It is a simple, effective method.

When erection has been completed at a given floor level, the laying of deckplates for use both as molding and as scaffolding begins (Fig. 2-108). The floor of this building is a lightweight reinforced-concrete structure. To ensure complete integration of the floor slabs with the steel beams, stud bolts are welded to the beams (Fig. 2-109). The PC-bar braces used as an earthquake-proofing element are shown in Figs. 2-110 and 111.

Once the deckplates have been laid, precast concrete slabs for the external curtain wall are mounted (Fig. 2-112).

Fig. 2-113 provides a general view of the building at successive stages of construction.

splice plates and the protrusion of the bolt heads, the sectional dimensions of the column joints tend to be large. Thus the column finish materials may have to be reduced in thickness. Use of field welding avoids this drawback.

The steel columns, each equivalent in length to a number of stories, are lifted by a tower crane as shown in Fig. 2-103. Each column is temporarily-fastened with ordinary bolts, using an erection piece, to the head of the previously installed column, as illustrated in Fig. 2-105 (b). The beams corresponding to each column (normally involving three or four stories) are bolted temporarily to each other. After all members are aligned, the high-strength bolts are fastened. Then field welding is performed (Fig. 2-106) on beam-to-beam joints, beam-to-column joints (Fig. 2-104) and column-to-column joints (Fig. 2-105), in that order. Deformation of members due to welding is avoided by a procedure in which two welders work opposite one another when welding a joint. The welding method employed is CO_2 gas semiautomatic arc welding.

For nondestructive inspection of field welds, the

The reasons why field welding has recently come into wide use for connection of joints in high-rise buildings are summarized below.

(1) Economic advantages

Compared with use of bracket-type joints with high-strength bolts, use of field welding reduces both the number of high-strength bolts required overall and the weight of steel products such as splice plates and bolts. The reduction of section area due to bolt holes can be avoided. Use of uniform types of connections reduces the number of fabrication steps and lowers production and shipping costs.

(2) Improved reliability of field welding

(3) Construction advantages

When the flanges of ultra-thick wide-flange beams are connected to columns by high-strength bolts, the number of bolts required is quite large. Due to the thickness of the

2) Example of construction

Using a typical high-rise building as an example, the steps by which a building is erected are summarized below. A high-rise building is chosen as the example because many new techniques and concepts have appeared in the field of high-rise construction, aimed basically at higher quality and greater economy in the construction of steel-frame buildings. Some of these techniques and concepts have also been applied widely to low- and medium-height steel-frame buildings. It is hoped that study of the applications of these methods will lead to still more new approaches that can be applied to steel-frame buildings in the future.

Our example is a high-rise building for office and shops, having four stories underground and 34 stories aboveground, with a two-story penthouse. Fig. 2-101 shows the standard floor plan. The standard floor height is 3.7 m and total height is 142 m above ground level. The building structure consists of a composite structure from the second floor down and a pure steel-frame structure from the third floor up, with a rigid-frame structure equipped with prestressed-concrete-bar braces serving as an earthquake-proofing element for part of the central core.

The columns of the steel-frame structure are made of square tubes and wide-flange beams of 450 to 700-mm outside dimensions, while the beams are made of wide-flange beams or castellated wide-flange beams. In the short-span (about 3 m) portions, bracket-type joints are provided in the middle of the beams. The flanges are butt-welded together and the webs are connected with friction joints using high-strength bolts. For the long-span portions, the girders are castellated wide-flange beams. Their flanges are butt-welded to the columns and the webs are connected to the columns by friction joints using high-strength bolts. When the columns are made of square tubes, the column joints are butt-welded together around the entire circumference.

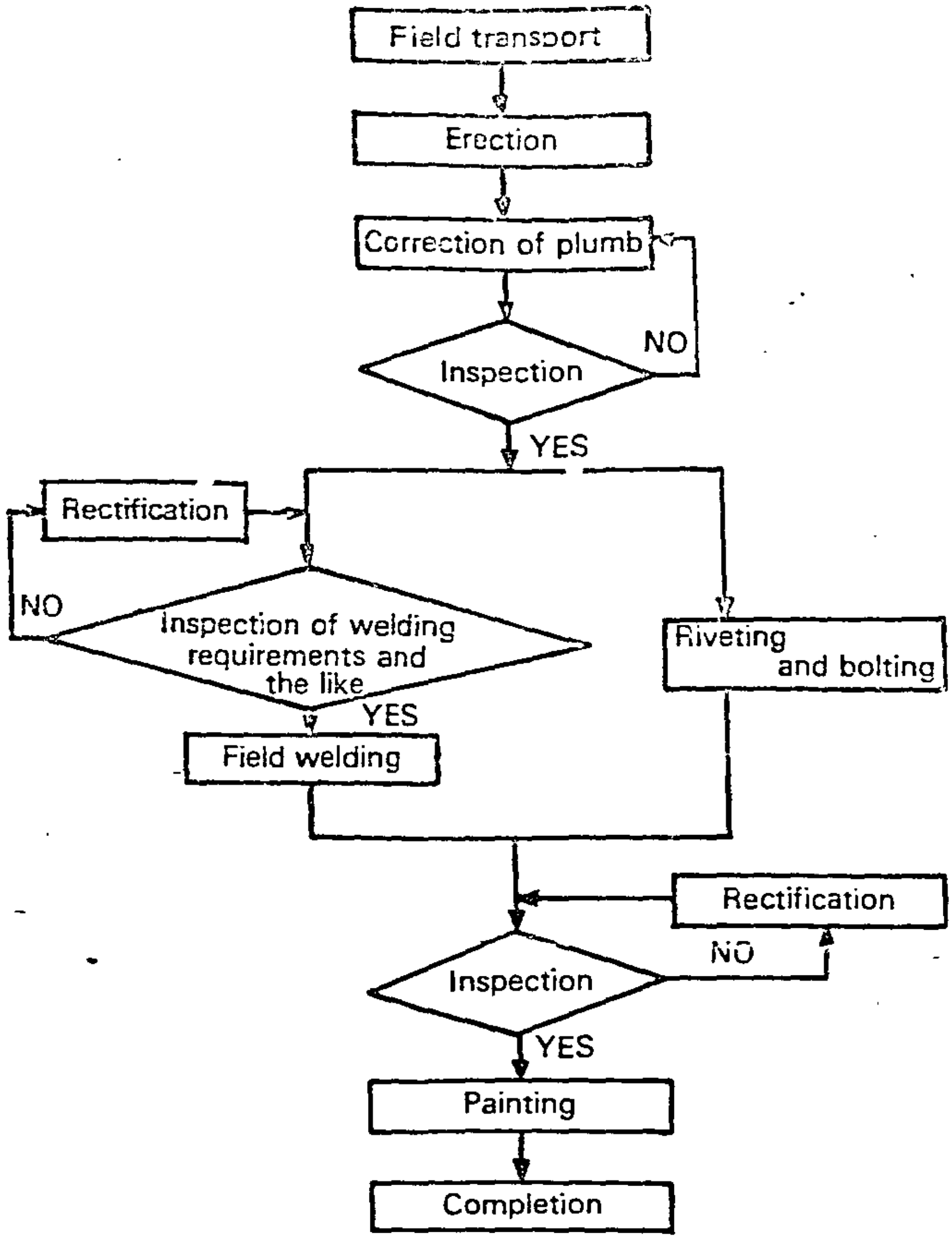
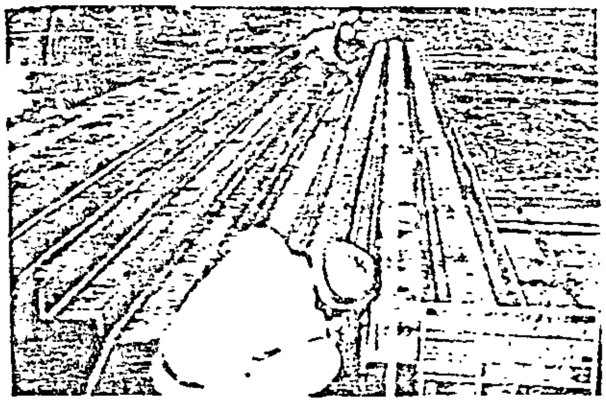
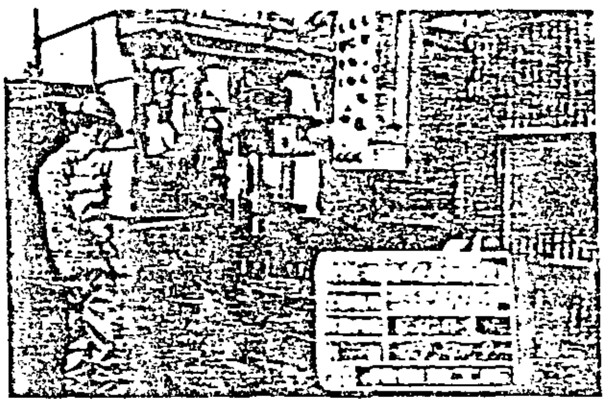


Fig. 2-100. Flowchart of field work

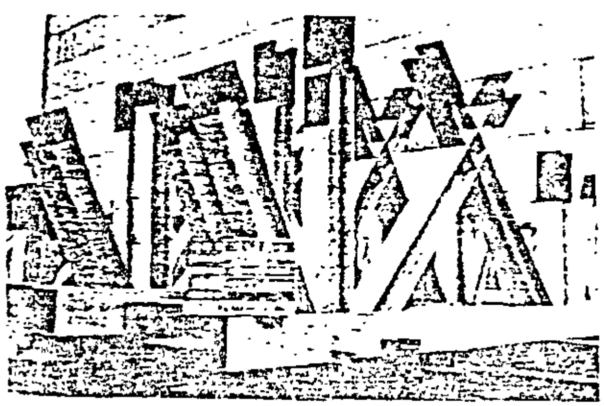


(b) Marking-off of wide-flange beams

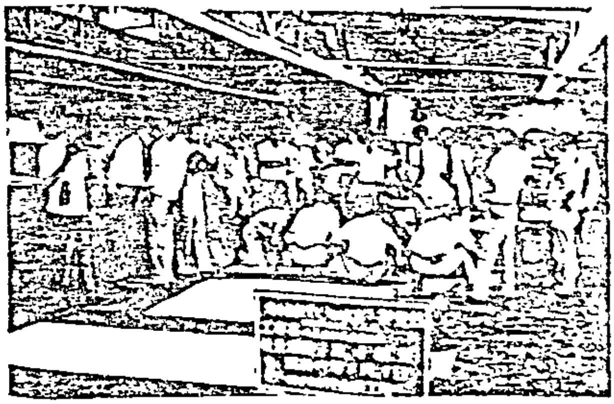


(d) Drilling

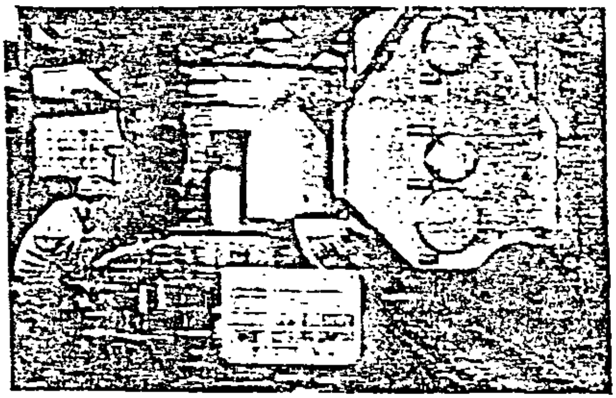
(f) Product inspection



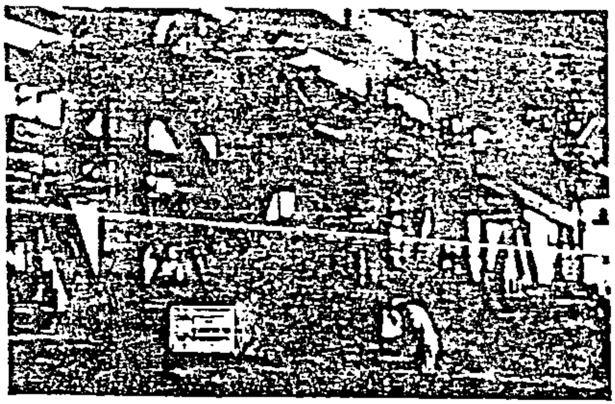
(h) Completion of painting



(a) Inspection of full-size drawings



(c) Cutting of wide-flange beams



(e) Welding at column shop

(g) Painting

Fig. 2-99. Examples of shop work

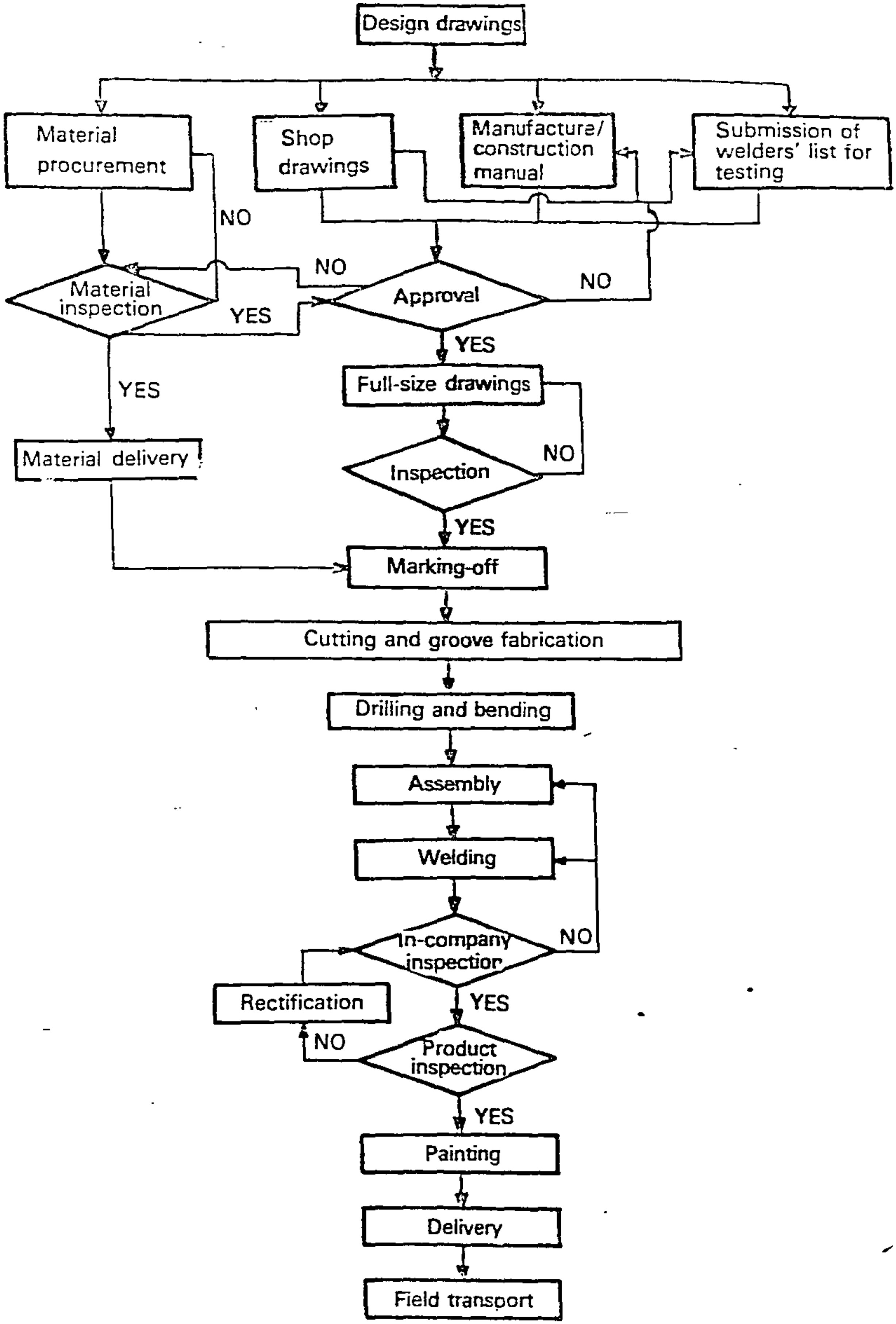


Fig. 2-93. Flowchart of shop work

2.5.3 Construction of Steel Structures

1) Construction procedure

When preparing the structural design of a steel structure, one should keep in mind always the procedure by which a building is erected. A good understanding by the designer of how steel members are made, fabricated and erected to form the completed structure leads to superior structural design. Lack of such understanding can lead to costly mistakes at the design stage: joints with high-strength bolts that are unfastenable manually; details that are impossible to weld; steel products cut in ways that make them difficult to transport; members so shaped that concentrated stress is unavoidable; joints for which the stress transmission mechanism is not clear; and so on. The construction of steel structures is outlined below, with reference to general precautions that should be observed.

Steel work is normally divided into shop work and field work. The flowchart in Fig. 2-98 indicates the series of steps from the receipt of design documents by the steel frame fabricator through the cutting and fabrication of steel frame members, their assembly and shipping from the shop. "Full-size drawings" in the flowchart refers to the making of a full-size drawing on the floor, by which detailed dimensions necessary for shop fabrication are determined and the necessary rules and templates are made. Computers are often used to store the reference data for all work from cutting to fabrication of members.

Fig. 2-99 shows some of these steps being carried out. A full-size inspection is underway in (a). "Marking-off" in (b) refers to indicating on the steel products themselves the cutting and fabrication work to be done.

The steps typically involved in field work are diagramed in Fig. 2-100.

3) Deformation and stress due to temperature changes

Every summer there are temporary interruptions to train service because of elongation and bending of rails by heat. The possibility of a similar effect on steel-frame structures should be considered. Let us examine the amount of expansion and contraction of steel products caused by seasonal temperature changes. The Architectural Institute of Japan's "Design Guidance for Tower-like Steel Structures and Commentary" (issued in 1980) provides a case study involving a change in temperature of about 40°C in a day's time.

Taking a simple beam of 30-m span as an example, and noting that the coefficient of linear expansion for steel products is 0.000012 (1/°C), the expansion and contraction of this beam due to temperature changes is given by

$$0.000012\,(1/℃)\times 40\,(℃)\times 3000\,(\mathrm{cm}) = 1.44\,(\mathrm{cm}) \tag{2-23}$$

If this expansion (or contraction) is expressed in terms of strain, we have

$$\frac{1.44}{3000}\times 100 = 0.048\% \tag{2-24}$$

Assuming that this beam is made of SS41 steel or a similar grade, the yield strain is about 0.12%. If the beam is restrained from moving at both ends, a stress of about 0.4 times the yield stress is produced by such a temperature change.

In some cases, such as that of a building that includes a smokestack, part of the structure is raised to high temperature while the rest is relatively close to normal temperature. For these structures, particular attention should be given to stresses and deformation caused by temperature differences, especially where parts at different temperatures are connected.

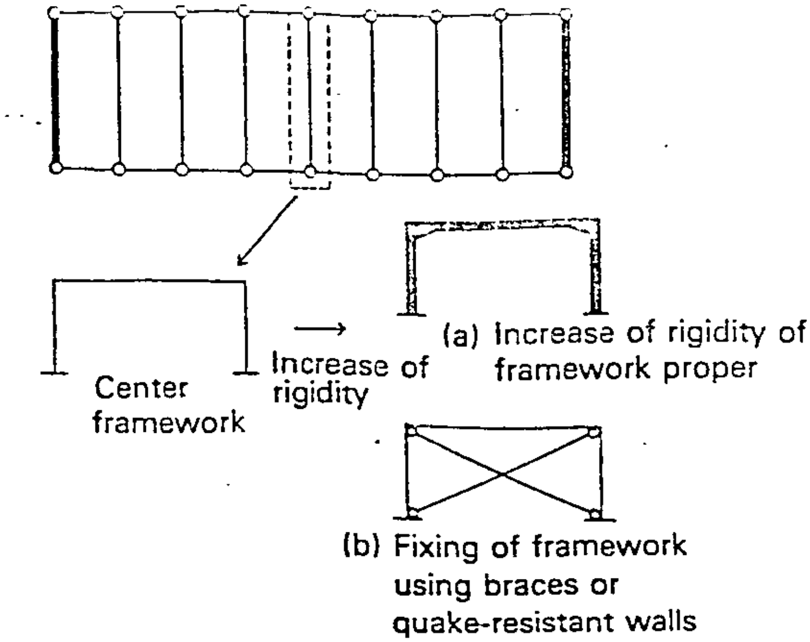


Fig. 2-95. Stiffening of building
structure in Fig. 2-93

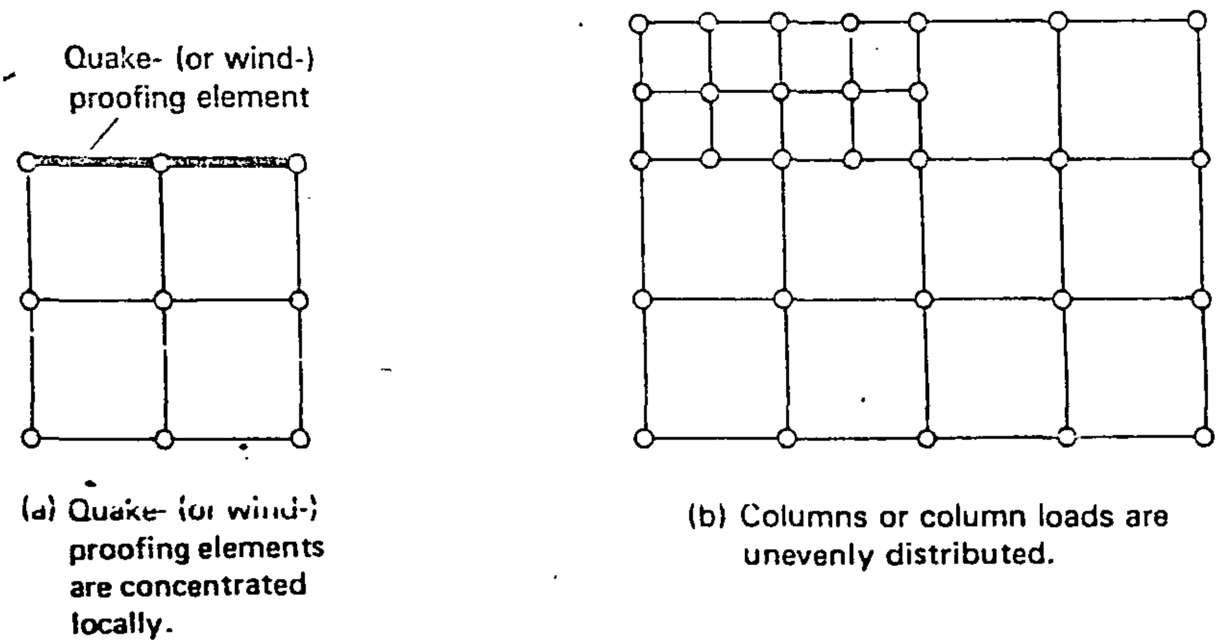


Fig. 2-96. Examples of building structure likely to be subjected to torsion

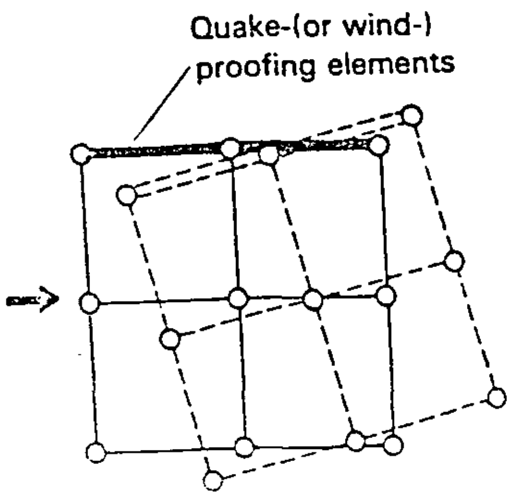


Fig. 2-97. Torsion of heavily eccentric
building structure

(2) Torsion of frame

The center of rigidity of the floor plane of a building sometimes deviates from the plane's center of gravity (Fig. 2-96), even though the stresses and deformation of the frame within that plane are within the specified limits. In that case, even when an earthquake or wind force acts in the expected direction the frame in the floor plane moves in parallel displacement in the same direction. The building becomes not only deformed but twisted as well (Fig. 2-97). This is particularly likely to occur when the earthquake-proofing elements are concentrated locally, as in Fig. 2-96 (a); when the columns are unevenly distributed, as in (b); or when the axial forces acting on the columns of a given floor are not uniform (that is, when the floor area of the floor above is smaller or when the live load is unevenly applied).

For buildings in which torsion is likely, the additional stresses caused by torsion of the entire building must be considered as well as the stresses and deformation produced in the respective planar frames. The occurrence of torsion is not only related to the main structural elements but is also greatly affected by the way in which secondary members such as partition walls and external walls — which may not be considered in structural calculations — are mounted. The designer must therefore give due care to the selection of these materials and the methods of mounting them.

Some members such as corner posts not only bear a vertical load but are also subjected to two-directional bending, axial and shearing forces due to horizontal forces such as seismic and wind forces. In these members, the stresses actually applied are more destructive than the stresses calculated for the planar frame by separating the forces in two directions by stress analysis. The danger is shown to be greater if the effects of torsion are taken into account. Special attention should be given to the corner posts of a building that is subject to torsion.

2) Deformation and vibration by horizontal forces

(1) Horizontal deformation of roof and floor elements

In steel-frame buildings, elements long and narrow in shape, as in Fig. 2-93, are often used. When a horizontal force is applied to a building having roof and floor elements of such shape and of low rigidity, these elements are sometimes deformed as shown in Fig. 2-94. This can be avoided by increasing the rigidity of these elements in ways such as increasing the section of the floor-element braces, arranging braces more densely or increasing the thickness of ferroconcrete slabs when they are used. Another method is to increase the rigidity of the frames in certain structural planes, normally every five or six spans (Fig. 2-95).

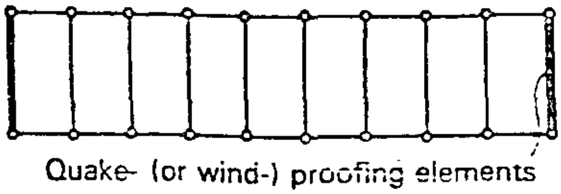


Fig. 2-93. Building structure with long, narrow elements

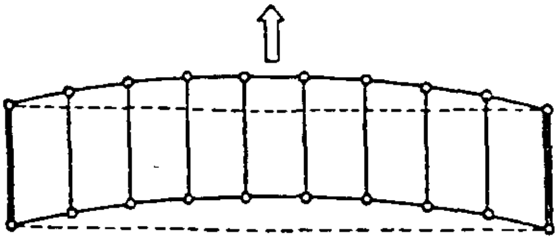


Fig. 2-94. Horizontal deformation of roof or floor element

vibration of the floor. The deflection of crane girders is limited to 1/500 to 1/1,200 or less, though it varies with the traveling speed of the crane and the type of work it will do.

Two kinds of vibration probems often arise in steel-frame buildings: vibrations caused by occupants moving about and those generated by the machinery or air-conditioning equipment installed in the building. The former problem, which is closely related to deflection of the building members, can be prevented to some extent by using high-rigidity members in the floor structure and the beams. Vibration from machinery, however, is likely to be transmitted throughout the building, its energy is large and in most cases it continues for long periods. Effective prevention is difficult.

The vibration energy generated by machinery is normally attenuated by use of rubber vibration insulators and the like. If necessary, the floor on which a machine is installed can be completely insulated and the machine separated from the building proper. As for air-conditioning equipment, care should also be taken about vibration and noise coming from the pipes and ducts.

2.5.2 Deformation and Vibration of Steel Structures

1) Deflection and vibration of floors

It is often said that the high strength of steel products allows the sections of building members to be small and the dead load of the building to be light. It is true that the strength of steel makes possible the use of small sections, on the basis of stress analysis alone. But with smaller sections comes greater susceptibility to deformation. If deflection is ignored and only the stresses produced in member sections are considered in design, problems with deformation will arise in the actual building. Its occupants may have a feeling of discomfort or unease, and the exterior appearance may be marred by cracks or visible deformations.

The Architectural Institute of Japan's "Design Standard for Steel Structures and Commentary" stipulates that beam deflection should normally be 1/300 of the span or less, and 1/250 for cantilever beams. These are general limits, and the deflection in any specific case should be decided on the basis of actual circumstances, considering the finish materials, intended use of the building and the

Consider first a case where a relatively lightweight crane is used. For a bracket-type crane (Fig. 2-92 (a)), the crane girder is supported by brackets extending from the columns. A bending moment is exerted on the columns when the load transmitted from the girder deviates from the vertical. Stress analysis of the main frames should also consider the horizontal force in the direction of crane travel which is produced when the brake is applied and which acts on the columns as an eccentric force.

When a crane of large size or high traveling speed is to be installed, thicker columns are used to support the crane girder, as shown in (b). The portions of these columns above the crane rails can be thin since they have only to bear the load of the roof. Accordingly, the points where the rigidity of the columns suddenly changes at the crane's level are sometimes pin-jointed. But it should be noted that if the upper portions of the columns are too low in rigidity they will lean outwards due to the thrust from the roof and become unstable.

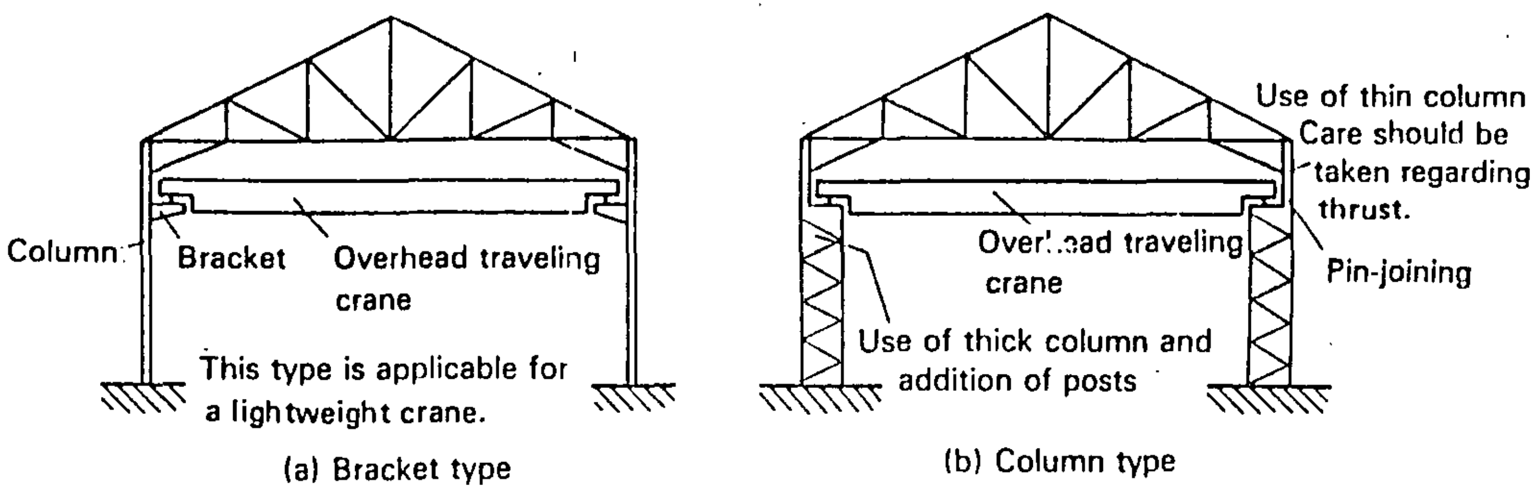


Fig. 2-92. Supporting systems for crane girder

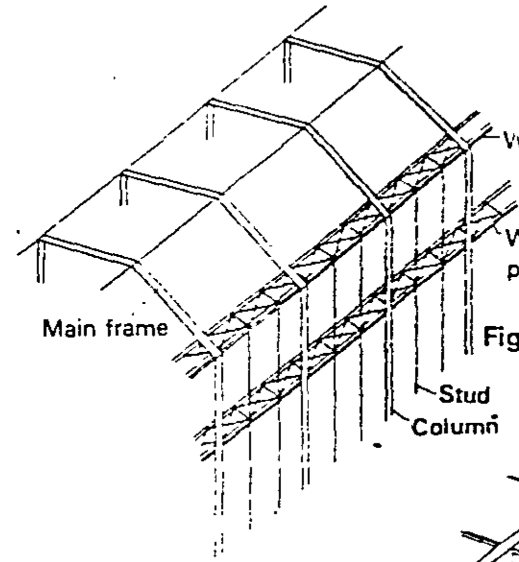


Fig. 2-89. Wind-bracing frames attached to cross-beam

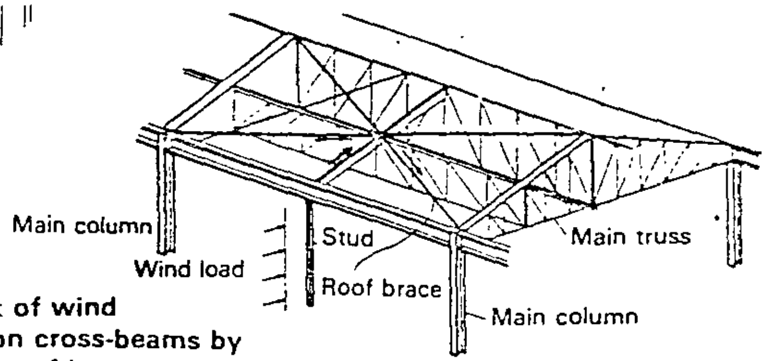
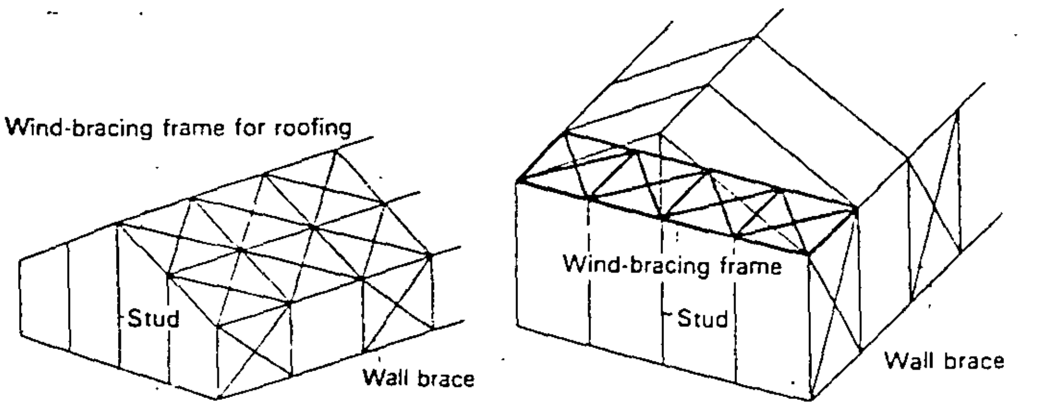


Fig. 2-90. Treatment of wind pressure on cross-beams by means of roof braces



(a) Pressure is transmitted to wall brace via roof brace.

(b) Pressure is transmitted to wall brace via wind-bracing frame.

Fig. 2-91. Treatment of wind pressure applied to gable planes

2) Walls

(1) Girts and studs

The walls receive the dead loads of the building materials and members as well as wind and earthquake forces. The loads are first transmitted to the girts, which in present practice are mainly lightweight steel shapes. Studs, which correspond to the sub-beams provided for the roof structure, are employed when the main frames are very far apart.

To transmit horizontal forces such as wind and earthquake forces to the main frames, wind-bracing frames are attached to the top and middle of the studs (Fig. 2-89), or braces are mounted in the planes of the main trusses and tie-beams (Fig. 2-90), so that the reaction of the studs is transmitted to the main frames. For buildings with very high eaves, in particular, wind-bracing frames are attached to the studs at mid-height. Applied in this way, wind-bracing frames are also a deterrent to buckling of the main columns in the weak-axis direction.

(2) Flow of horizontal forces in the ridge direction

Even if a building is constructed with rigid-frame structures and knee-braced truss structures in the span direction, braced truss structures are often employed in the ridge direction. In that case, horizontal forces acting in the ridge direction are transmitted through braces mounted in the planes of the main trusses and the wind-bracing frames in the gable planes, passing to the braces in the plane of the walls in the ridge direction (Fig. 2-91).

3) Buildings equipped with a crane

For a building equipped with an overhead traveling crane, the vertical, horizontal and impact forces acting on the building because of crane operation must be taken into account.

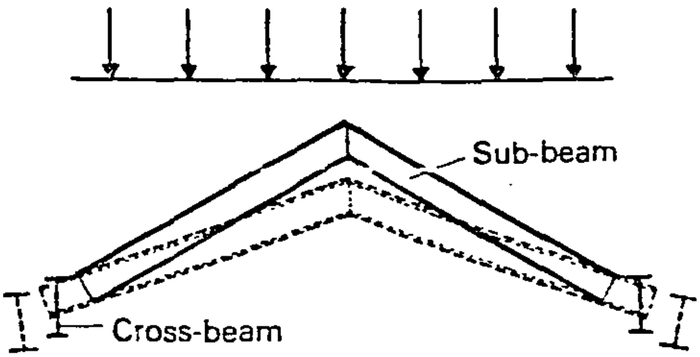


Fig. 2-87. Thrust of sub-beam

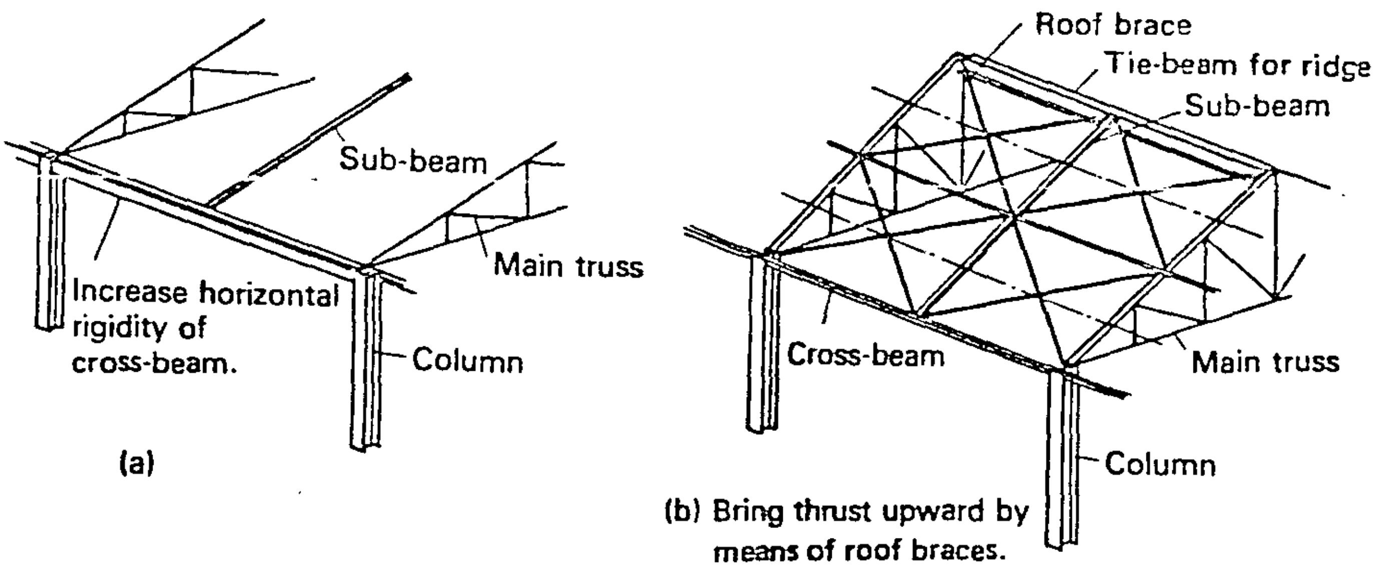
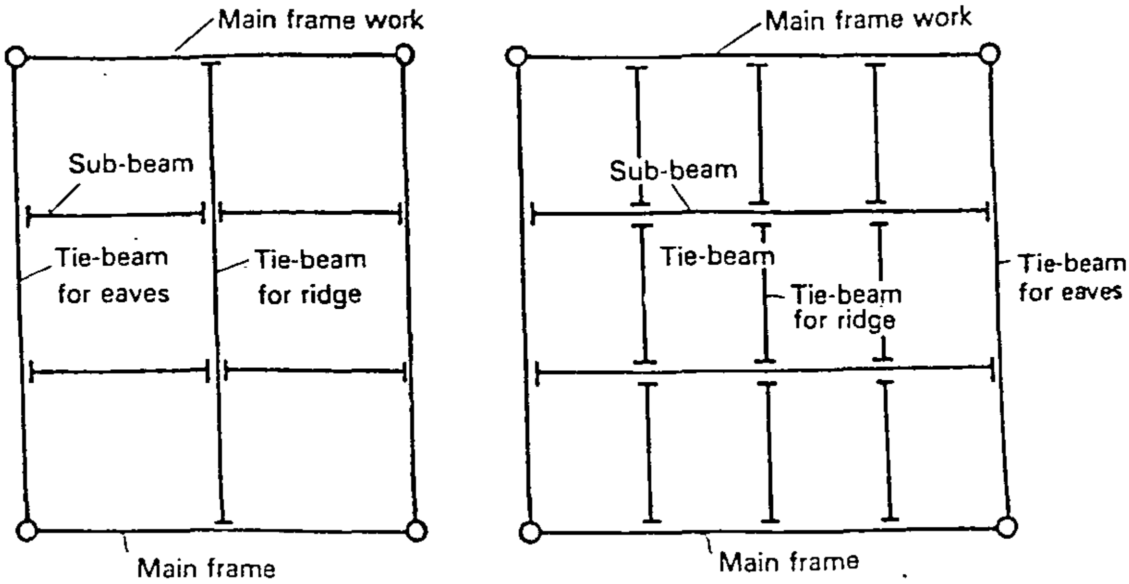


Fig. 2-88. Treatment of sub-beam thrust



(a) Tie-beams are preferable. (b) Sub-beams are preferable.

Fig. 2-84. Erection of sub-beams

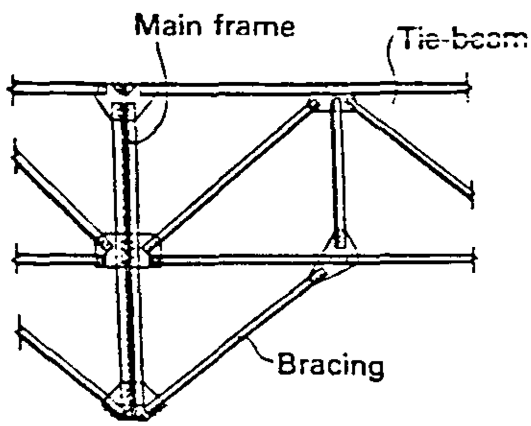
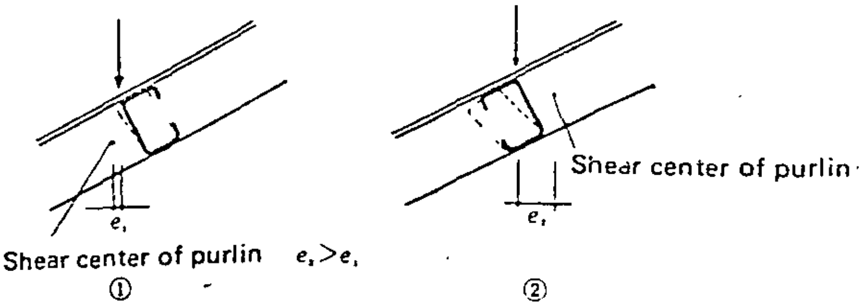


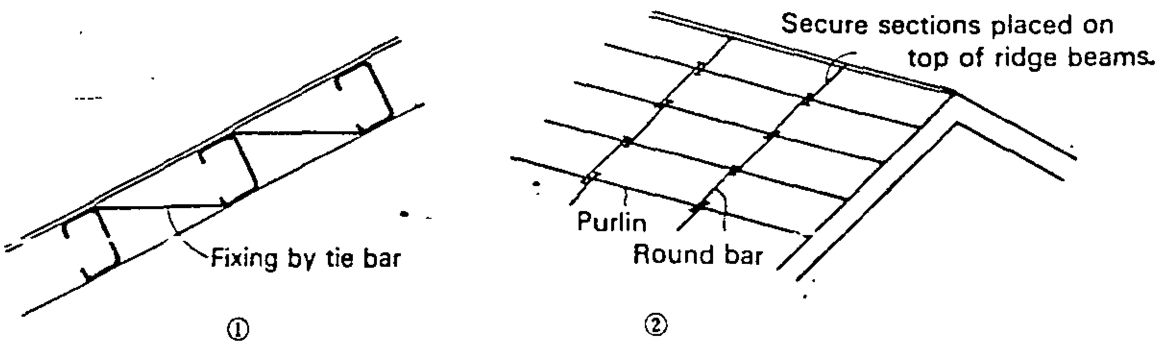
Fig. 2-86. Arrangement of main frame and tie-beam

(3) Tie-beams

The tie-beams transmit concentrated loads from the sub-beams (sub-trusses) to the main frames. They are necessary in many cases to restrain the deflection of the main frames and to prevent buckling of the lower chord member of the main frames (Fig. 2-86). Note that the thrust (horizontal reaction) of the sub-beams acts on the cross-beams (Fig. 2-87) and may deform the cross-beams. How this problem is avoided is illustrated in Fig. 2-88.



(a) Torsion of purlin



(b) Reinforcement of purlins against torsion

Fig. 2-83. Forces acting on purlin and reinforcement of purlin

The vulnerability of lightweight lip channels to torsion depends on their orientation on the roof. Of the two cases shown in Fig. 2-83 (a), torsion is more likely to occur in case ②. This is because the load-application point and the shear center of the purlin are too far apart. As a countermeasure, the purlins should be arranged in alternate orientations or reinforced with tie-bars or round steel, as in (b). Beams and trusses borne by the purlins are normally placed at intervals of about 4 m for economy in use of purlins.

The simplest approach is to match the intervals of the purlin support points (bearing materials) with the intervals of the main structural frames. This, however, increases the number of gable-roofed frames, which is unfavorable in terms of both cost and column layout in plan. It is better to space the main frames adequately (about 4-m interval) and provide them with sub-beams (sub-trusses).

(2) Sub-beams

Fig. 2-84 shows two methods of mounting the sub-beams. In method (a), the sub-beams (sub-trusses) are supported by tie-beams such as ridge beams and cross-beams that link the main frames. In method (b), the sub-beams are the same length as the main frames and are supported by cross-beams. Since in method (a) the span of the sub-beam (sub-truss) is far shorter than in method (b), steel products of small size may be used as the sub-beams.

Sub-beams (sub-trusses) are regarded as simple end-supported beams, and their deflection must be checked during design. Lateral buckling can occur under the condition of small downward load if a large upward load is caused by wind. Lateral buckling can be prevented by providing bracing at the lower chord member and at right angles to the sub-beam (Fig. 2-85).

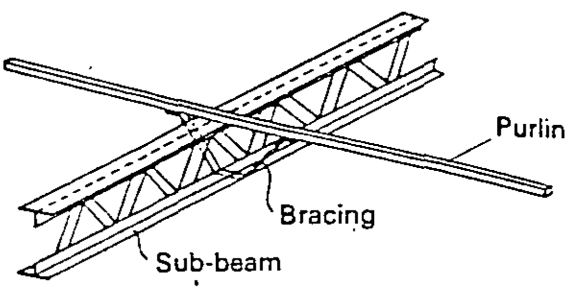
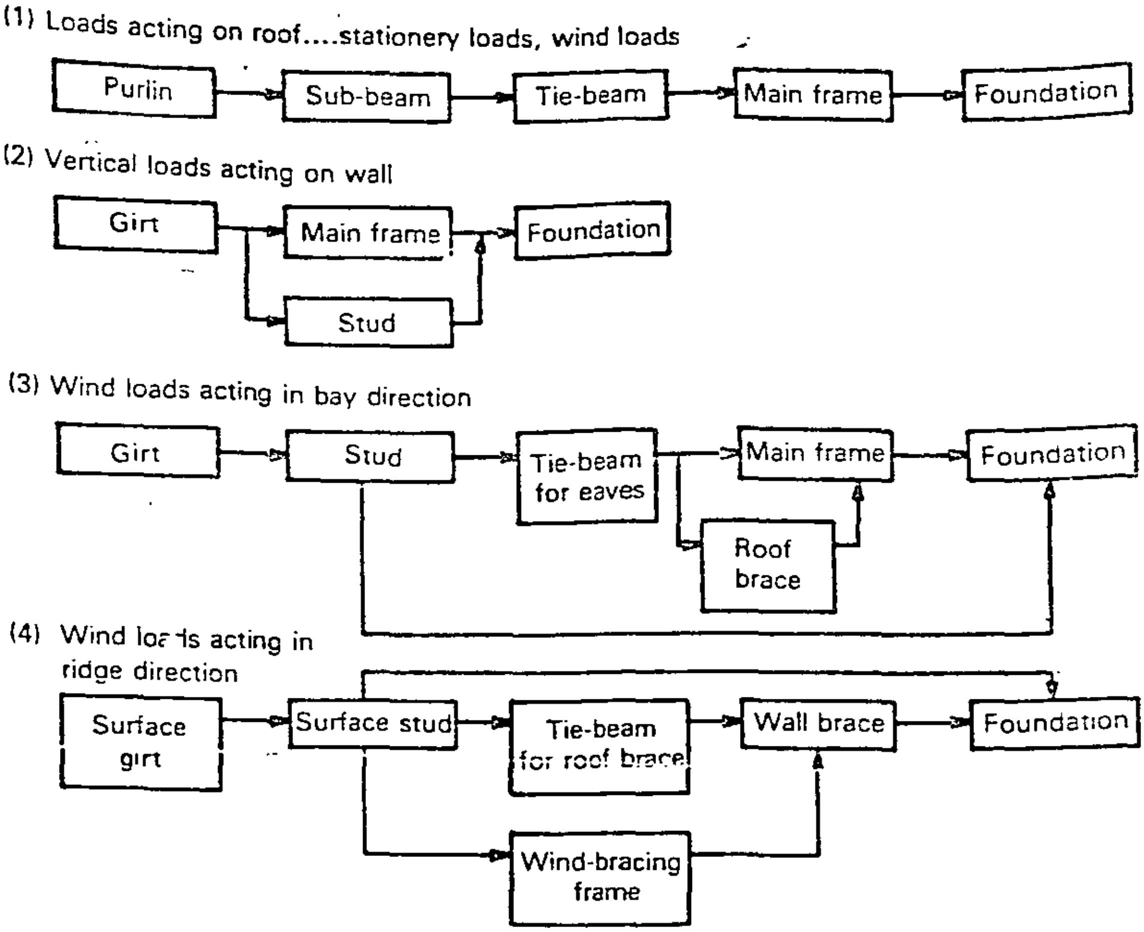


Fig. 2-85. Bracing for sub-beam



(b) Flow of forces

Fig. 2-82. Flow of forces in gabled frame structure

SECTION 2.5 FRAME ERECTION

.5.1 Flow of Forces

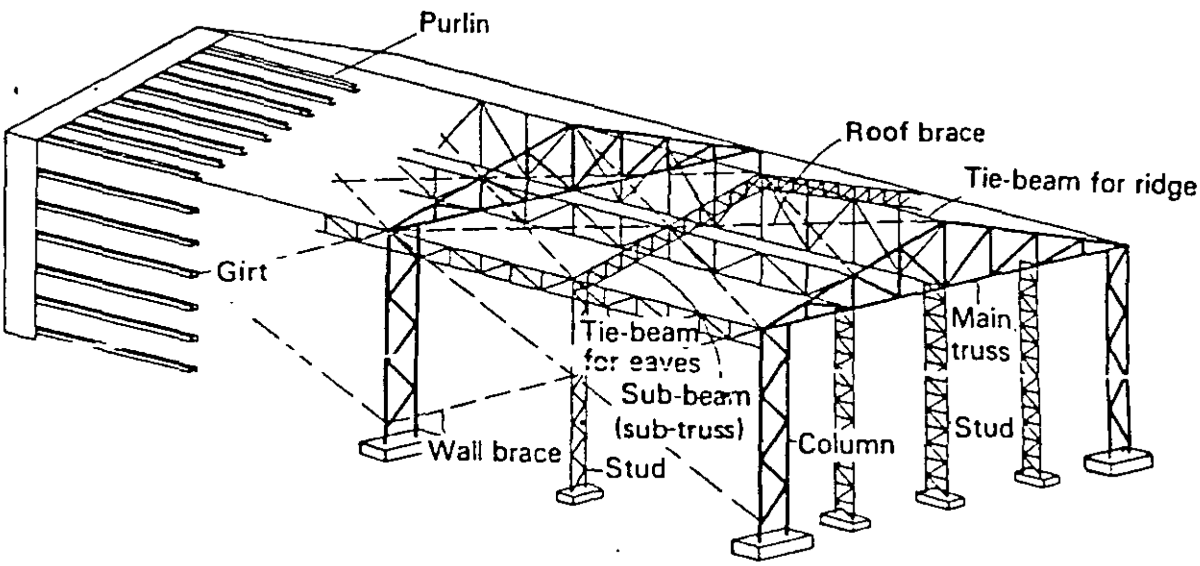
We now consider how forces applied to the roof and walls of a steel-frame structure are transmitted through the structure to the ground, using a simple gable-roofed frame (Fig. 2-82) as an example.

1) Roof truss

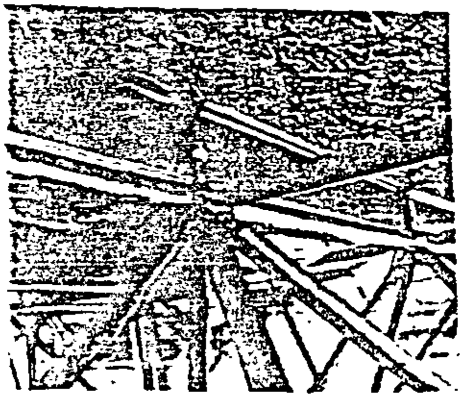
(1) Purlins

The purlins directly bear the weight of the roofing materials plus wind loads. The pitch of the purlins is normally decided by the distribution of the roofing materials. When the above-mentioned loads are light, lightweight lip channels are often used as purlins. Deflection and torsion are the main points to consider in deciding the section of the purlins. Roofing materials such as slate which are lightweight and rather brittle will be damaged if the purlins deflect excessively. Desirable deflection limits are 1/200 or less when slate is used and 1/150 or less for more flexible materials such as steel and plastic sheets.

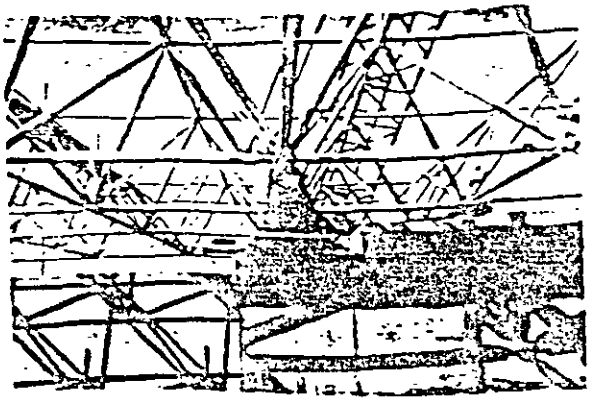
Due attention should be paid to the extent of inclination of the purlins in designing a steeply sloped roof.



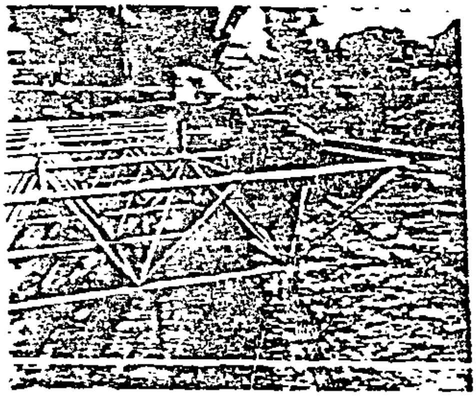
(a) Example of typical steel-frame building



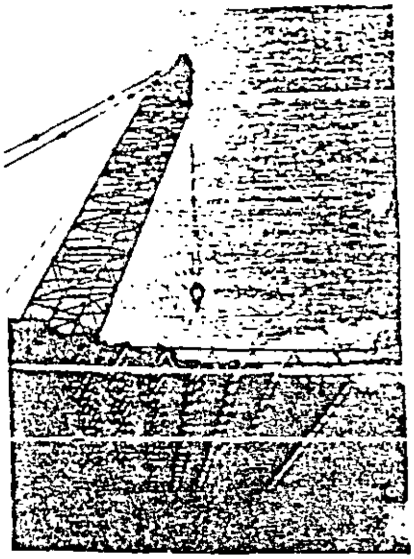
(a) Space truss joints



(b) Space truss supports



:) Field assembly of space truss



(d) Lifting of space truss

(e) Installation of space truss

Fig. 2-81. Example of dense-type space truss

The dense-type space truss in Fig. 2-81 is made of small-diameter steel tubes of the same type, intersecting at polyhedral bolted joints as shown in (a). Supporting points for this space truss are provided at proper points of the reinforced concrete walls as in (b).

After erection on the ground, a lightweight space truss like those just described is lifted by a truck crane and installed at the desired site. For larger space trusses, hydraulic jacks are used to raise them to the proper position for installation.

Where a great number of members converge in a dense-type space truss, the method of joining them and the shape of the joint are often vital points in design. When deciding the shape of a space truss, the joining method must be considered at the same time.

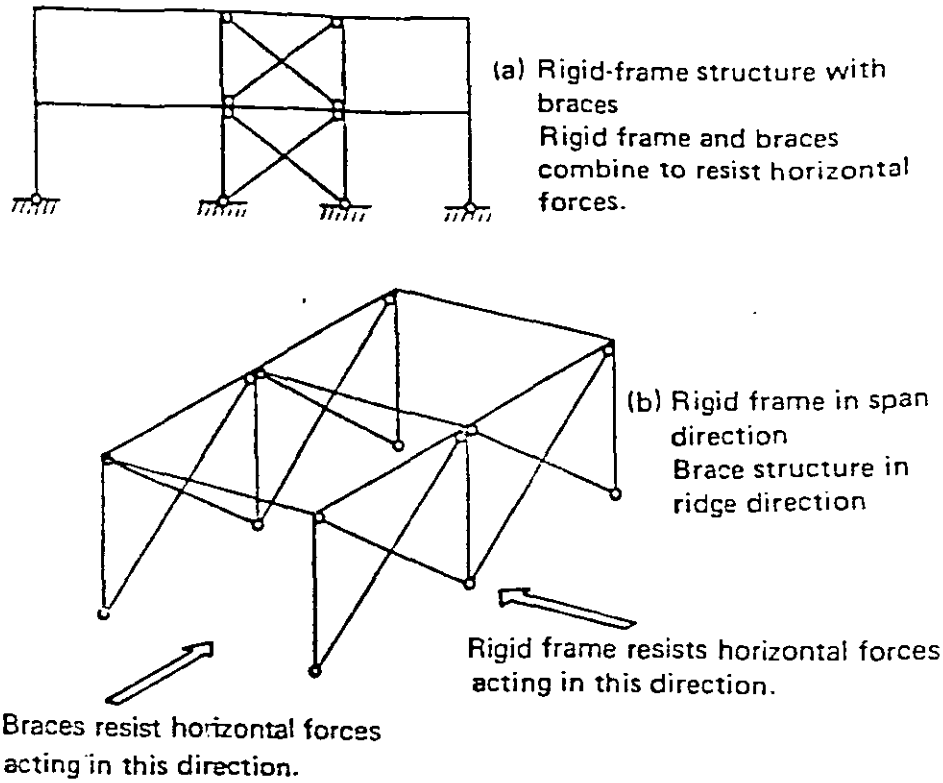


Fig. 2-78. Examples of combined structures

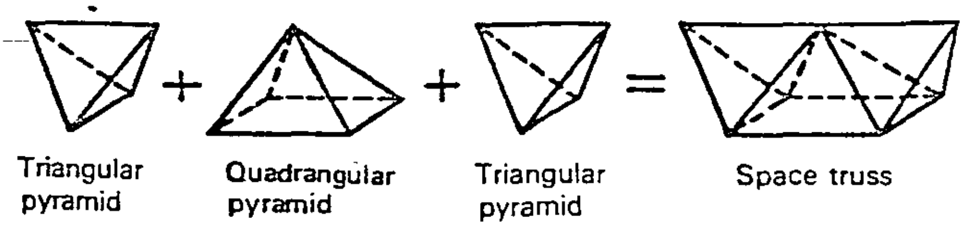
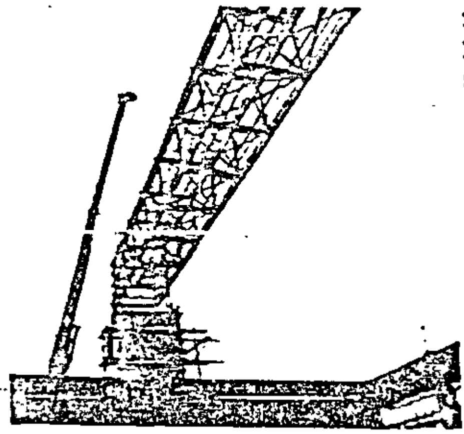


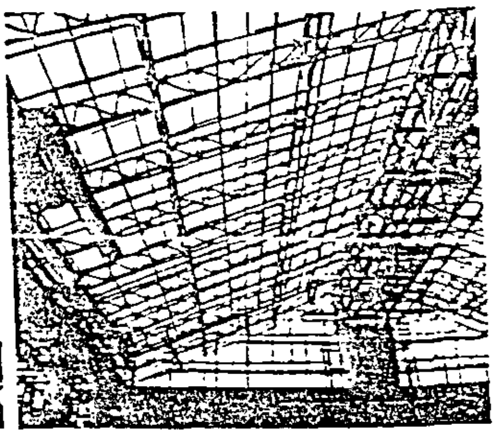
Fig. 2-79. Composition of space truss

2) Examples of the space structure

Space trusses, as a representative type of space structure, are chosen here for discussion. They include the interlacing type, in which truss girders of small height are interconnected; the dense type (Fig. 2-79), which is built up by a close packing of triangular and quadrangular pyramids made of wire rods; and immense truss girders combining plane trusses with triangular and quadrangular prisms (Fig. 2-80).



(a) Installation



(b) Completion of steel-frame erection

Fig. 2-80. Example of space truss girder

2.4.3 Combined Structures

The combined structure incorporates both the rigid frame and the truss. Fig. 2-78 (a) shows a rigid-frame structure fitted with inside braces. The frame shown in (b) consists of a rigid-frame structure in one direction and a truss structure in the other. A frame in which braces are built into some part of a rigid-frame structure is less vulnerable to deformation in the horizontal direction than is a pure rigid-frame structure, and also has greater resistance to column buckling.

A structure having wind- and earthquake-resistant walls playing the role of a truss can be regarded as a combined structure. Such walls in steel structures are made of ferroconcrete or of steel. Trusses equipped with braces and wind- and earthquake-resistant walls collectively are called wind- and earthquake-resistant elements.

2.4.4 Space Structures

1) Characteristics of space structures

All the frames described so far are combinations of single-plane frames, designed by separating the loads acting on the building into those acting in the respective planes of these frames. But it is also possible to design a building frame as a single, integral three-dimensional shape. This is called a space structure, and such structures can be built to enclose a huge space.

Space structures are classified as follows.

Table 2-4. Types of space structure

- Space structures
  - Space frame
    - Components of plane surface
      - Flat type
      - Folded plate
    - Components of curved surface
      - Dome type
      - Shell type
  - Stressed-skin structure
  - Suspension structure
  - Tube structure

Minimizing the types of members used as columns and beams in a rigid-frame steel structure prevents errors and saves labor in construction. This becomes more important as the number of floors of a building increases. It is rare to use different column sections for each floor, but normally the section is changed for every three floors.

As a general rule for the size of members in a pure rigid-frame structure, the column section height/floor height ratio should be in the range of 1/8 to 1/5, while the beam section height/span ratio should be about 1/15 to 1/10. These ratios, however, vary greatly with the actual loading condition, span and floor height, and should be decided according to the circumstances of each specific case.

Another useful point about beams: when working on the structural design, it is important to be aware of the positions, sizes and number of through-beam holes in relation to the progress of the equipment plan. In office buildings, for example, since ducts for the air-conditioning system are arranged above the ceiling board, holes for the ducts are often provided in the beam webs to secure the lowest possible floor height.

(3) Floor structure

There are many types of floor structures for steel-framed buildings. They can be classified by material as follows.

☐ Concrete floor	Prefabricated products: ALC (autoclaved lightweight concrete) slab and PC (precast concrete) slab; Ferroconcrete cast-in-place slab
☐ Steel floor	Keystone plate, deck plate, checkered plate, expanded metal, etc.
☐ Composite floor	A composite of steel floor and concrete floor

The slabs can be classified according to mechanical characteristics as one-way slabs or two-way slabs.

(2) Choice of sections: minimizing the types of members used

In structural design and architectural calculations, as in other industrial fields, computers are now widely used. They make it far easier to make stress calculations, determine sections and find values of deformation and yield strength. In the past, structural designers spent most of the limited time available for design in struggling with computations by slide rule or calculator. The computer is a welcome step forward, giving designers more time to devote to the planning of more rational and economical buildings.

The computer rapidly delivers numerical data and solutions. However, it cannot make engineering judgments, even those that are fairly easy for a designer to make. For example, if the shape, dimensions and design loads of a building are given, the computer will almost immediately print out a correct solution, including a list of members needed. But among the many members specified, there may not be two of the same dimensions.

An experienced structural designer would never specify members in that way. He limits them to a certain number of sizes, and uses members of the same size for similar structures and under similar loading conditions. A computer program that incorporates such engineering judgment can be written, but it would be very complex and in many cases its operation would consume an uneconomically large amount of computer time. Despite the availability and use of computers, therefore, the structural designer's job of taking the results of calculation and applying them according to good engineering judgment has not really changed.

4) Multi-story rigid-frame structures

The design of houses, offices and warehouses often employs the multi-story rigid-frame structure. The dead load is much larger than that of the single-story structure, and for this reason the member sections are normally decided in Japanese practice by the stresses that an earthquake might produce.

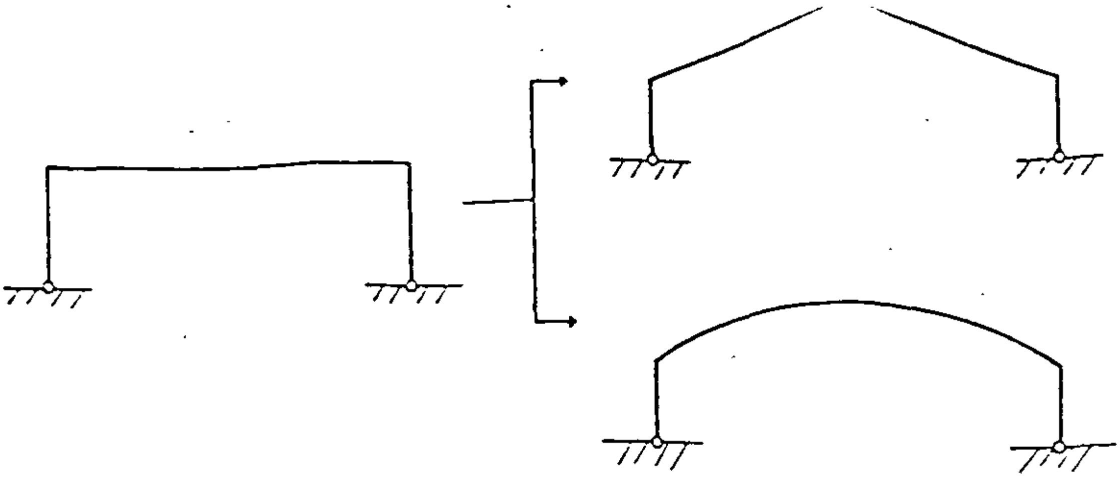
The leading points in planning a multi-story rigid-frame structure are the column spacing and the sections of the members and floor structure.

(1) Column spacing: large or small span?

Column spacing cannot be arbitrarily decided. The shape of the building, number of floors, floor height, intended use and site conditions must all be considered. These factors permitting, choosing a greater span is often preferable. This is because a greater span makes possible large interior spaces that are free of columns and walls. Even if the members are somewhat larger and heavier, they are fewer in total number so that there is no marked increase in total weight. Fewer members also means fewer beam-to-column connections to be made, thus reducing fabrication and construction work.

When wide-flange beams are used as column members, it is sometimes difficult to provide a large span in both the x and y directions because of the anisotropy of wide-flange beams. Often, the problem can be solved by using prefabricated columns made up of wide-flange beams. Square or round tubes may also be used, and are available in a number of types.

The recent trend in office building construction is toward wider column spacing, since large column-free spaces greatly simplifies the major alterations required by a change of tenants or changes in the way office work is done.



For structures having large spans, a gable-type or arch-type rigid frame is advantageous.

Fig. 2-75. Rigid-frame structures with large span

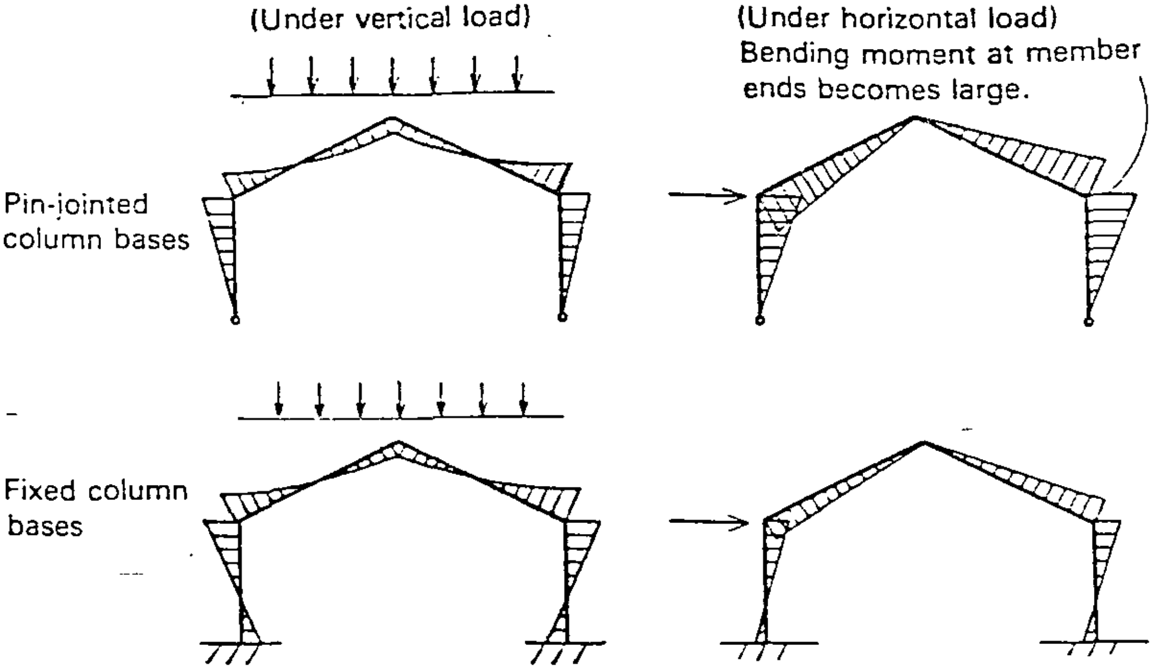


Fig. 2-76. Difference in distribution of bending moment according to type of column base

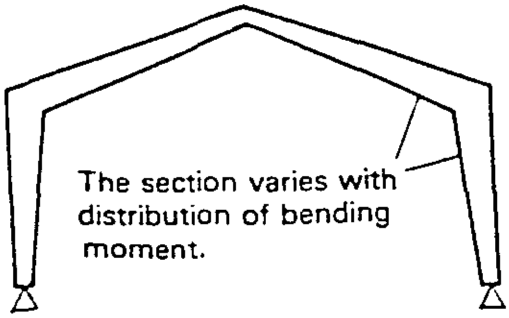


Fig. 2-77. Tapered rigid-frame structure

uneconomical, and there are advantages in terms of structure and flashing in choosing instead a trussed girder, a gable-type rigid-frame structure in which the beam is bent at the center of the span, or an arch-type rigid-frame structure (Fig. 2-75).

Since the dead load of a single-story rigid-frame structure is relatively small, the member sections are determined mainly by the stresses due to wind load. This is particularly true of buildings with high eaves. The light weight of the building permits the use of a smaller foundation, which is economical but requires careful design of the column base. The difference in bending moment distribution between pin-jointed and fixed column bases is diagramed in Fig. 2-76.

The stresses produced in the upper steel-frame members are smaller when the column bases are fixed than when they are pin-jointed. In fact, however, the assumption that a column base is fixed is unrealistic unless the foundation is securely laid on bedrock. Otherwise, the foundation rotates or moves to some extent due to the deformation of the surrounding soil. Therefore, members should be determined by regarding the fixed column base as something between the pin type and the fixed type, or close to the semi-fixed type. A tapered rigid-frame structure, such as that in Fig. 2-77, is possible in which the section varies according to the distribution of bending moment.

In a building with a span longer than its height or greater than about 18 m, a large thrust (horizontal reaction at the column base) is produced. This creates the possibility of the foundation moving horizontally, widening the space between column bases. In such a building, tie members may be used to connect the column bases, or separate foundations or tie beams may be provided.

rangement of purlins and rafters, etc.)

(3) Selection of floor framing method

(4) Determination of design loads

(5) Determination of spacing between rigid frames

(6) Selection of column base type (rigid or pinned)

(7) Selection of members and method of connection (types of steel shapes; use of high-strength bolts or other method)

(8) Provision for reinforcement against lateral buckling and local buckling of members

(9) Checking of secondary stress due to temperature changes

(10) Design of foundation

These points will be discussed further in regard to single-story and multi-story rigid-frame structures, respectively.

3) Single-story rigid-frame structures

Of all rigid-frame steel structures, the single-story structure encloses a given space with the least weight of backing materials and finish materials. It is mostly used for buildings such as factories, warehouses and gymnasiums. The span cannot be chosen indiscriminately since it varies with the type of building (shape, use, etc.) and the loading conditions, but it is said that the shortest span used is about 9 m and the longest is about 60 m. The spacing between rigid frames is generally about 4.5 m to 12 m.

The shape of the structure is in most cases determined by design requirements including the type of roofing materials selected. If the span is large, beam height must generally be large where the design calls for a rectangular rigid-frame structure using solid-web beams. This is

However, when a horizontal force acts on an knee-braced truss and the column bases are pin-jointed to their footings, the bending moment of each column is largest at the point where the knee brace is attached. A better solution is to use fixed column bases and reduce the column bending moments as shown in (c).

It should be noted that when a parallel-chord truss is employed as a roof truss, as in (d), the truss is similar in shape to a large beam and thus the members near the ends of the truss are subjected to anxial forces due to the bending moment at the top of each column.

2.4.2 Rigid-Frame Structures

1) Characteristics of rigid-frame structures

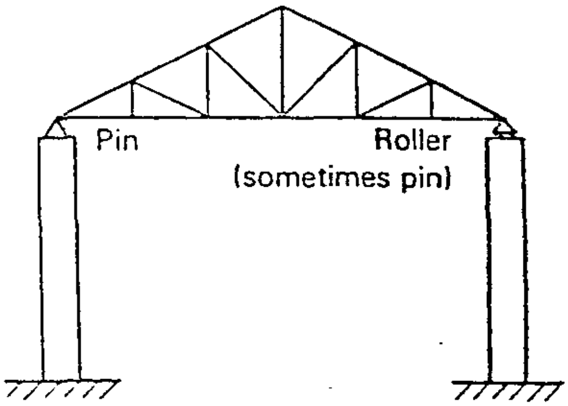
In a rigid-frame structure, beams are connected to columns by rigid joints. The trusses discussed above are all pin-jointed structures. Rigid-frame structures, by contrast, are subjected not only to axial forces but also to large bending moments and shearing forces. Solid-web steel products, including rolled steel shapes such as wide-flange beams and welded steel shapes are used to fabricate rigid-frame structures. As a result, they consume more steel than do truss structures. Their advantages, however, include simplicity of shape, shorter construction time and a lower floor height than the truss structure allows. Accordingly, rigid-frame structures are finding increasing use.

2) Structural design of rigid-frame structures

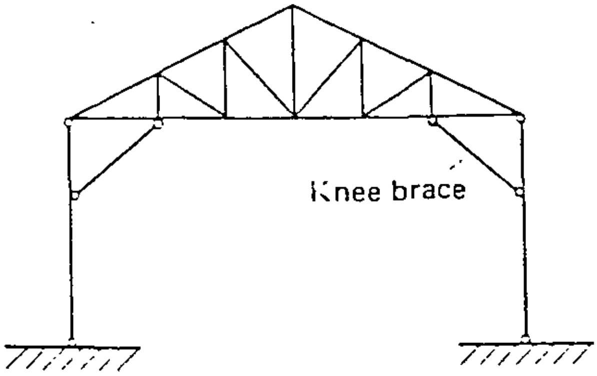
The items involved in designing a rigid-frame steel structure are as follows.

(1) Determination of shape (span, floor height, shape of roof, etc.)

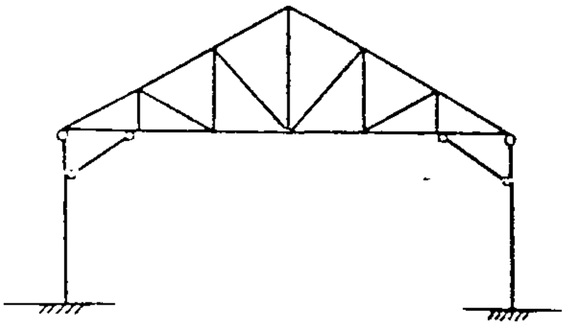
(2) Selection of roofing method (roofing materials, ar-



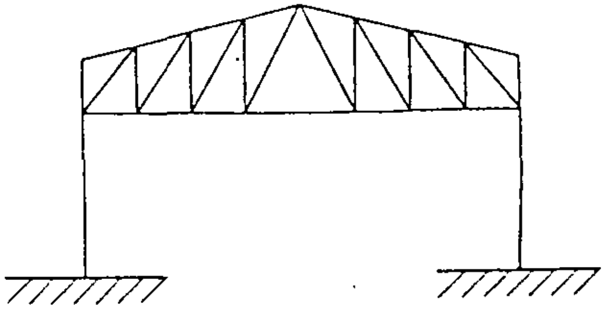
(a) Simple bearing type suitable for low-rise structures with large span



(b) Truss with knee braces (pin-jointed column bases)



(c) Truss with knee braces (fixed column bases)



(d) Extended end-member type

Fig. 2-74. Buildings with roof truss

(2) Roof trusses

The simplest method of supporting the roof truss for a building is to use a pin bearing for one supporting point of the truss and a roller bearing for the other (Fig. 2-74 (a)). To resist horizontal forces acting on the truss, the building columns should be of cantilever type. In this way the column bending moments due to the horizontal forces and the column axial forces due to vertical loads must be transmitted through the foundation to the ground.

Though simple in structure and offering an easy stress analysis since the roof truss and columns are separated, this design approach has the drawbacks that the column section must be thick at the column bases and the foundation must be large. The design shown in (a) is thus suitable for structures with relatively large span and low height which must bear large vertical forces and therefore have large foundations. This holds true even if both supporting points of the truss are pin-jointed. Applications are rather limited, since both the columns and the foundation must be large.

It might be asked if using pin joints for the column bases will solve this problem. Since the structure would be unstable if both the truss supporting points and the column bases are pin-jointed, some method of stabilizing the truss-to-column joints is needed. A conceivable answer is the design shown in (b), where knee braces are added as diagonal members that connect the columns to the truss. They play exactly the same role as the angle braces in a wooden framework or those in a steel floor frame.

3) Structural design of trusses

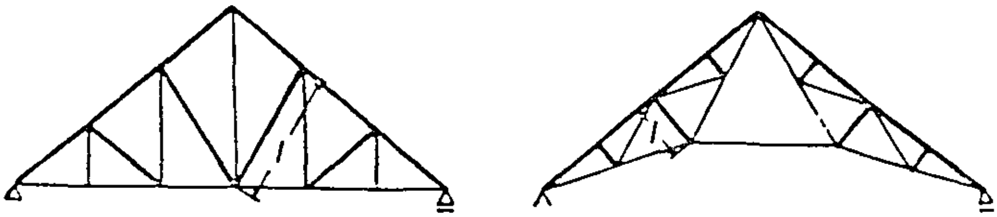
(1) Plane truss and three-dimensional truss

The trusses discussed so far are plane trusses: all their members lie in the same plane, and they can resist only the loads that act within that plane. The truss structures actually used are plane trusses arranged in parallel and connected at certain spans by other trusses, called tie trusses, that act as stays for the main trusses.

Though the phrase "truss structure" usually brings to mind a gabled truss or parallel-chord truss that serves as the main element, the tie truss should not be neglected in making stress analyses. The tie trusses are indispensable in making the most effective use of the main trusses in a structure. In short, a truss structure is a three-dimensional arrangement of plane trusses, each of which resists a design load while supporting one another.

In most cases, the role played by each plane truss in a structure is known, and the forces that act on the three-dimensional truss can be found by dividing it into its component plane trusses for analysis. But some truss structures cannot be divided into plane trusses, due to the method of truss construction or the manner in which the forces are applied. Such structures are called space trusses. A plane truss may be compared to a thin plate with holes, while a space truss is like a plate of considerable thickness.

(a) When pitch is small



(b) When pitch is large

Fig. 2-72. Comparison of the length of compression members between king post truss and Fink truss (subjected to vertical loads)

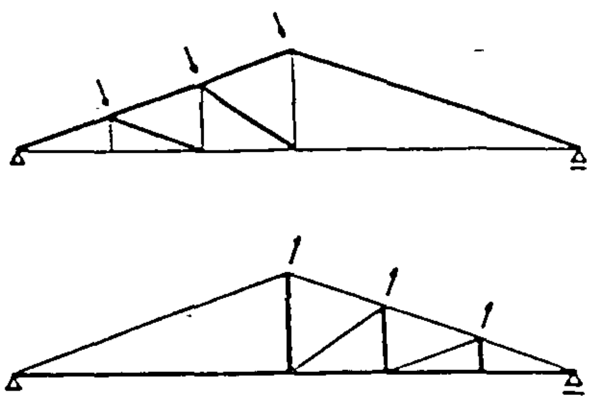


Fig. 2-73. Stress on truss subjected to loads other than vertical ones

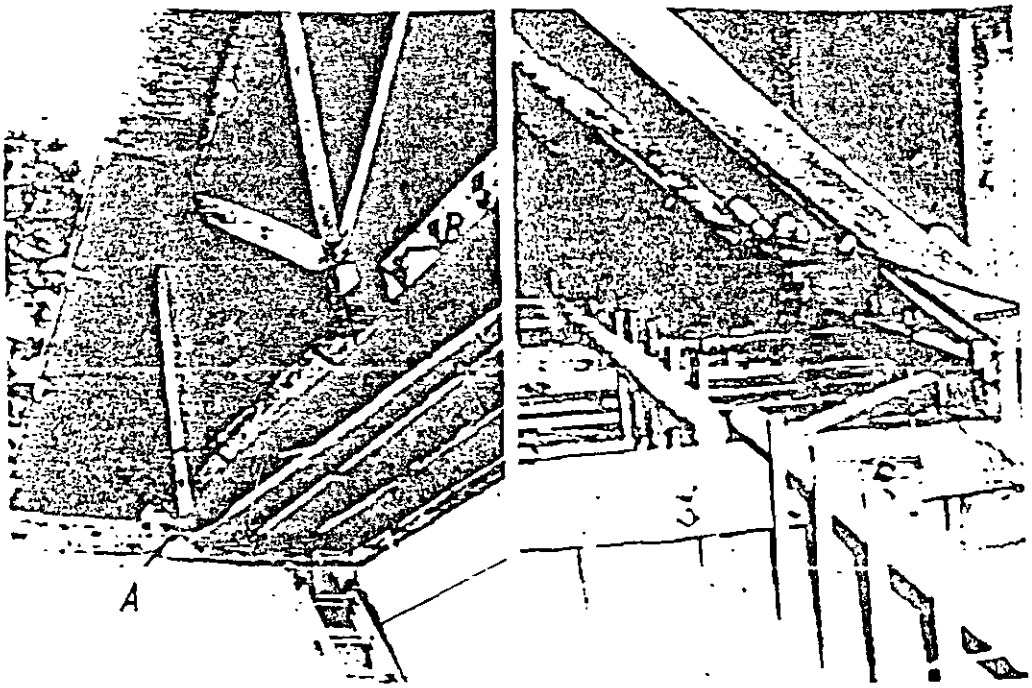
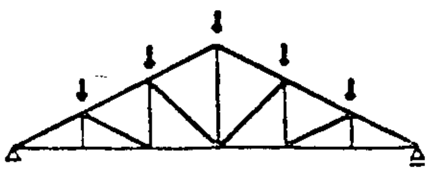
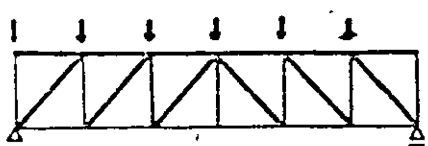


Fig. 2-71. Example of wooden king post truss

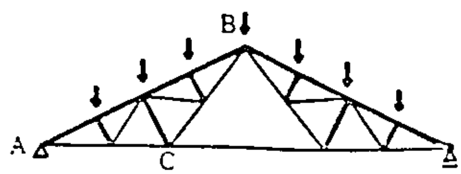


(a) King post truss

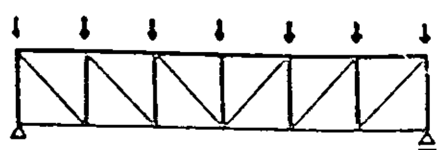


(b) Howe truss

Trusses in which diagonal members serve as compression members when vertical loads are applied



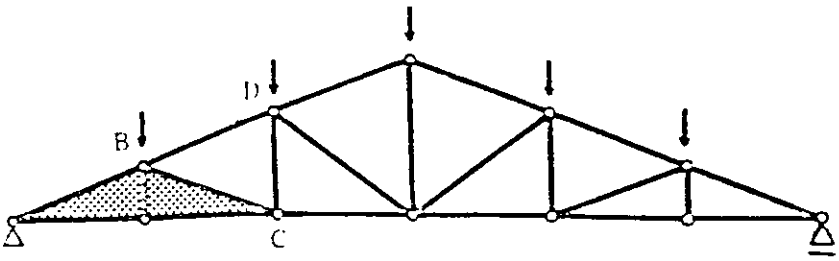
(c) Fink truss



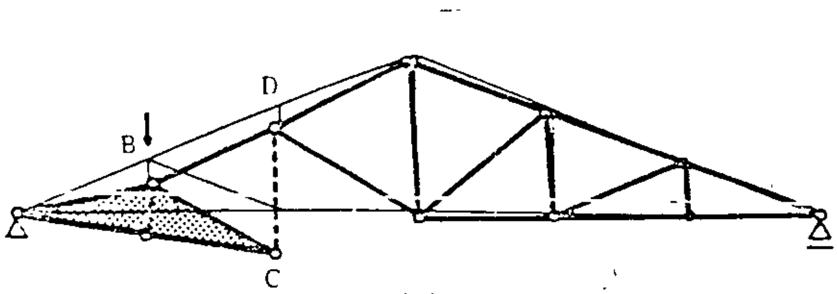
(d) Pratt truss

Trusses in which diagonal members serve as tension members when vertical loads are applied

Fig. 2-70. Families of gabled trusses and parallel-chord trusses



(a)



(b)

Fig. 2-67. Function of post members in king post truss

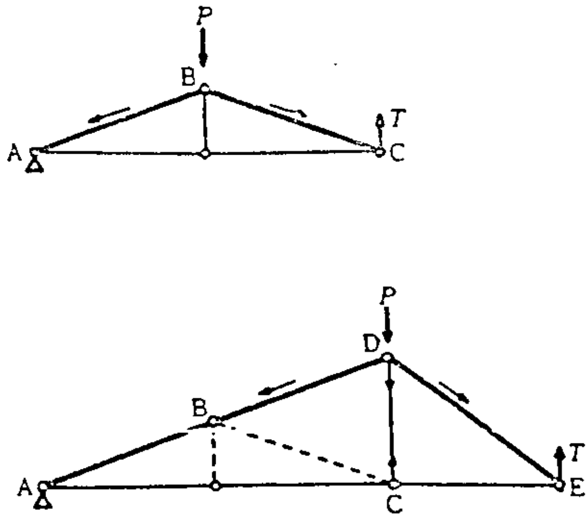
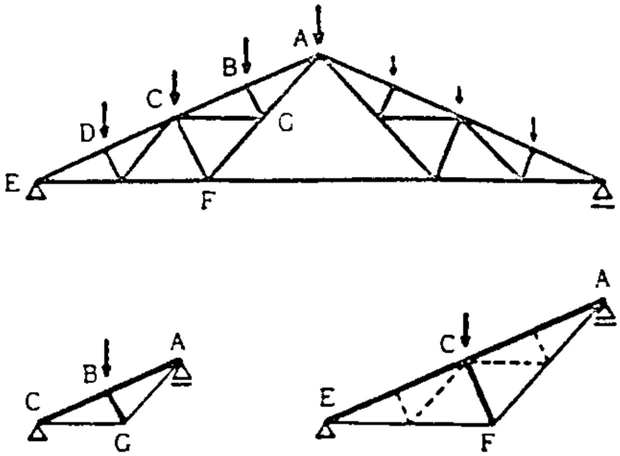
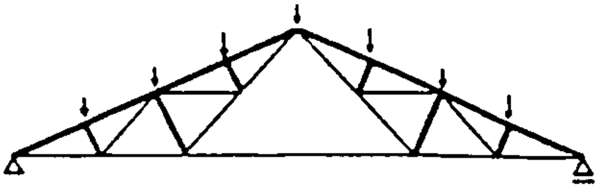


Fig. 2-68. Function of diagonal members in king post truss



(a)



(b) Stress on Fink truss

Fig. 2-69. Flow of forces in Fink truss

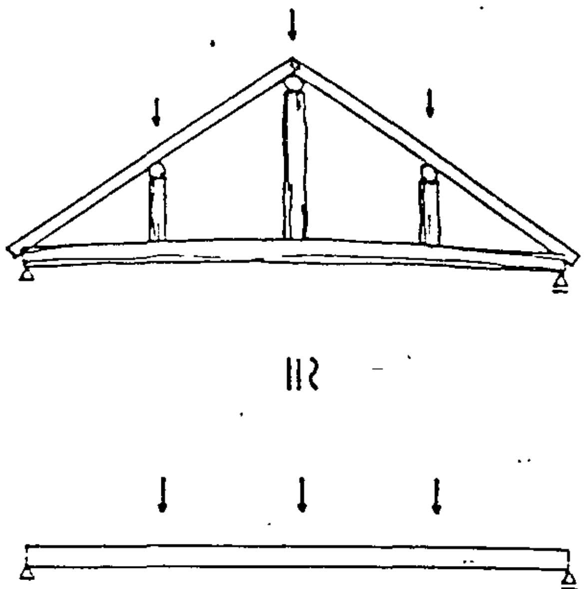


Fig. 2-65. Construction of traditional Japanese roof

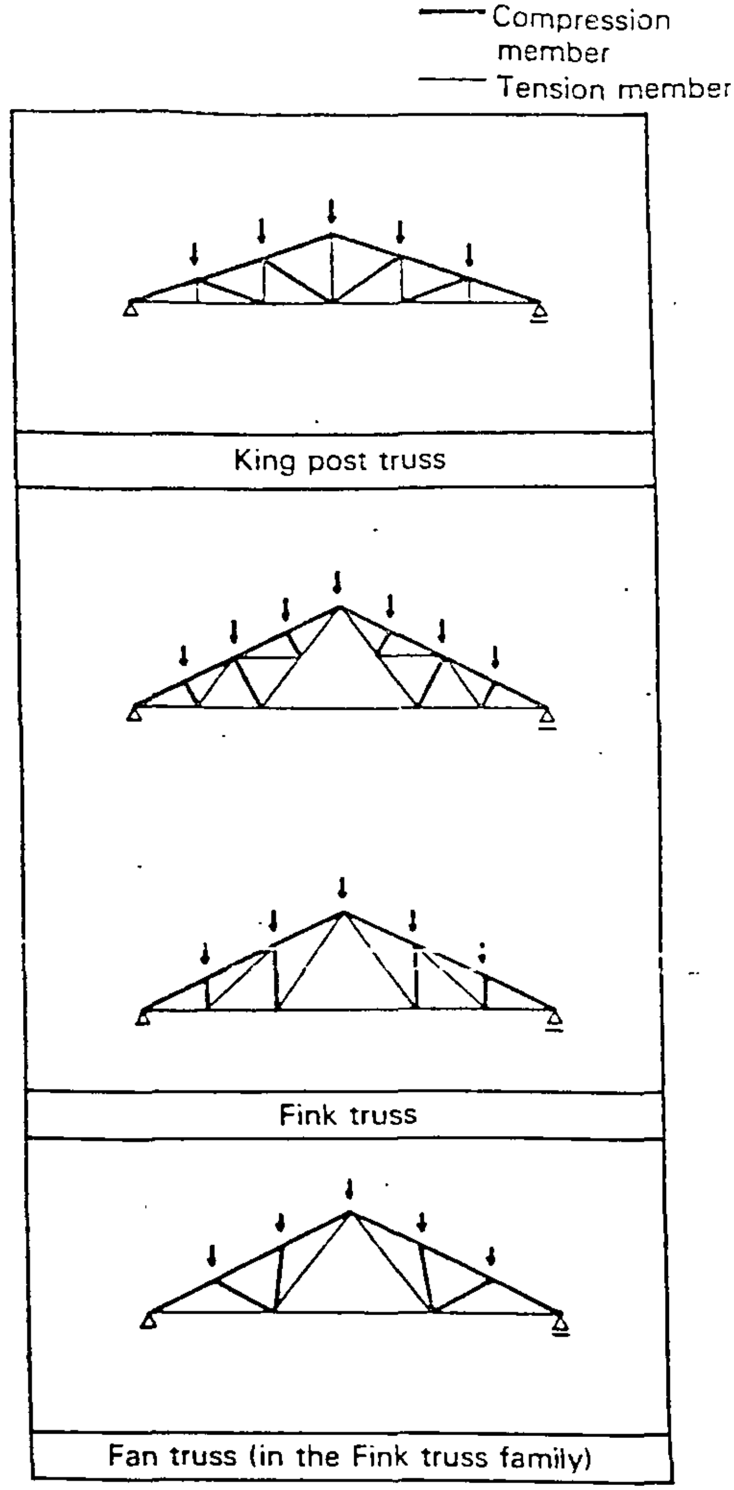


Fig. 2-66. Representative gabled trusses

The king post truss, then, is better regarded as basically a truss for wooden buildings, as is the Howe truss. Shown in Fig. 2-71 is a wooden king post truss supporting the roof of a brick building at Kyoto University. Since the posts receive tensile forces when vertical loads are applied to this truss, the center post is equipped with a metal stiffener at its lower end B to withstand tension, while posts A on both sides are used to suspend the lower chord member through bolted connections. Meanwhile, the tenons provided at joint C of the diagonal members with the central post transmit compressive forces smoothly.

In actual practice, the king post truss is as widely used as the Fink truss in steel-frame buildings. One reason is that the pitch of the roof for such buildings is not generally very large, so that there is not much difference in length of compression members between the king post truss and the Fink truss. This is shown in Fig. 2-72. But the difference becomes pronounced as the pitch of the roof increases. In the king post truss the diagonal members near the center, though subjected to the greatest compressive force, must be considerably longer. In the Fink truss, the struts can be kept short, even for a steeply-pitched roof. The Fink truss is then the better choice, since it makes the most of the merits of steel members.

Stresses in the members of a truss when vertical loads are applied have been described above. Steel-frame trusses are often employed in structures having wide spans, and in that application the dominant loads are indeed vertical. When the truss is subjected to strong winds or earthquake, however, a compressive force may well be produced for a short time in members that are normally expected to bear tensile force (Fig. 2-73). These facts should be kept in mind when examining design conditions.

Looking again at element ABC, it is evident from Fig. 2-68 that a compressive force is applied to diagonal member BC. In truss element ADE, point D is subjected to vertical force P and the tensile force T of the post. The diagonal members DE of a king post truss are compression members. The stress distribution in a king post truss is summarized in Fig. 2-66.

Turning now to the Fink truss, its nature is indicated in Fig. 2-69 (a). As in the Pratt truss, the basic system of the Fink truss is composed of downward-pointing triangular elements. Thus it is clear that, for example, struts BG and CF serve as compression members and that the diagonal members AG and AF are tension members. The axial force diagram can be drawn as in (b).

In this summary of the fundamentals of parallel-chord trusses and gabled trusses, one final point should be emphasized. The fact that there is a geometric similarity between two structures in the way their members are arranged does not necessarily mean that the stresses produced in corresponding members are similar.

In Fig. 2-70, for example, it is seen that the arrangement of members is similar for the king post truss of (a) and the Pratt truss of (d). Also, element ABC in the Fink truss of (c) is similar to the king post truss. Yet, when vertical loads are applied as shown, the diagonal members of the king post truss serve as compression members, but in the Fink truss and Pratt truss they are tension members. Judged by the function of the diagonal members, the king post truss should actually be regarded as a relative of the Howe truss, shown in (b). It is dynamically adequate to regard the Fink truss as a relative of the Pratt truss, by the same logic.

2) Gabled truss

For the gable roofs of steel-frame buildings, gabled trusses are often used rather than the parallel-chord types described above. It should first be noted that the roof of a traditional Japanese house, though a gable roof, is not a trussed roof. In this structure, shown in Fig. 2-65, the load of the roof is transmitted through the posts to the tie-beam. The tie-beam is basically a bending member and the posts do no more than distribute the load. In contrast to a truss, which achieves a balance of axial forces, the Japanese roof "truss" is structurally rather similar to a beam because it supports the load by bending.

Among gabled steel-frame trusses, the most commonly used are the king post truss and the Fink truss, shown in Fig. 2-66, and trusses in their families. The gabled truss, like the parallel-chord truss, is comprised of two types of members: tension and compression members.

How are the stresses produced and how are they balanced? Let us begin with the king post truss, depicted in Fig. 2-67. Triangle ABC, the hatched portion in the drawing, may be considered a basic element of the king post truss subjected to vertical loads. ABC is suspended from post CD, because if post CD were removed from the truss shown in (a), element ABC would begin turning about point B, be deformed as in (b) and collapse. This does not occur because a tensile force is applied to post CD, thus suspending element ABC. Thus the posts of the king post truss that support the loads illustrated are tension members.

When a truss is formed by members of a building frame, buckling of the compression members must be avoided. Basically, the compression members should be short. It is not desirable to apply compressive forces to diagonal members that are longer than the posts. For this reason, the Howe truss is rarely used in steel-frame construction, though it may be considered a suitable type for wooden trusses.

Let us now review the unstable truss in Fig. 2-58. Another way to prevent joint G from moving downward is to place members along diagonal lines FC and CH, thus stabilizing the truss. This concept is illustrated in Fig. 2-61. In this case, since the diagonal members FC and CH bear tensile forces, strut GC serves as a compression member. This type of truss is generally called a Warren truss.

In the Pratt truss, the basic idea is to suspend the joints of the lower chord successively by the diagonal members. In the Howe truss, it is to hold up the joints of the upper chord by the diagonal members. In the Warren truss, the diagonal members are arranged so as to alternately hold up and suspend the joints of the two chords. Figs. 2-62 and 2-63 show examples of multi-span Warren trusses. Note that in both the lower supporting point type and the upper supporting point type, the stresses in posts and diagonals are reversed from the center of the truss outwards.

If the truss is designed so that no vertical loads are applied to joints Ⓐ, Ⓑ and Ⓒ through the posts (Fig. 2-64 (a)), these posts remain unstressed and thus can be eliminated to create the extremely simple Warren truss shown in (b). Such a truss, with fewer members intersecting at the joints, offers advantages such as ease of connection of the frame members and is also used for bridges. Since in this type of Warren truss the upper chord members receive compressive forces, care should be taken to avoid buckling by adding reinforcement.

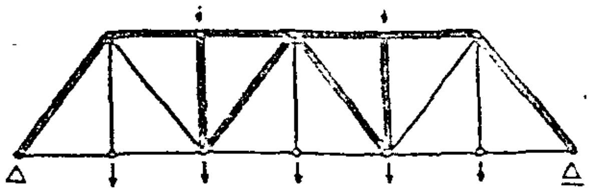


Fig. 2-62. Warren truss of the lower supporting point type

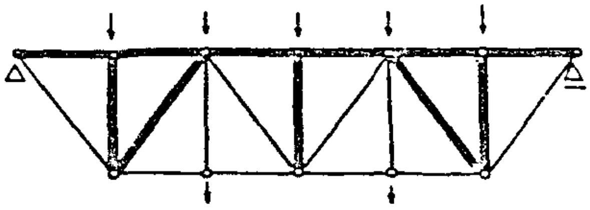
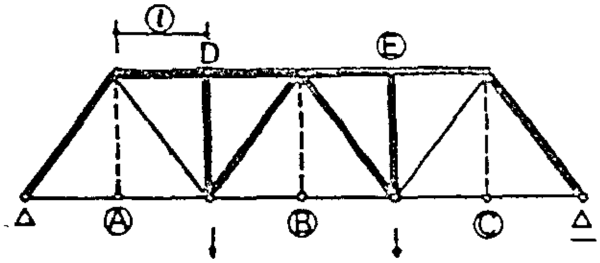
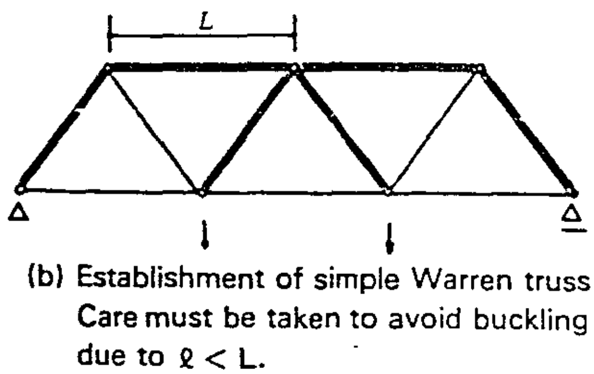


Fig. 2-63. Warren truss of the upper supporting point type

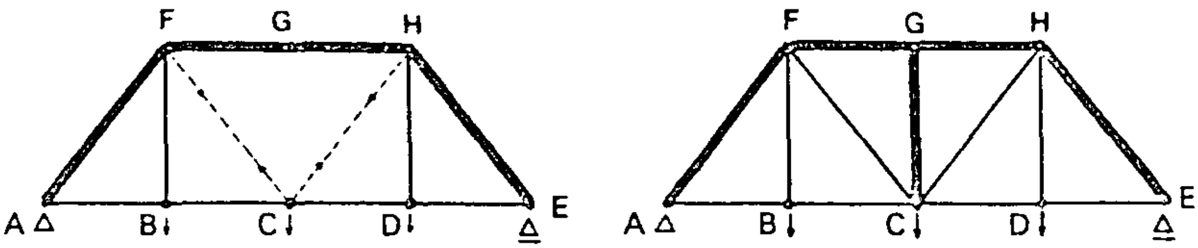


(a) If the truss is designed so that no loads act on joints A, B, C, D and E, the posts become unstressed members.



(b) Establishment of simple Warren truss Care must be taken to avoid buckling due to $\ell < L$.

Fig. 2-64. Transformation of Warren truss



(a) Stabilization of truss where the vertical movement of the joint C of the bottom chord is restrained by the disposition of slanting members along the directions FC and CH.

(b) Basic pattern of Warren truss

Fig. 2-61. Concept of Warren truss

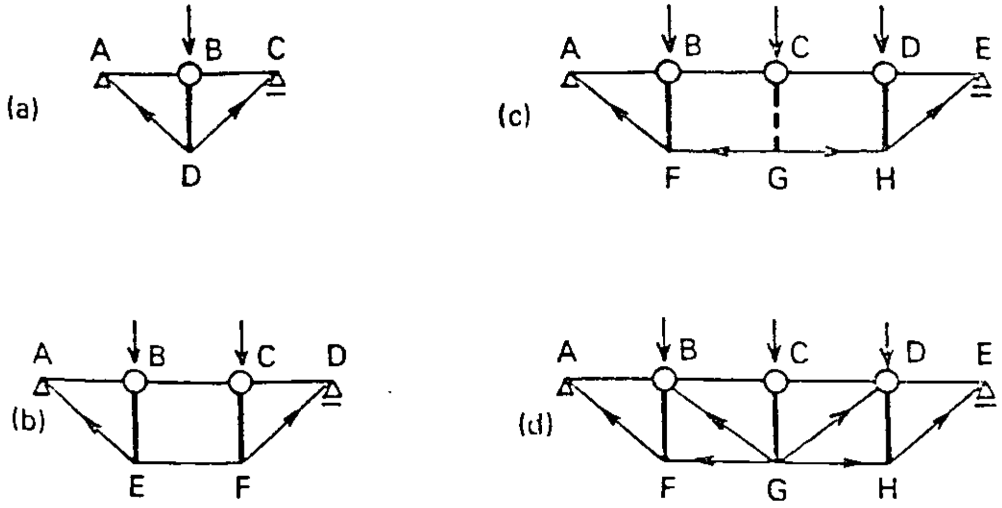
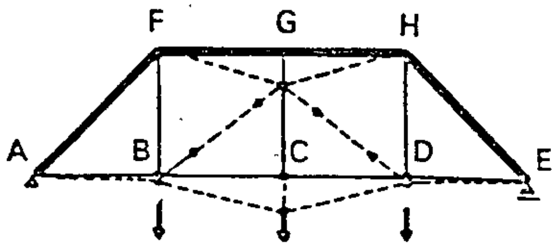


Fig. 2-57. Balance of forces in the multi-span Pratt truss



Place diagonal members along directions BG and GD and the joint of the upper chord can be restrained from moving vertically.

Fig. 2-58. Stabilization of truss

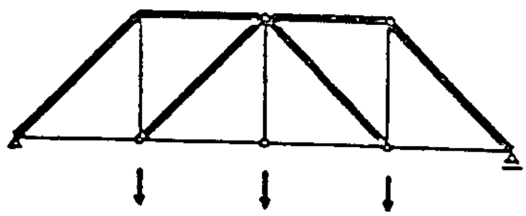


Fig. 2-59. Concept of Howe truss

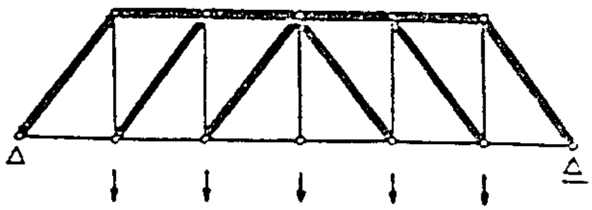


Fig. 2-60. Multi-span Howe truss

In the four-span truss shown in (c), however, the center strut is suspended idly and does not function as part of the load-bearing structure, even though it is formed by the same method as the trusses in (a) and (b). The tensile forces in the lower chords GF and GH are not joined by the vertical component of force from post CG, and the truss cannot resist that load. Thus further reinforcement is required for the square portion BDHF. If portion BCD, regarded here as a two-span cross-beam, is replaced with the member shown in (a), a balance of forces is achieved and the resulting truss (d) is stable. The Pratt truss thus features a rational balanced system achieved by the combination of tension members and compression members, in addition of the dynamic beauty of the truss itself.

(2) Howe truss and Warren truss

Let us now consider trusses where the supporting points are on the lower chord, as shown in Fig. 2-58, rather than on the upper chord as for the Pratt trusses described above.

The truss in Fig. 2-58 is unstable; the plane BDHF is easily deformed into the shape indicated by dotted lines. Some method is needed to restrain strut GC from moving vertically. This can be accomplished by placing members along the diagonals BG and GD, which prevents joint G in the upper chord from moving lower. Thus is the truss shown in Fig. 2-59 formed. The diagonal members serve as compression members as they function to hold up the upper-chord joints.

This concept can be applied to a multi-span truss in the same manner as multi-span Pratt trusses are designed, and an example is shown in Fig. 2-60. This type of structure is usually called a Howe truss. It differs from the Pratt truss mainly in that the functions of the posts and diagonal members are reversed. In the Pratt truss, posts receive compressive forces and diagonals receive tensile forces. In the Howe truss, posts receive tensile forces and diagonals receive compressive forces.

SECTION 2.4 TYPES OF BUILDING FRAME

4.1 Truss Structure

In a truss structure, the stresses to which the members are subjected are mainly axial forces. It is thus higher in sectional efficiency than structures which include bending members such as beams. Because of this fact and its simple, rational shape, the truss structure is widely used in steel-frame construction.

Truss structures can be broadly classified according to the arrangement of the members, into statically determinate .d indeterminate trusses. Calculation of the stress acting in the members is easier for statically determinate trusses. This section describes the basic features of the two most important types of statically determinate trusses: the parallel-chord truss and the gabled truss.

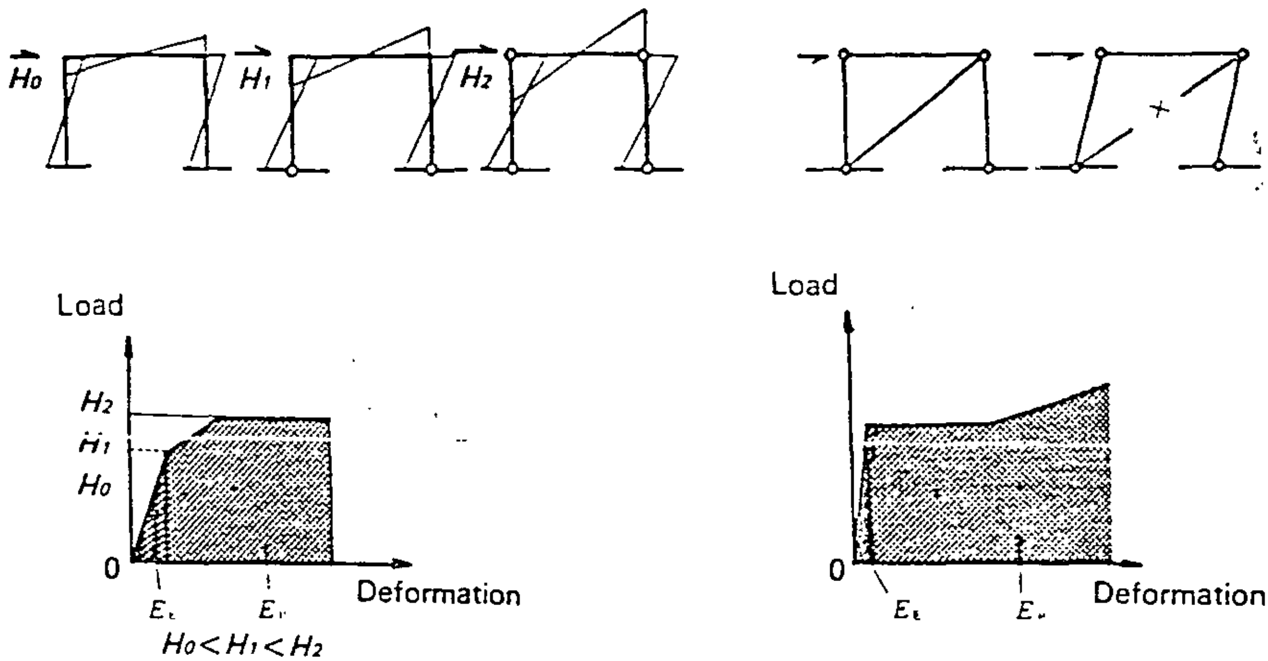
1) Parallel-chord truss

Typical of the parallel-chord trusses commonly used in steel-frame buildings are the Pratt truss and the Warren truss.

: Pratt truss

In the Pratt truss, the upper chord member and the posts serve as compression members and the lower chord member and diagonal members are tension members. When a load is applied to a two-span Pratt truss, as in Fig. 2-57 (a), a rational balanced system can be obtained in which the compressive force in the post is guided from point D to the diagonal members connected to A and C, and the compressive reaction produced is borne by the upper chords. ıe same kind of balance can be achieved for a three-span truss, as in (b).

considerable plastic deformation until the brace itself breaks. Therefore, provision can be made to absorb a considerable amount of consumed energy E_p. In short, by giving the connections of a braced frame a high-strength design the frame's capacity to absorb earthquake energy can be greatly increased.



(a) Statically indeterminate rigid-frame structure

(b) Statically determinate braced structure

Fig. 2-56. Load and deformation for a braced structure

2.3.5 Connections of Braces

Recent research in Japan on the earthquake resistance of the braced steel-frame building has clarified a number of problems. As a result, design policy for connections is changing.

Generally, if a building is subjected to a strong earthquake, such as the strongest that occurs in several dozen years, the elastic vibration limit of the building is exceeded and local plastic deformations occur. The energy of such an earthquake is sometimes so large that the entire building undergoes plastic deformation. It is of course possible to design a building frame that maintains its elasticity even in earthquakes of great size that occur very rarely. Such a building, however, would be very expensive.

Therefore, designers are now shifting to the ultimate-strength design method, in which the frame is allowed to undergo plastic deformation to some extent. The plastic work of the members internally dampens the vibration, thereby reducing the response to an earthquake and increasing the earthquake resistance of the building.

In general, the relation between load and deformation for a rigid steel frame is that shown in Fig. 2-56 (a). It is known that even if part of the frame yields under load H_1, the stress is redistributed and the yield strength increases up to load H_2 at which the next member yields. Accordingly, the energy E_p which is consumed in causing the frame to yield completely is far greater than the potential energy E_E which the structure can store before its elastic limit is reached. This is true of all statically indeterminate structures.

In the braced frame shown in Fig. 2-56 (b), on the other hand, the horizontal force is borne entirely by the brace. In such a statically determinate frame, when the connection of the brace is broken the frame instantly loses its resistance to the horizontal force and collapses. However, when the offset yield stress of the connection is greater than the yield axial force of the brace, the brace connection will undergo

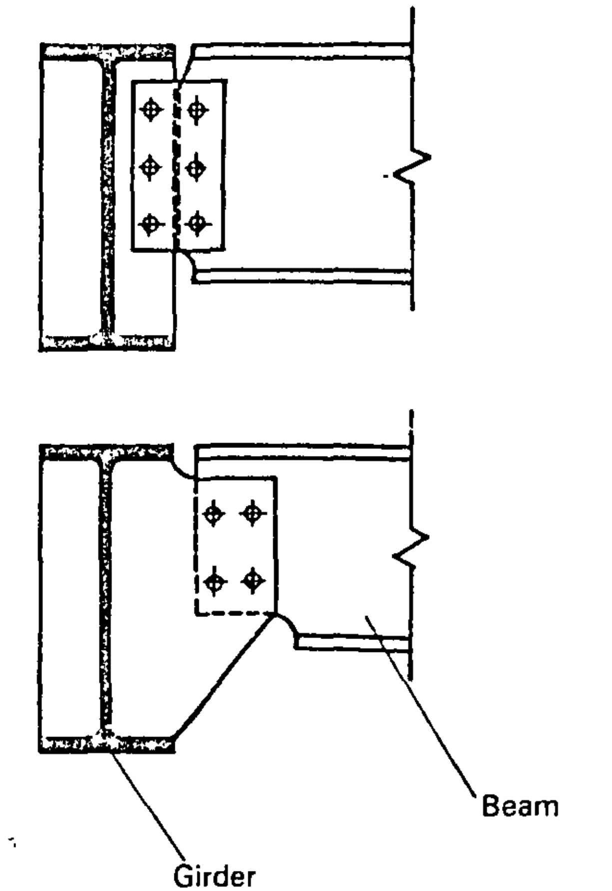


Fig. 2-54. Examples of beam-to-girder connections

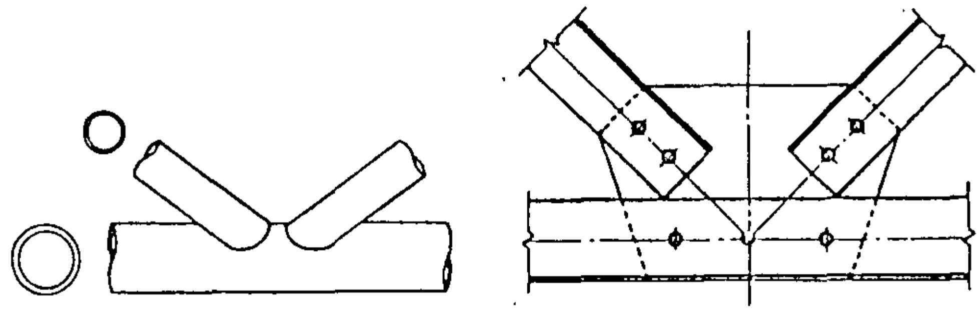
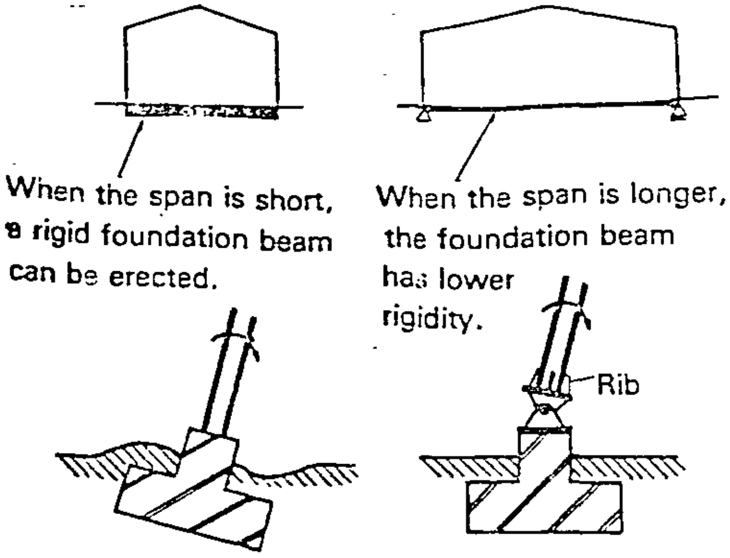


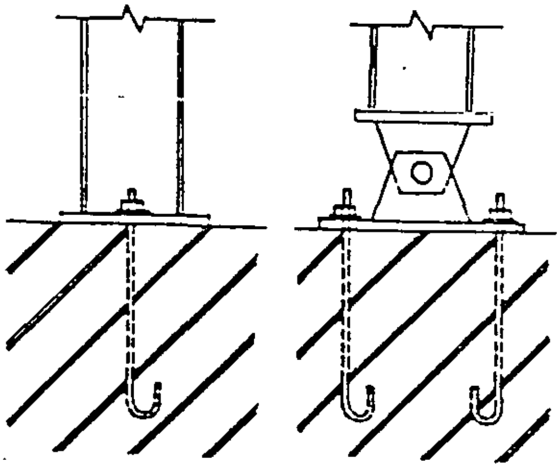
Fig. 2-55. Details of truss joints



(a) Even when the column base is fixed, the foundation can rotate.

(b) Pin-type column base prevents the foundation from rotating.

Fig. 2-52.



(a) Simple pin-type column base (for a small-scale structure)

(b) Perfect pin-type column base

Fig. 2-53. Examples of pin-type column bases

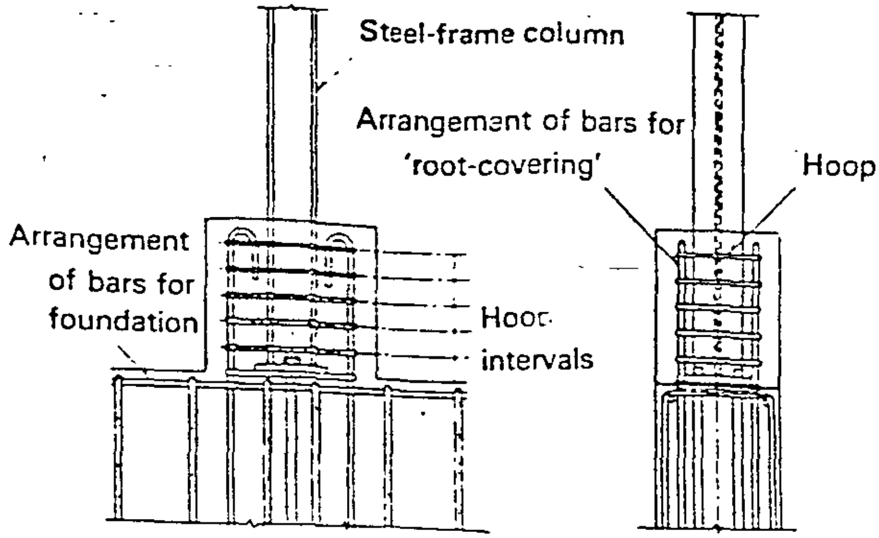


Fig. 2-49. Example of 'root-covered' column base

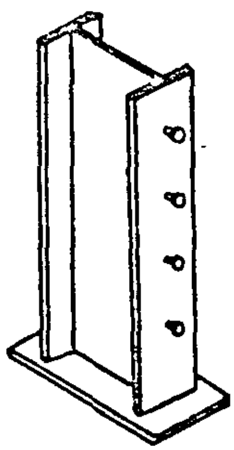


Fig. 2-50. 'Root-covered' column base with studs

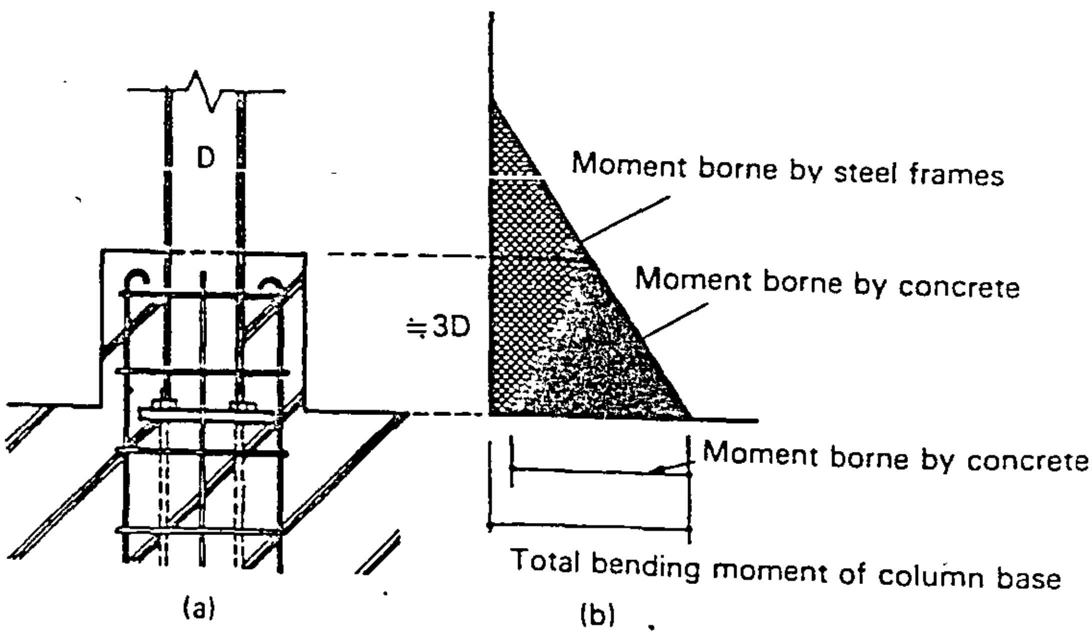
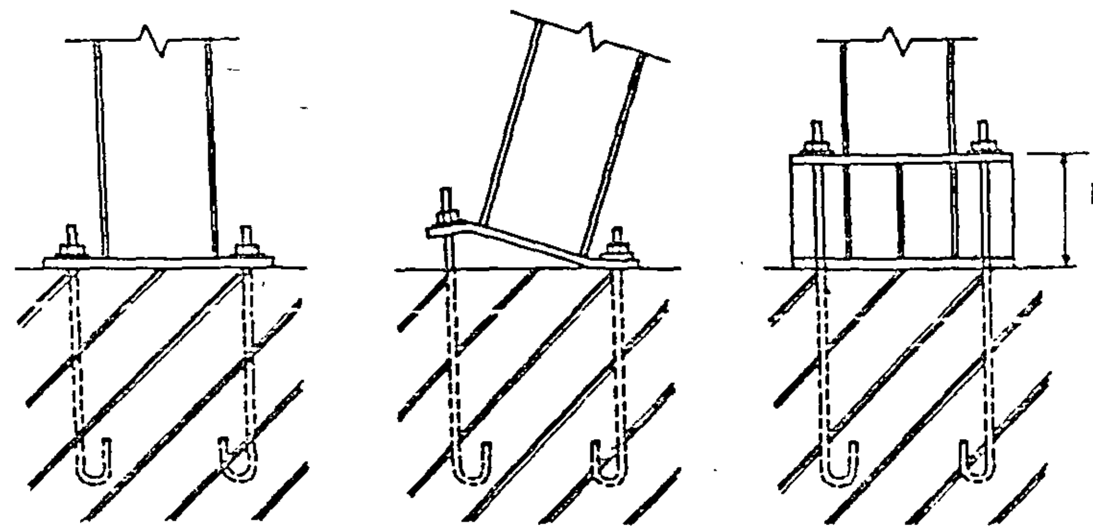


Fig. 2-51. Distribution of design moment for 'root-covered' column base



(a) When the moment is small

(b) Deformation of base plate

(c) Long-bolt type

Fig. 2-48. Examples of fixed column bases for steel-frame structure

The pin bearing is often used intentionally in steel-frame buildings. It is generally believed that a rigid joint, which has a high degree of redundancy, increases the reliability of the structure. In some cases, however, reliability can be improved by use of a type of structure having a clear boundary condition, such as the pin-joint structure. When a semi-rigid joint is adopted despite the assumption in analysis that a rigid joint will be used, the pin joint is the desirable choice in most cases since it also lessens the effects of secondary stress caused by temperature or uneven settling.

It should be noted that beam-to-girder connections, truss joints and brace joints are usually treated as pin joints. Fig. 2-54 illustrates beam-to-girder connections. Normally, it is sufficient to connect the web of the beam through a gusset plate to the girder for transmission of the shearing force.

Fig. 2-55 shows details of truss joints. In stress calculation of a truss it is assumed that the joints are pin joints — that is, the joints are not supposed to resist bending. In practice, however, true pin joints are rarely used. In most cases, a joint is connected with two or more high-strength bolts and a gusset plate, or is welded. Yet, even though truss joints are thus close to being rigid joints, truss behavior coincides roughly with calculations made on the assumption of pin joints. This has been confirmed, but it is best to design so that the central axes of the members intersect at one point as much as possible. Otherwise, secondary stresses such as bending moment and shearing forces will be produced at the joint.

The column base, being embedded in ferroconcrete, has very high rigidity. But if it undergoes repeated loading, as in an earthquake, the ferroconcrete may be damaged, reducing the stability of the foundation. To avoid this problem, two measures are recommended. The "root covering" should be provided to a sufficient depth ($\geq$ 3D, D: column width), and stud connectors or the like should be welded to the column base to ensure unity with the surrounding ferroconcrete (Fig. 2-50).

In many cases, the "root-covered" column base is designed to let the reinforced-concrete structure bear most of the bending moment by treating the steel column base as a pin-type base. Even in such a case it is considered necessary to make the column base almost a fixed base, as in Fig. 2-51 (a), so as to provide it with reserve yield strength.

2) Pin-type column base

Steel construction is widely employed for large-span buildings. In the structural design of such a building, it is difficult to design a footing beam having high rigidity. The foundation must be a tie-rod type, aimed at preventing opening of the frame, or an independent type (Fig. 2-52). For both types of foundation, neither of which has resistance to rotation, the pin-type column base is always used. Fig. 2-53 shows such bases. For small steel-frame buildings, the type shown in (a) can be used, but for larger buildings the type shown in (b) is preferable.

2.3.4 Column Bases

The column bases of a building are highly important structural elements since they transmit the vertical load of the frame to the foundation and are also required to withstand large horizontal shearing forces at times of earthquake or strong wind.

The column bases of a steel-frame building are acted upon by a complex stress pattern because they are the contact point between two different kinds of structure: the reinforced-concrete foundation and the steel superstructure. This is liable to cause problems at the work stage if due care is not taken.

The two classes of column bases — the fixed type and the pin type — are described below.

1) Fixed column base

Typical fixed column bases are shown in Fig. 2-48. When the bending moment of the column base is small, a base plate welded to the column section and fixed by anchor bolts to the foundation, as shown in (a), is sufficient. If the base plate is thin, however, it is subject to bending deformation such as that depicted in (b) and must be reinforced with ribs. If the bending moment of the column base is much greater, it is better to use a long-bolt column base as in (c), where the base plate is a wide-flange beam or other member having a highly rigid section.

Another design method for fixed column bases is the "root covering" approach. This method, as indicated in Fig. 2-49, is to cover the steel column base with ferroconcrete, which is a kind of steel-reinforced concrete structure.

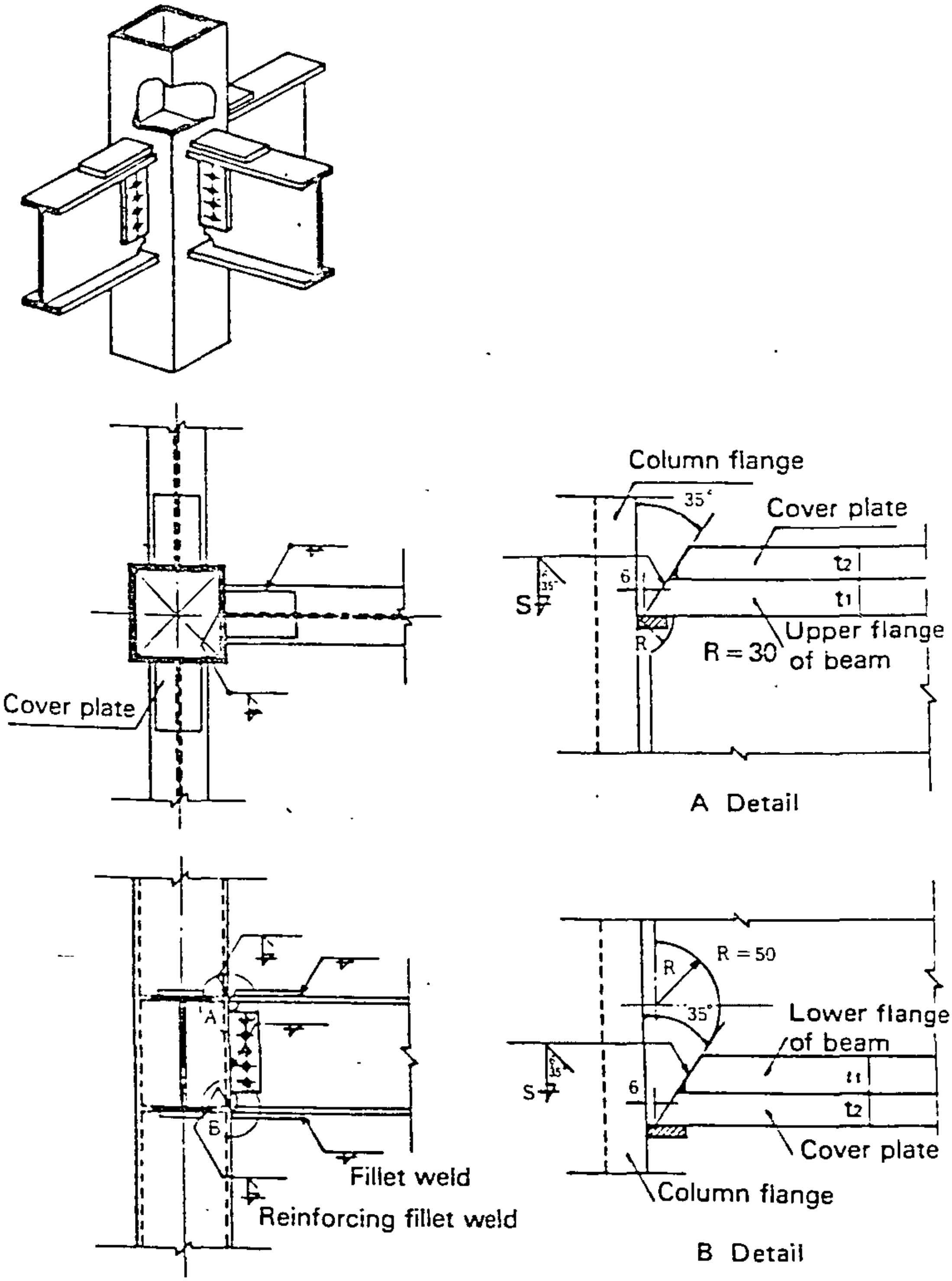


Fig. 2-47. Details of field connections

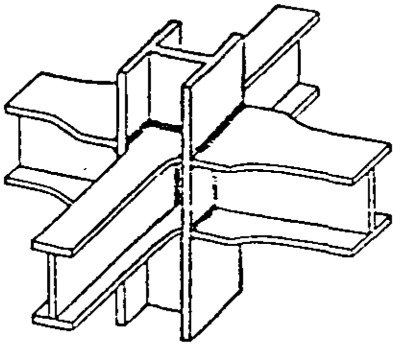


Fig. 2-45. Example of bracket-type connection using wide-flange beams

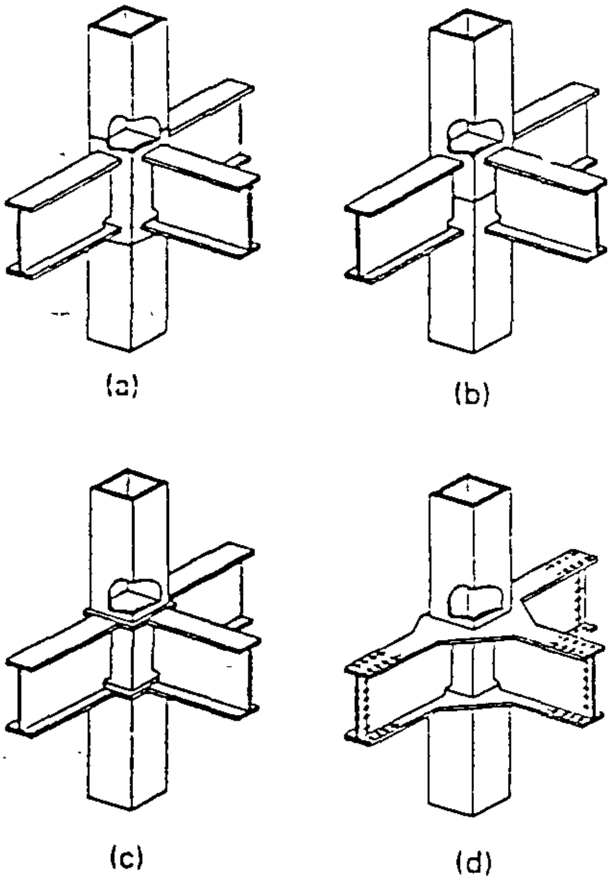


Fig. 2-46. Types of connections using square tubes

4) **Examples of beam-to-column connections**

Fig. 2-45 shows a bracket-type connection using wide-flange beams. When square tubes are used for the columns, fabrication of their connections is far more complicated than in the case of wide-flange beams. The connections now in use can be classified into three types, shown in Fig. 2-46.

(1) Column piercing type

☐ In this type of connection, the beams enter the column. To accomodate horizontal stiffeners inside the connection, the column may be divided at two locations as in (a) or at one location as in (b).

(2) Horizontal stiffener piercing type

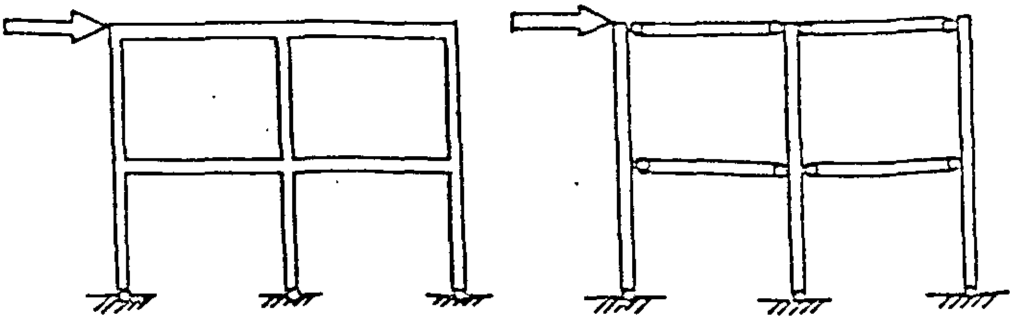
☐ In this type, shown in (c), the horizontal stiffeners extend outside the connection and are welded directly to the beam flanges. It is sometimes called a column piercing type also, since in appearance the beams seem to enter the column.

(3) Outside stiffener type

☐ Instead of stiffeners positioned inside the column, this type uses horizontal haunches at beam ends, shown in (d). The axial forces in the beams go round the column and are transmitted through the circumferential stiffeners.

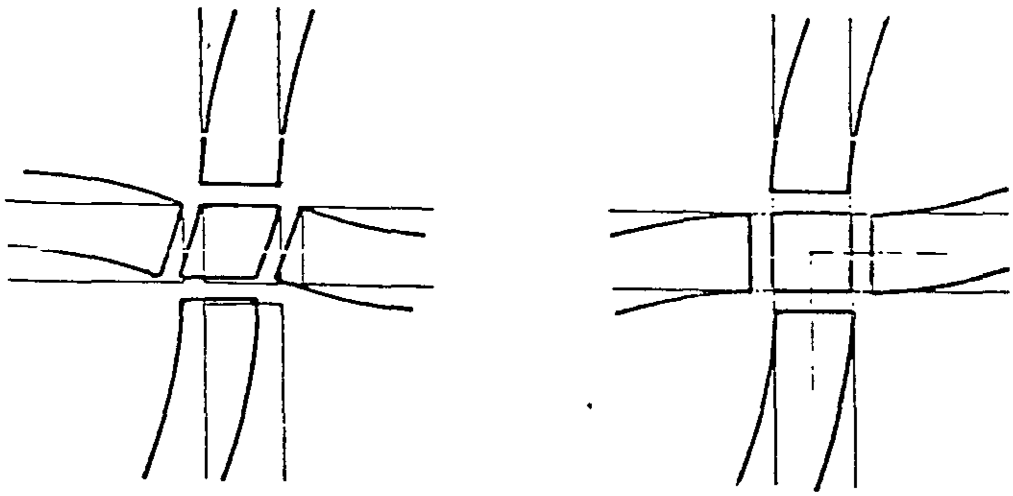
Fabrication of beam-to-column connections should be carefully done, since the large bending moment and shearing force that occur at the ends of the beams must be transmitted smoothly to the columns. Fig. 2-47 shows an example of field connection. Beam flanges should be butt-welded to the column so that their whole sectional area is effectively used. It is necessary in welding to secure an adequate edge root gap and to use end tabs for complete reliability. It should be remembered that in field connection especially the working condition is bound to be worse for the lower flange than for the upper one.

To connect the webs, bolting or fillet welding is used in many cases. If design conditions are demanding, however, butt welding may be necessary and a reinforcing cover plate is often used for the beam flanges.



(a) Statically indeterminate frame (b) Unstable frame

Fig. 2-43. Resistance to horizontal forces of joints and rigid frame



(a) When panels are flexible (b) When panels are rigid

When panels are subjected to a shearing deformation, the relative story displacement increases far more in (a) than in (b).

Fig. 2-44. Increase of relative story displacement due to shearing deformation of panels

3) Rigidity of connections

An important point to note is that the earthquake resistance of a steel-frame structure varies with the design of the connections, even when the same kinds of structural members are used. An extreme case is shown in Fig. 2-43. Though members of the same kinds are used and the column bases of both frames are pin-fixed, framework (b) is obviously unstable, with no resistance to horizontal forces, while framework (a) is a rigidly-jointed, statically indeterminate frame with a high degree of redundancy and fairly high resistance to horizontal forces.

This distinct difference between the two frames is determined by a very simple criterion: whether or not the beam-to-column joints are rigid connections, that is, whether the connections are designed to transmit the bending moment. In rigid-frame construction, the rigidity and yield strength of the connections are the most vital factors in the structure's resistance to horizontal forces. Beam-to-column joints should be regarded not as mere connecting points of members, but as wind- and earthquake-resisting elements in the rigid-frame structure.

Moreover, the moment distribution method and the slope-deflection method commonly used in stress analysis of the framework, as well as the ordinary rigidity matrix method, are all based on the premise that the joints are rigidly connected. If a connection of insufficient rigidity is subjected to a shearing deformation, as depicted in Fig. 2-44, the relative story displacement is increased, and this is clearly undesirable.

In designing the connections of beams to columns, therefore, adequate rigidity should receive as much attention as adequate strength. Only then can be beams and columns that comprise the framework deliver their full sectional performance. From this standpoint, provision of a haunch at the end of the beam is very advantageous for wind- and earthquake-resistant design.

The panel should be safe if

$$\tau \leq f_s \qquad (2\text{-}22)$$

where f_s = unit yield shearing stress. If $\tau > f_s$, the panel thickness t is insufficient and the panel must be reinforced. Fig. 2-42 shows an example of how the connection panel may be reinforced. In this method a doubler plate is attached to the panel by full-circumference welding so as to increase the panel thickness. The doubler plate is normally fillet-welded around its circumference.

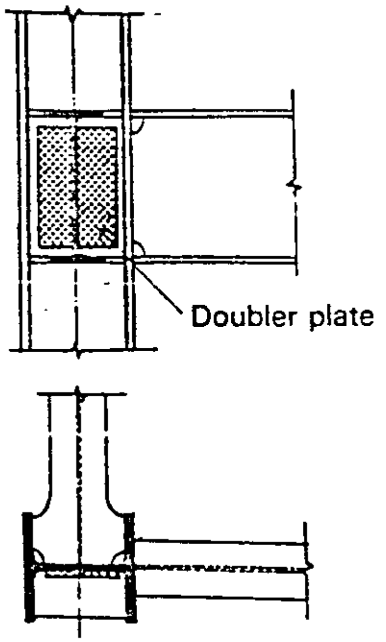


Fig. 2-42. Example of reinforcement of connection panel using a doubler plate

Therefore, the following relation holds true between the axial force in the beam and the shearing force in the column.

$$N_A - N_B = Q_C - Q_D \qquad (2\text{-}18)$$

Substituting Eq. 2-18 into Eq. 2-15, we have

$$\begin{aligned} Q_1 &= \frac{1}{h_b}(M_A + M_B) + \frac{1}{2}(Q_C - Q_D) - Q_C \\ &= \frac{1}{h_b}(M_A + M_B) - \frac{1}{2}(Q_C + Q_D) \end{aligned} \qquad (2\text{-}19)$$

Likewise, substituting Eq. 2-18 into Eq. 2-16,

$$Q_2 = \underset{\text{(beam term)}}{\frac{1}{h_b}(M_A + M_B)} - \underset{\text{(column term)}}{\frac{1}{2}(Q_C + Q_D)} = Q_1 = Q \qquad (2\text{-}20)$$

These equations show that the connection panel is deformed by a shearing force equal to the couple of forces related to the beam bending moment minus the average of the shearing forces from the upper and lower floors. This is illustrated in Fig. 2-41 (e), and (f) shows the typical stress condition of the connection panel. The external shearing force Q is balanced by the shearing resistance of the panel, as shown in (g).

If the average unit shearing stress is denoted by τ, then $t \times h_c \times \tau = Q$. Therefore, τ can be evaluated as

$$\begin{aligned} \therefore \quad \tau &= \frac{Q}{t \cdot h_c} = \frac{1}{t \cdot h_c}\left\{\frac{1}{h_b}(M_A + M_B) - \frac{1}{2}(Q_C + Q_D)\right\} \\ &= \frac{1}{t \cdot h_c \cdot h_b}\left\{(M_A + M_B) - \frac{h_b}{2}(Q_C + Q_D)\right\} \end{aligned} \qquad (2\text{-}21)$$

2) Connections under horizontal load

A typical bending moment distribution in a rigid-frame structure under horizontal load is diagramed in Fig. 2-41 (a), and the distribution around a beams-to-column connection is shown in detail in (b). Note particularly that the bending moment of either beam or column suddenly changes its sign within a small area of the connection panel zone. Since the gradient of a bending moment diagram usually corresponds to the shearing moment, it is apparent that a stress field in which shearing force dominates is formed in the panel zone. Fig. 2-41 (c) indicates the stresses acting on the connection panel zone; they are identified by subscripts referring to the various members.

If the bending moment acting on the beam is replaced by an equivalent couple of forces, as was done in the analysis of vertical loads, the axial force acting on the connection panel zone is obtained. This is shown in Fig. 2-41 (d). The sum of forces acting on the upper end of the panel, Q_1, is given by

$$Q_1=\frac{1}{h_b}\ (M_A+M_B)+\frac{1}{2}\ (N_A-N_B)-Q_C \qquad (2\text{-}15)$$

The sum of forces acting on the lower end of the panel, Q_2, is likewise given by

$$Q_2=\frac{1}{h_b}\ (M_A+M_B)+\frac{1}{2}\ (N_B-N_A)-Q_D \qquad (2\text{-}16)$$

Considering the balance of the horizontal forces in Fig. 45 (c), we have

$$N_A+Q_D=N_B+Q_C \qquad (2\text{-}17)$$

2.3.3 Connections of Beams to Columns

In design drawings of a steel-frame structure, the connections of beams to columns are often not clearly visible. They are enclosed by the end widths of the intersecting columns and beams. These connections transmit sectional forces from the beams to the columns, and may also connect the columns of two floor levels.

When horizontal forces act on the structure, bending moment and shearing force from the beams on both sides of a connection are transmitted through the joints, which may place them in a highly stressed condition. Beam-to-column connections must have ample yield strength and rigidity, and the adjoining beams and columns must have ample plastic deformation capability, if the structure is to have adequate resistance to wind and earthquake forces.

The behavior of these connections under vertical and horizontal loads is explained below.

1) Connections under vertical load

Fig. 2-40 (a) is a bending moment diagram of a steel structure under vertical load, and (b) is an enlargement of the hatched portion of the diagram. M_0 denotes the bending moment at the beam ends, and h_b is the distance between the beam's bending moment centers. The axial forces in the beam flanges that act on the connection with the column can be calculated by M_0/h_b. As shown in Fig. 2-40 (c), these tensile and compressive forces on opposite sides of the column are symmetrical. Accordingly, if the yield strength of the column flanges is small, local deformation occurs in those flanges, as shown in (d), and the beams will deflect greatly. Sufficient care must be taken to avoid this. Horizontal stiffeners, as shown in (e), can be added to the connection. These stiffeners are also called diaphragms.

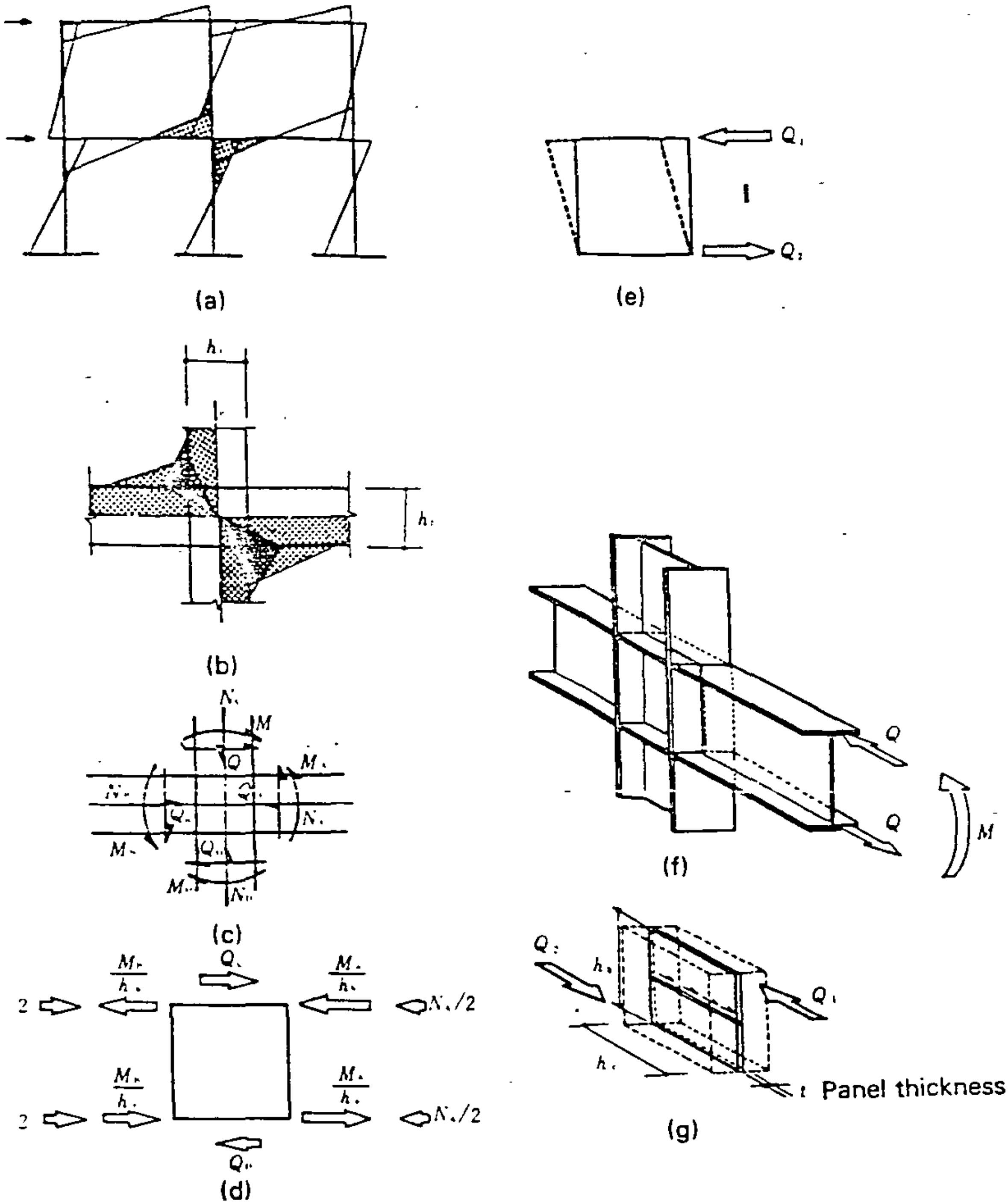


Fig. 2-41. Stresses in connection panel zone under horizontal load

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MODULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA:

MANUAL DE CONSTRUCCIÓN DE TANQUES DE 500,000 LBS

EXPOSITOR: ING. MARIO BARRETO MORALES
PALACIO DE MINERÍA
OCTUBRE DE 1998

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INDICE

INTRODUCCION

EN LA PREPARACIÓN DE ÉSTE MANUAL, SE CONJUGA LA PROPIA EXPERIENCIA DEL AUTOR CON LAS RECOMENDACIONES TÉCNICO-PRÁCTICAS DE TRES IMPORTANTES COMPAÑÍAS: CHICAGO BRIDGE AND IRON COMPANY (CBI), PITTSBURGH DES MOINES STEEL COMPANY (PDM) Y KAWASAKI HEAVY INDUSTRIES LTD (KHI), LAS DOS PRIMERAS NORTEAMERICANAS Y LA TERCERA DEL JAPÓN, QUIENES HAN FABRICADO Y MONTADO GRANDES TANQUES SOLDADOS DE TECHO FLOTANTE DE MÁS DE UN MILLÓN DE BARRILES (DE MÁS DE 159 MILLONES DE LITROS).

HAY UNA GRAN DIFERENCIA ENTRE UN TECHO CÓNICO (TC) Y UNO FLOTANTE (TF); MIENTRAS QUE EL PRIMERO ES FIJO, APOYADO EN UNA ESTRUCTURA METÁLICA O ESTÁ AUTOSOPORTADO, EL FLOTANTE ACTÚA COMO UN PISTÓN CON UNA HOLGURA ENTRE TECHO Y PARED DE LA ENVOLVENTE MUY REDUCIDA Y PARA QUE FUNCIONE CORRECTAMENTE, LA HORIZONTALIDAD, REDONDEZ Y VERTICALIDAD DE LA ENVOLVENTE, DEBERÁ QUEDAR SIEMPRE DENTRO DE LAS TOLERANCIAS QUE PERMITA EL CÓDIGO, EN ÉSTE CASO EL API STANDARD 650. ESTO SE LOGRARÁ, SIGUIENDO FIELMENTE LAS INSTRUCCIONES DE MONTAJE DE ESTE MANUAL EN LO QUE SE REFIERE A LA PROPIA ERECCIÓN, AL USO OBLIGATORIO Y CORRECTO DE LOS HERRAJES COMO CANDADOS, SEPARADORES, RIGIDIZANTES, PUNZONES, CUÑAS, ETC., Y A LOS PROCEDIMIENTOS ADECUADOS DE SOLDADURA. SOLO ASÍ SE OBTENDRÁN TANQUES BIEN CONSTRUIDOS DENTRO DE LAS NORMAS DE SEGURIDAD EXIGIDAS, SE ELIMINARÁN COSTOSAS REPARACIONES Y OPERARÁN A ENTERA SATISFACCIÓN DE LOS USUARIOS.

LA INFORMACIÓN CONTENIDA EN ESTE MANUAL, ES SUPLEMENTARIA A LA DEL API 650; SIRVE DE APOYO Y ESTÁ DE ACUERDO CON LOS REQUERIMIENTOS DE DISEÑO. ES APLICABLE A LA ERECCIÓN DE TANQUES (TF) DE CUALQUIER CAPACIDAD, PERO SE LE DÁ MÁS IMPORTANCIA A LOS DE 500,000 BLS. DE 85.34 M. DE DIÁMETRO Y 14.63 M. DE ALTURA DE ENVOLVENTE (280' X 48'), DESPLANTADOS SOBRE ANILLOS DE CONCRETO Y CON EL TECHO A BASE DE PONTONES , BOYAS Y SELLO FLEXIBLE (INGENIERÍA PDM).

1.0 GENERALIDADES, TRABAJOS PRELIMINARES.

EN ESTA PRIMERA SECCIÓN DEL MANUAL, SE INDICAN ALGUNAS REVISIONES Y RECOMENDACIONES PREVIAS, QUE SON NECESARIAS PARA UN BUEN INICIO DE LA CONSTRUCCIÓN DE UN TANQUE. POR EJEMPLO: -- UNA REVISIÓN MUY IMPORTANTE, ES LA DEL HERRAJE AUXILIAR DE - ARMADO. SI LA CÍA. CONTRATISTA NO EXHIBE UN LOTE COMPLETO DE TAL HERRAMIENTA, NO PODRÁ INICIAR EL MONTAJE. UNA RECOMENDACIÓN ESPECIAL, ES AQUELLA EN LA QUE EL SUPERVISOR DE PEMEX Y EL RESIDENTE DE LA CONSTRUCTORA, JUNTO CON SU MAESTRO MONTADOR, DEBERÁN REUNIRSE ANTES DE INICIAR LA CONSTRUCCIÓN, PARA ESTUDIAR Y ANALIZAR CONCIENZUDAMENTE EL MANUAL Y LOS PLANOS DE PROYECTO, DE FABRICACIÓN Y LOS DE MONTAJE, FORMULANDO UNA COMUNICACIÓN CON LAS DUDAS QUE PUDIERAN TENER, LA QUE SERÁ - ENVIADA POR CONDUCTO DE LA SUPERINTENDENCIA LOCAL DE CONSTRUCCIÓN, A LA COORDINACIÓN EJECUTIVA DE CONSTRUCCIÓN PARA LAS - ACLARACIONES CORRESPONDIENTES.

1.1. REVISIONES Y RECOMENDACIONES PREVIAS.

1. RECABAR EN LA OBRA PARA CONSULTA, UN JUEGO COMPLETO DE - PLANOS DE MONTAJE Y DE FABRICACIÓN DE LAS PARTES CONSTITUTIVAS DEL TANQUE: FONDO, ENVOLVENTE, TECHO, SELLO, PONTÓN Y ACCESORIOS COMO LA GUÍA ANTIRROTACIÓN, POSTES, ESCALERAS INTERIORES Y EXTERIORES, PUERTAS DE LIMPIEZA, REGISTROS DE HOMBRE, BOQUILLAS, DRENAJES, ETC. LAS COPIAS SERÁN REPRODUCIBLES Y SACADAS DE PLANOS TIPO ORIGINALES; NO SE ADMITIRÁ REPRODUCCIÓN DE COPIAS ILEGIBLES. LLEVARÁN LOS DATOS COMPLETOS DE LA OBRA QUE SE TRATA: UBICACIÓN, PROYECTO, REQUISICIÓN, PEDIDO, SELLOS DE REVISIONES CON FIRMAS AUTORIZADAS Y LA LEYENDA "APROBADO PARA - CONSTRUCCION"

2. REVISAR Y REPORTAR LA LLEGADA DE LOS MATERIALES Y ACCESORIOS, ANOTANDO EN BITÁCORA EL ESTADO DE LOS MISMOS, PRINCIPALMENTE EL MATERIAL DE PLACAS DE FONDO, ENVOLVENTE Y TECHO. PASAR EL REPORTE A LA SUPERINTENDENCIA LOCAL DE CONSTRUCCIÓN.

3. REVISAR EL MATERIAL DEL TANQUE QUE SE VA A MONTAR, DE ACUERDO CON LAS LISTAS DE EMBARQUES Y LISTAS DE MATERIAL ANOTADAS EN LOS PLANOS DE FABRICACIÓN Y MONTAJE, PARA ASEGURAR LA TOTALIDAD DE LAS PIEZAS EN EL CAMPO.

4. CONSERVACIÓN DE MATERIALES: SE PROCURARÁ ALMACENAR A LA INTEMPERIE, LAS PLACAS DE LA ENVOLVENTE, EN LA FORMA MÁS ADECUADA PARA EVITAR QUE PIERDAN SU CURVATURA; EN LA MISMA FORMA SE ALMACENARÁN LAS PLACAS PLANAS DEL FONDO Y TECHO PARA QUE NO SE DEFORMEN. ESTOS MATERIALES SE PROTEGERÁN DE LA INTEMPERIE, APLICANDO A TODA LA SUPERFICIE DE LA PLACA SUPERIOR DE CADA ESTIBA, DOS MANOS DE PINTURA ANTICORROSIVA, IGUALMENTE DEBERÁ HACERSE LA PROTECCIÓN DE LOS BORDES Y BISELES DE LAS PLACAS. LOS DEMÁS MATERIALES COMO EL ESTRUCTURAL, BOQUILLAS, TORNILLOS, HERRAJES, ETC., TAMBIÉN SE ALMACENARÁN CONVENIENTEMENTE PARA SU PROTECCIÓN Y CONTROL.

5. LA SOLDADURA SE ALMACENARÁ EN EL LUGAR ADECUADO PARA PRESERVARLA DE LA HUMEDAD. LA TEMPERATURA DE ALMACENAMIENTO SE FIJARÁ DE ACUERDO CON EL TIPO DE ELECTRODO Y DE LAS ESPECIFICACIONES CORRESPONDIENTES DE CÓDIGO Y/O LAS DEL FABRICANTE. ESPECIAL ATENCIÓN SE TENDRÁ CON LA SOLDADURA DE BAJO HIDRÓGENO.

1.2 EQUIPO Y HERRAMIENTA DE MONTAJE.

A CONTINUACIÓN SE RECOMIENDAN LAS CANTIDADES BÁSICAS DE EQUIPO Y HERRAMIENTA QUE SE REQUIERE PARA EL MONTAJE DE UN TANQUE DE 500,000 BLS. EN CASO QUE SE NECESITE MONTAR MÁS TANQUES - DE LA MISMA O DE MENOR CAPACIDAD, LAS CANTIDADES SE INCREMENTARÁN O DISMINUIRÁN PROPORCIONALMENTE:

A. 20 MÁQUINAS DE SOLDAR, ROTATORIAS O DE RECTIFICADOR PARA SOLDADURA MANUAL, CAPACIDAD 300 AMP.

B. 2 MÁQUINAS AUTOMÁTICAS DE ARCO SUMERGIDO PARA SOLDAR JUNTAS HORIZONTALES.

C. 2 PLANTAS GENERADORAS DE 400 K.V.A., CON MOTOR DE COMBUSTIÓN INTERNA.
20 CABLES DE TIERRA DE 20 METROS DE LONGITUD.
1,200 METROS DE CABLE DE COBRE FLEXIBLE, CALIBRE 2/0 PARA PORTA-ELECTRODOS.

D. 2 COMPRESORAS DE 300 PIES CÚBICOS POR MINUTO Y PRESIÓN - DE 7 KG/CM2, CON MOTOR DIESEL, PARA SUMINISTRAR AIRE PARA ARCO-AIRE, HERRAMIENTAS NEUMÁTICAS, PINTURA, ETC.
100 METROS DE MANGUERA FLEXIBLE PARA PRESIÓN DE 10 KG/CM2 Y DIÁMETRO DE 51 MM. (2").
800 METROS DE MANGUERA, IDEM PERO DE 10 MM. (3/8") DE -- DIÁMETRO.

E. 2 GRÚAS PARA ARMAR FONDOS Y ENVOLVENTES CON CAPACIDAD DE 20 TON., CON LLANTAS NEUMÁTICAS Y PLUMA DE 20 METROS.

F. UN MONTACARGA CON LLANTAS NEUMÁTICAS Y CAPACIDAD DE 8 - TON.

G. UNA CAMIONETA DE REDILAS CON CAPACIDAD DE 3 TON.

H. 5 TIRFORDS DE 2 TON. DE CAPACIDAD Y 25 METROS DE CABLE.

I. 5 EQUIPOS DE CORTE PARA OXI-ACETILENO, CON MANGUERAS DE 30 METROS.

J. 10 EQUIPOS DE ARCO-AIRE.
15 ESMERILES NEUMÁTICOS O ELÉCTRICOS.
10 CINCELES NEUMÁTICOS.

K. HERRAMIENTA DIVERSA PARA MONTAJE Y SOLDADURA: MARTILLOS DE BOLA, MARROS, MACETAS, BARRETAS (GRIFAS), LLAVES, -- DISCOS ABRASIVOS, MANGAS, CARETAS, LONAS, SOMBRILLAS, - GOGGLES, ETC.

L. TABLONES PARA ANDAMIOS EN NÚMERO SUFICIENTE DE 2" POR 10" POR 10'.
MÉNSULAS PARA ANDAMIOS EN NÚMERO SUFICIENTE.
ANDAMIOS TUBULARES DESMONTABLES O DEL TIPO GÓNDOLA CORREDIZOS.

1.2.1 HERRAJES PARA ARMAR Y AJUSTAR JUNTAS ENTRE PLACAS.

LA CÍA. CONTRATISTA DEBERÁ EXHIBIR EN LA OBRA, UN LOTE COMPLETO DE HERRAJES EN CANTIDADES SUFICIENTES, ANTES DE INICIAR CUALQUIER OPERACIÓN DE MONTAJE. EL SUPERVISOR DE PEMEX TENDRÁ LA OBLIGACIÓN DE REVISAR DICHO LOTE EN FORMA EXHAUSTIVA PARA

CHAZAR A SU JUICIO, TODAS AQUELLAS PIEZAS QUE SE ENCUENTREN - EN MAL ESTADO, YA SEA POR USO EXCESIVO, POR MAL TRATO O DIMENSIONES DIFERENTES A LAS ESPECIFICADAS EN LOS DIBUJOS CORRESPONDIENTES, INCLUIDOS EN ESTE MANUAL.

1.2.2 HERRAMIENTA ADICIONAL PARA MANIOBRAS Y MANEJO DE PLACAS.

LA SIGUIENTE LISTA CORRESPONDE A LA HERRAMIENTA QUE SE NECESITA PARA LAS MANIOBRAS DE DESCARGA, ALMACENAJE Y ACARREOS HASTA EL LUGAR DE LA ERECCIÓN DE LAS PLACAS, QUE POR SU TAMAÑO Y PESO REQUIEREN SER MANEJADAS CON EL EQUIPO DE GRÚAS, INDICADO EN EL PÁRRAFO 1.2.

A. 2 PIEZAS. BALANCÍN DE 7.00 M. DE CLARO, ENTRE APOYOS Y CAPACIDAD DE 8 TON. (PREFERIBLE TUBULAR).

B. 6 PIEZAS. PERROS PARA PLACA DE 38 MM. (1 1/2") DE ESPESOR MÁXIMO CON MORDAZAS ENDURECIDAS.

C. 6 PIEZAS. IDEM PERO PARA PLACA DE 19 MM. (3/4) DE ESPESOR MÁXIMO.

D. 4 PIEZAS. BARRAS REDONDAS DE ACERO LAMINADO Y PUNTA CÓNICA DE 38 MM. (1 1/2") DE DIÁMETRO Y 1.50 M. DE LONGITUD.

E. 4 PIEZAS. IDEM PERO DE 0.75 M. DE LARGO Y 19 MM. (3/4") - DE DIÁMETRO.

F. 4 PIEZAS. GRIFAS DE 38 MM. (1 1/2") DE DIÁMETRO Y 1.50 M. DE LARGO.

G. 20 PIEZAS. CINCELES DE ACERO LAMINADO DE 25 MM. (1") DE - DIÁMETRO Y 200 MM. (8") DE LARGO.

H. 20 PIEZAS. MARTILLO DE BOLA DE 900 GRAMOS (2LBS).

I. 10 PIEZAS. GRILLETES DE TORNILLO DE 19 MM. (3/4").

J. 6 PIEZAS. IDEM PERO DE 38 MM. (1 1/2").

K. 6 PIEZAS. ESTROBOS DE CABLE DE ACERO DE 19 MM. (3/4") - DE DIÁMETRO Y 4.00 M. DE LONGITUD.

L. 3 PIEZAS. IDEM PERO DE 25 MM. (1") DE DIÁMETRO Y 4.00 M. DE LONGITUD.

1.3 CIMENTACIÓN. REVISIONES Y TRAZOS.

LOS GRANDES TANQUES Y AQUELLOS CON PAREDES MUY ALTAS, TRASMITEN CARGAS CONSIDERABLES A LOS CIMIENTOS BAJO LA ENVOLVENTE. ESTO ES MUY IMPORTANTE EN TANQUES CON TECHO FLOTANTE, EN LO QUE SE REFIERE A ASENTAMIENTOS Y POR LO TANTO, A DEFORMACIONES DE LAS PLACAS DE LA ENVOLVENTE. EN ÉSTE CASO, O EN CUALQUIER OTRO, DONDE LA CAPACIDAD DE UN CIMIENTO PARA TRASMITIR LAS CARGAS ES DUDOSA, SE RECOMIENDA USAR UNA CIMENTACIÓN A - BASE DE ANILLOS BAJO LA ENVOLVENTE, QUE PUEDEN SER DE CONCRETO ARMADO O DE PIEDRA TRITURADA O GRAVA GRUESA. EN NUESTRO PAÍS, SE HA GENERALIZADO EL USO DE LOS ANILLOS DE CONCRETO PARA -- CUALQUIER CAPACIDAD DE TANQUES, LOS CUALES SON DISEÑADOS Y - CONSTRUIDOS DE ACUERDO CON LAS RECOMENDACIONES DEL API STD. 650. APÉNDICE B, SECCIÓN B-4, PÁRRAFO B.4.3 Y A LOS CÓDIGOS - ACI 318 Y ANSI A 89.1. SIN EMBARGO, EN ESTE MANUAL SE ESTÁN - CONSIDERANDO TAMBIÉN LOS ANILLOS DE PIEDRA O GRAVA, YA QUE ES POSIBLE USARLOS EN TANQUES DE MEDIANA Y BAJA CAPACIDAD CON TECHOS FLOTANTES O FIJOS Y EN TERRENOS RESISTENTES. VÉASE EL --

MISMO APÉNDICE B DEL API, PÁRRAFO B.4.4 Y FIGURA B-2 PARA EL DISEÑO Y CONSTRUCCIÓN DE ÉSTOS ANILLOS.

1.3.1 REVISIONES GENERALES.

LAS CIMENTACIONES CONSTRUIDAS SEAN DE CONCRETO O DE PIEDRA, ESTARÁN SUJETAS A LAS SIGUIENTES REVISIONES ANTES DE PROCEDER A LA ERECCIÓN DEL TANQUE, CONJUNTAMENTE POR EL RESIDENTE DE LA CONTRATISTA Y POR EL SUPERVISOR DE PEMEX:

1. EL RADIO MEDIO DEL ANILLO DEBERÁ SER EL CORRECTO, SEGÚN DISEÑO CON UNA TOLERANCIA DE ± 25 MM. (1").

2. LAS DIMENSIONES DEL ANILLO SERÁN REVISADAS, ASÍ COMO LA LOCALIZACIÓN DE REBAJES PARA LAS PUERTAS DE LIMPIEZA. (VÉASE SECCIÓN 3.7.7 FIGURA 3-9 DEL API 650).

3. SE EXAMINARÁN LAS DIMENSIONES, LOCALIZACIÓN Y ELEVACIÓN DE LAS TUBERÍAS SUBTERRÁNEAS Y LAS EXCAVACIONES.

4. LA PENDIENTE DE LA BASE (PENDIENTE DEL FONDO DEL TANQUE) Y LA ELEVACIÓN DE LA CORONA EN EL CENTRO DEL TANQUE, SERÁN REVISADAS Y PROBADAS, DE ACUERDO A LOS PLANOS DE CIMENTACIÓN.

5. LA BASE DEBERÁ SER COMPACTADA, UNIFORME Y CONFIGURADA APROPIADAMENTE. LA SUPERFICIE DEBERÁ ESTAR LIBRE DE PIEDRAS DE DIÁMETROS MAYORES DE 25 MM. (1"). VÉASE API STD 650, APÉNDICE B SECCIÓN 3.3.

6. SI LA BASE ESTÁ PETROLIZADA Y EL ACEITE CUBRE LA SUPERFICIE O ESTÁ SATURADA A TAL PUNTO QUE PUDIERA MANAR O FLUIR A TRAVÉS DE LAS JUNTAS DEL FONDO, DEBERÁ SER CORREGIDA POR EL CONTRATISTA DE LA CIMENTACIÓN, ANTES DE INICIAR EL MONTAJE DE LA ENVOLVENTE.

1.3.1.1. ASIENTO DE LA OBRA.

CUANDO LLEGUE EL CONTRATISTA Y SU RESIDENTE AL LUGAR DE LA OBRA, DEBERÁ REVISAR TANTO LA CIMENTACIÓN, ASÍ COMO SUS ÁREAS ADYACENTES. CONCEPTOS A REVISAR INCLUYEN: UN DRENAJE ADECUADO, ACCESOS Y ZONAS RIESGOSAS COMO TUBOS Y CABLES ELÉCTRICOS Y EN GENERAL UNA ADECUADA ÁREA DE TRABAJO.

1.3.1.2. INVESTIGACIÓN DEL SUBSUELO.

HÁGANSE SEIS (6) PERFORACIONES SIMÉTRICAS ALREDEDOR DEL PERÍMETRO DE LA ENVOLVENTE Y UNA (1) EN EL CENTRO DEL TANQUE, CON OBJETO DE INVESTIGAR LAS IRREGULARIDADES DEL SUBSUELO COMO: PIEDRAS AFLORANDO, CAVIDADES DE ARCILLA, VACÍOS, ETC. ESTO ES IMPORTANTE DEBIDO A QUE ÉSTAS IRREGULARIDADES PUEDEN LLEGAR A PRODUCIR ASENTAMIENTOS DESIGUALES.

SI EL ÁREA ALREDEDOR DEL TANQUE ES BLANDA Y LODOSA, ENTERRAR UNA VARILLA REDONDA DE 13 MM. (1/2") A UN LADO DEL CIMIENTO EN DISTINTOS LUGARES PARA ASEGURARSE QUE LA BASE NO ESTÁ DESPLANTADA SOBRE MATERIAL SUELTO (BASURA, ABONO, ETC.) LAS CONDICIONES DEL SITIO DE LA ERECCIÓN PUEDEN NO SER LAS QUE SE ESPECIFICAN EN EL DISEÑO, EN CUYO CASO EL RESIDENTE DE -

PEMEX Y EL DEL CONTRATISTA, NO DEBERÁN TITUBEAR EN NOTIFICAR A LA SUPERINTENDENCIA LOCAL DE CONSTRUCCIÓN, SI EL SITIO NO ES ACCESIBLE O SI NO ESTÁ DE ACUERDO CON LAS ESPECIFICACIONES.

1.3.2 <u>TRAZOS PRELIMINARES. CENTRO Y EJES DEL TANQUE.</u>

EN LA MAYORÍA DE LOS CONTRATOS ES DE LA RESPONSABILIDAD DE PEMEX, ESTABLECER PUNTOS DE REFERENCIA ADECUADOS QUE PERMITEN LA LOCALIZACIÓN EXACTA DEL CENTRO DE UN TANQUE Y DE LOS EJES DEL MISMO. SIN EMBARGO, EL CONTRATISTA DEL MONTAJE TIENE LA OBLIGACIÓN, A TRAVÉS DE SU PERSONAL, DE VERIFICAR CON EL SUPERVISOR DE PEMEX, LA EXACTITUD DE ESTOS PUNTOS.

1.3.2.1. <u>CENTRO.</u>

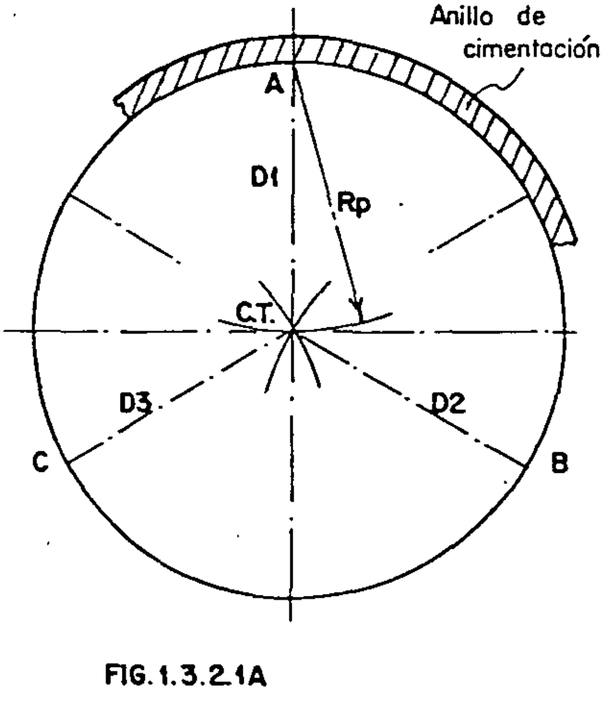
ES NECESARIO LOCALIZAR EL CENTRO DEL TANQUE EN LA BASE, ANTES QUE SEAN TENDIDAS LAS PLACAS DEL FONDO. ALGUNAS VECES SE CONSERVA EL CENTRO ORIGINAL QUE SIRVIÓ PARA LA CONSTRUCCIÓN DEL ANILLO DE CIMENTACIÓN (LOCALIZADO POR COORDENADAS EN LOS PLANOS GENERALES DE PROYECTO). SIN EMBARGO, EL RESIDENTE DE LA CONTRATISTA DEBERÁ ASEGURARSE QUE ESTE CENTRO ES CORRECTO Y NO SUPONERLO SIMPLEMENTE. SI NO HAY NINGUNA ESTACA O SEÑAL QUE MARQUE EL CENTRO, ESTE SE LOCALIZARÁ DE LA SIGUIENTE MANERA:

1. MÍDASE EL DIÁMETRO DE LA BASE EN TRES LUGARES APROXIMADAMENTE A 120° (DIÁMETRO INTERIOR DEL ANILLO DE CIMENTACIÓN)

2. CALCULAR UN DIÁMETRO PROMEDIO DE LAS MEDICIONES ANTERIORES Y DETERMINAR EL RADIO PROMEDIO.

3. SOSTENER UN EXTREMO DE LA CINTA METÁLICA EN UN PUNTO "A" DEL DIÁMETRO INTERIOR DEL ANILLO Y DESCRIBIR UN ARCO CON EL RADIO CALCULADO, CRUZANDO EL CENTRO DE LA BASE.

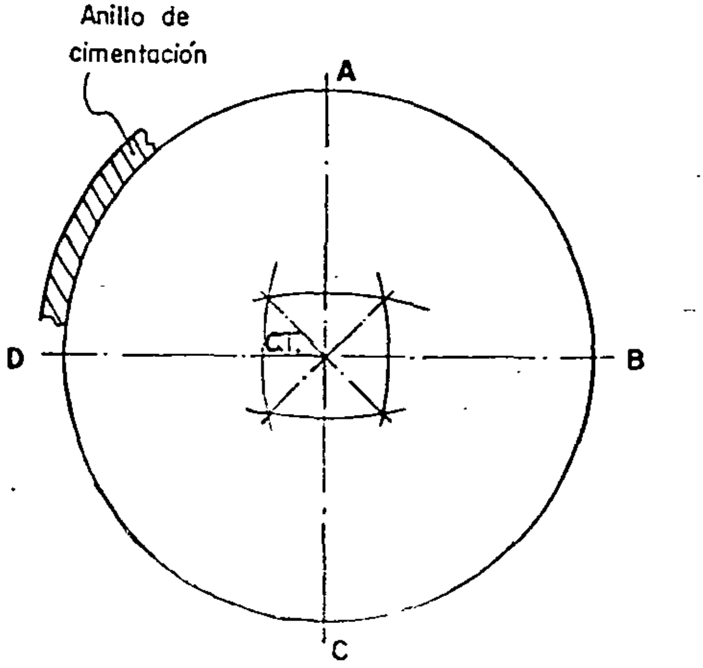
4. EN OTROS DOS PUNTOS B Y C DE LA PARED INTERIOR DEL ANILLO A 120° APROXIMADAMENTE DEL PRIMERO, REPETIR EL PASO 3 - - (FIGURA 1.3.2.1.A).



$$Dp = \frac{D1 + D2 + D3}{3}$$

$$Rp = \frac{Dp}{2}$$

FIG.1.3.2.1A



5. LA INTERSECCIÓN DE LOS TRES ARCOS DÁ EL CENTRO BUSCADO. MARCARLO CON UNA ESTACA.

6. OTRO PROCEDIMIENTO PARA LOCALIZAR EL CENTRO CUANDO NO -- EXISTE, ES EL SIGUIENTE: FIJAR CUATRO PUNTOS A, B, C Y D APROXIMADAMENTE A 90° DE SEPARACIÓN Y TRAZAR CUATRO ARCOS DESDE ESTOS PUNTOS, CON UN RADIO UN POCO MAYOR QUE EL -- REAL. EL CRUCE DE LAS DIAGONALES TRAZADAS EN LA INTERSECCIÓN DE LOS ARCOS, DÁ EL CENTRO DEL TANQUE. (FIGURA NO. 1.3.2.1B).

7. DESPUÉS QUE HA SIDO LOCALIZADO EL CENTRO, MÍDASE EL RADIO DEL TANQUE EN TODAS DIRECCIONES (DEBERÁ COINCIDIR CON EL EJE DEL ANILLO) PARA CONFIRMAR QUE LAS DIMENSIONES DE LA BASE SON LAS ADECUADAS PARA EL TANQUE QUE SE VA A MONTAR Y QUE EL CENTRO ESTÉ CORRECTAMENTE FIJADO.

1.3.2.2 <u>ORIENTACIÓN Y EJES DEL TANQUE</u>.

LA ORIENTACIÓN INDICADA EN LOS PLANOS, ESTÁ REFERENCIADA GENERALMENTE AL NORTE O EJE 0°. ESTE NORTE CONSTRUCTIVO O DE DIBUJO PUEDE NO COINCIDIR CON EL NORTE REAL, DE MODO QUE ES MUY IMPORTANTE VERIFICAR CON PERSONAL DE PEMEX QUE LA ORIENTACIÓN DE BOQUILLAS, PUERTAS, ETC., ESTÉN DE ACUERDO CON LO ESPECIFICADO EN EL DISEÑO DEL TANQUE.

PARA REFERENCIAS POSTERIORES, DEBERÁN TRAZARSE CON EXACTITUD LOS EJES N-S Y E-W (0°-180° Y 90°-270°) DE ACUERDO CON LAS INDICACIONES DE LOS PLANOS. MÁRQUESE EN LA CARA SUPERIOR DEL ANILLO DE CONCRETO Y PÁSENSE A SU CARA EXTERIOR, DE MODO QUE

NO SE BORREN. CON ESTOS TRAZOS SERÁ FÁCIL ENCONTRAR CON EXACTITUD EL CENTRO DEL TANQUE DESPUÉS QUE SE HA TENDIDO EL FONDO (FIGURA 1.3.2.2).

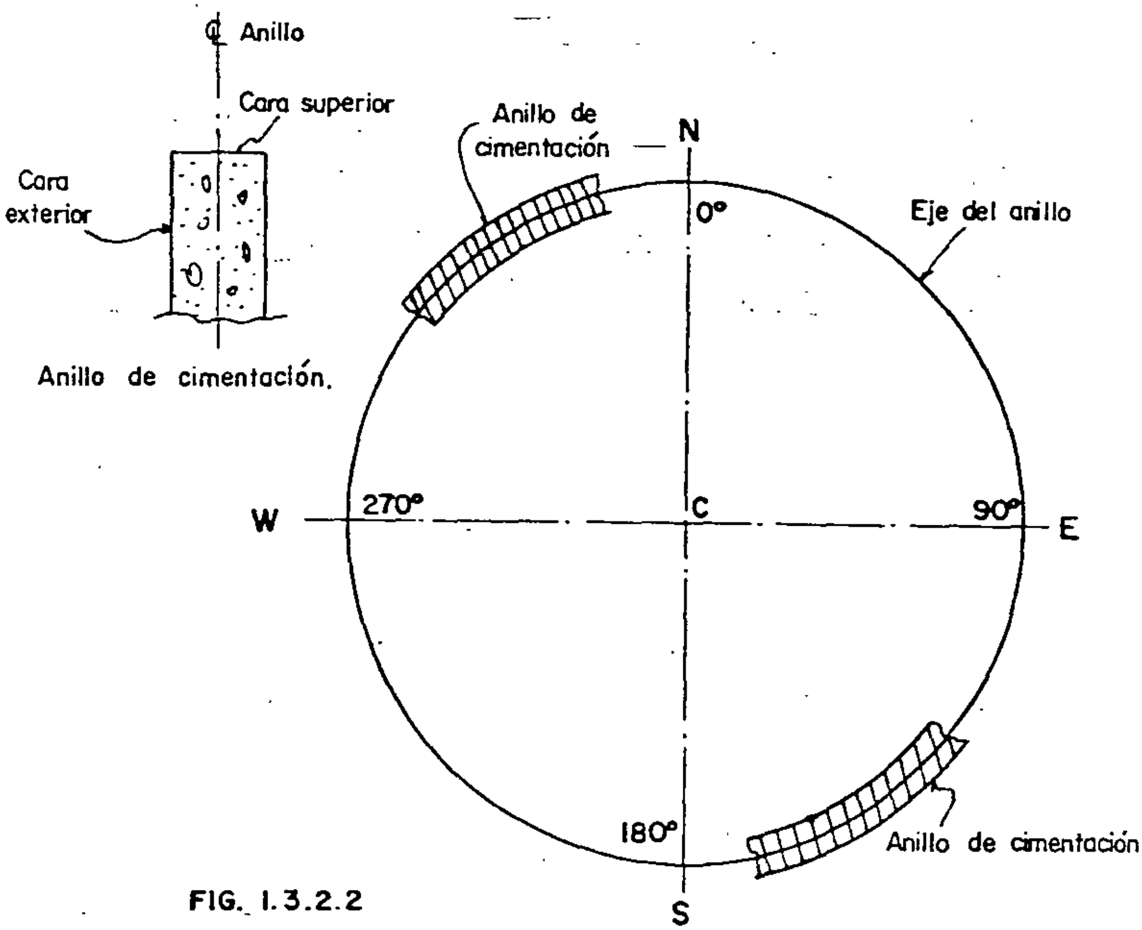


FIG. 1.3.2.2

1.3.2.3. <u>TRAZOS EN EL ANILLO DE CIMENTACIÓN.</u>

CON EL RADIO CORRESPONDIENTE AL MEDIO ESPESOR DE LAS PLACAS DEL PRIMER ANILLO DE LA ENVOLVENTE Y AUXILIADO CON LA CINTA METÁLICA, TRAZAR UN CÍRCULO SOBRE LA CARA SUPERIOR DEL ANILLO. DESDE EL PUNTO DE INICIO DEL MONTAJE DE LA ENVOLVENTE INDICADO EN LOS PLANOS, TRÁCENSE EN EL CÍRCULO LAS CUERDAS

DE CADA PLACA DEL PRIMER ANILLO CUYA LONGITUD DEBERÁ VENIR - CALCULADA EN LOS DIBUJOS DE LA ENVOLVENTE Y LOCALIZAR EN DICHAS CUERDAS LA MITAD DE CADA PLACA. PROYECTAR ESTOS PUNTOS RADIALMENTE EN EL ANILLO DE CONCRETO O DE PIEDRA, CALCULANDO QUE QUEDEN FUERA DE LAS PLACAS PERIFÉRICAS YA SEAN ANULARES O IRREGULARES. MARCARLOS CON PINTURA O EN OTRA FORMA DE TAL MANERA QUE NO SE BORREN (VER FIGURA 1.3.2.3A).

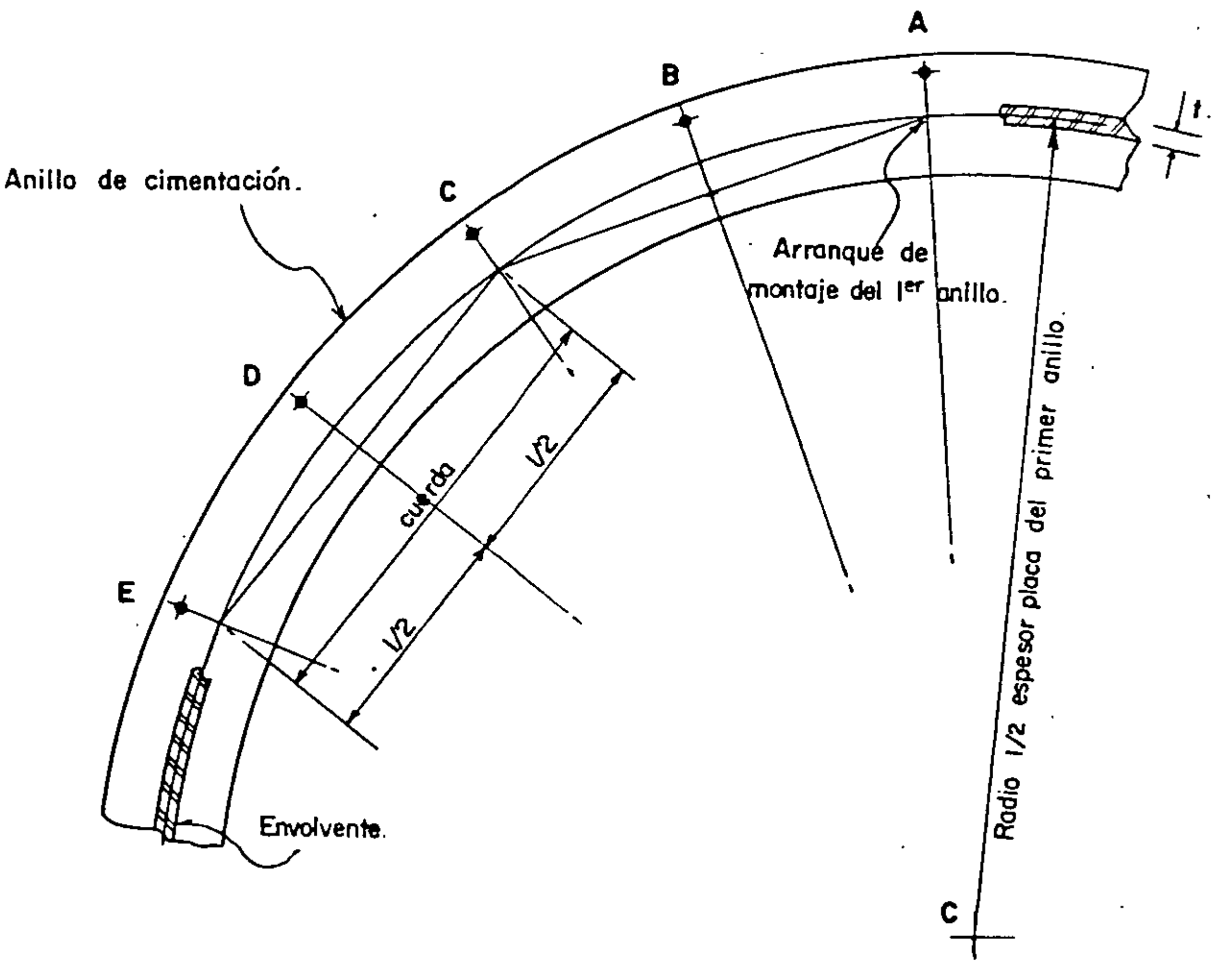


FIG. 1.3.2.3A

CON UN RADIO IGUAL AL RADIO AJUSTADO DE LA PERIFERIA DE LAS PLACAS ANULARES, AUMENTANDO 30 MM. (VÉASE PÁRRAFO 2.1.1 PARA ESTE RADIO AJUSTADO), TRÁCESE UN CÍRCULO DE REFERENCIA SOBRE EL ANILLO DE CONCRETO. SOBRE ESTE CÍRCULO MARCAR LA POSICIÓN CORRECTA DE LAS JUNTAS RADIALES ENTRE PLACAS ANULARES, CUIDANDO DE NO HACER COINCIDIR LOS TRAZOS DE LAS JUNTAS VERTICALES DE LA ENVOLVENTE MARCADAS SEGÚN EL PÁRRAFO ANTERIOR CON ÉSTOS ÚLTIMOS TRAZOS. LA DISTANCIA MÍNIMA ENTRE AMBAS JUNTAS ES DE 300 MM. (VÉASE LA FIGURA 1.3.2.3.B).

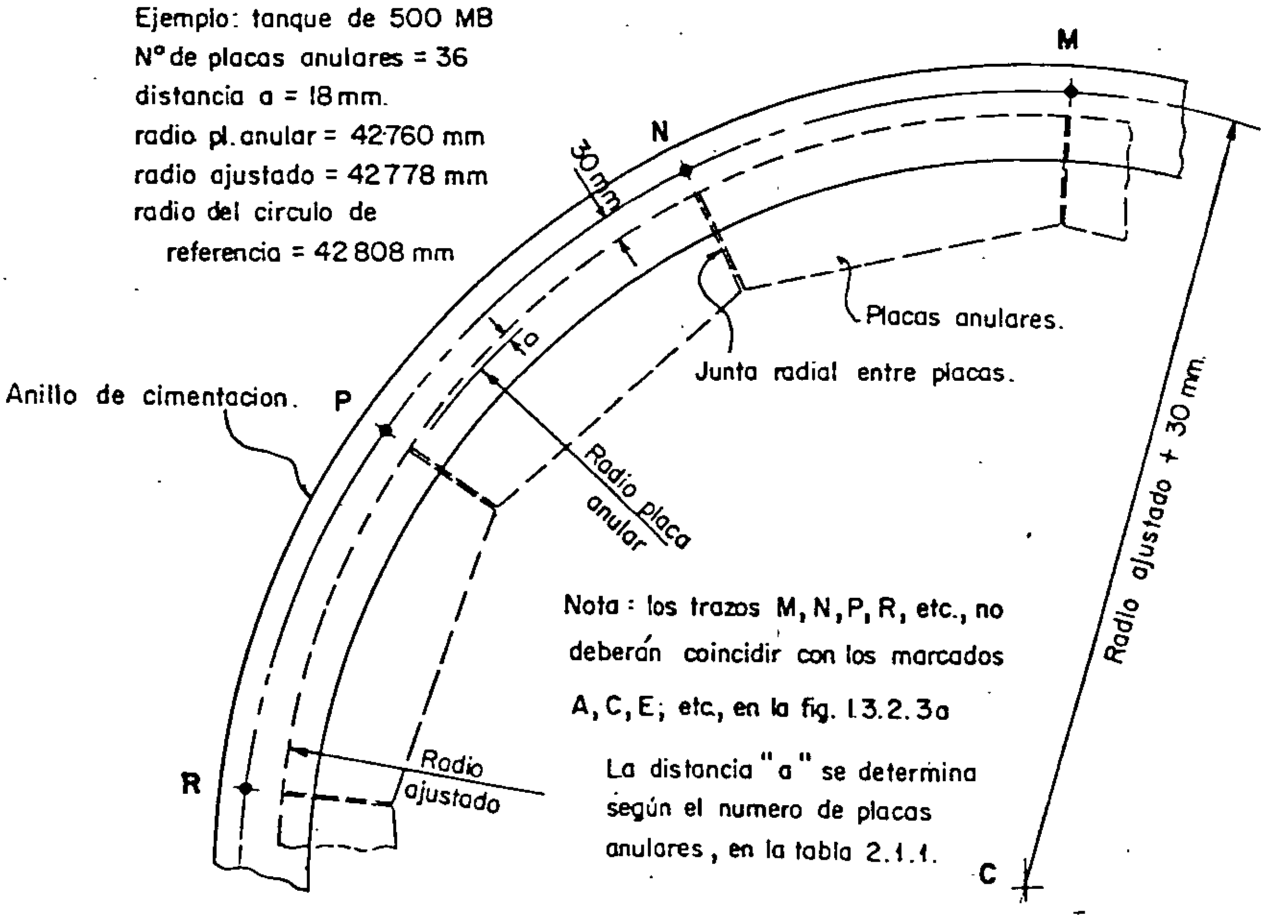


FIG. 1.3.2.3B

COMO EJEMPLO, APLIQUEMOS EL INSTRUCTIVO ANTERIOR A UN TAN-- QUE DE 500,000 BLS. DE CAPACIDAD IDÉNTICO A LAS FABRICADAS Y MONTADAS PARA PEMEX CON INGENIERÍA PDM (VÉASE LA FIGURA - 1.3.2.3.B PARA LOS CÁLCULOS).

1.4 NIVELACIÓN, VERTICALIDAD Y REDONDEZ. TOLERANCIAS.

PARA ASEGURAR EL MONTAJE CORRECTO DE UN TANQUE DE TECHO FLOTANTE Y QUE POSTERIORMENTE DEBA FUNCIONAR SIN PROBLEMAS, SE NECESITA REVISAR PRIMERO EN LA CIMENTACIÓN Y DESPUÉS EN LAS DISTINTAS ETAPAS DE LA ERECCIÓN DE LA ENVOLVENTE, QUE LOS - REQUERIMIENTOS DE NIVELACIÓN, VERTICALIDAD Y REDONDEZ DE ÉSTAS PARTES, SE ENCUENTREN DENTRO DE LAS TOLERANCIAS MARCA-- DAS EN EL CÓDIGO API. ES PUÉS, RESPONSABILIDAD DEL SUPERVISOR DE PEMEX Y DEL RESIDENTE DE LA CONTRATISTA, QUE ÉSTA -- DISPOSICIÓN SE LLEVE A CABO Y SE COMPLEMENTE CON LOS REGISTROS CORRESPONDIENTES PARA COMPARAR EL COMPORTAMIENTO DE LA UNIDAD ANTES Y DESPUÉS DE LAS PRUEBAS A QUE SE SOMETERÁ.

EN ESTA SECCIÓN DEL MANUAL SE FIJAN LAS TOLERANCIAS Y SE ESTABLECEN LOS PROCEDIMIENTOS PAPA MANTENER LA CIMENTACIÓN DE UN TANQUE A NIVEL, ASÍ COMO LA REDONDEZ DEL MISMO DURANTE - LA ERECCIÓN. EL ENRASE DE UN ANILLO DE CONCRETO O DE PIEDRA TRITURADA O GRAVA GRUESA FUERA DE NIVEL, PUEDE ORIGINAR:

1. DEFORMACIÓN (PANDEADURAS Y PARTES PLANAS) EN LA ENVOLVENTE.

2. TANQUES FUERA DE REDONDEZ.

3. TANQUES FUERA DE VERTICALIDAD

4. SEPARACIONES IRREGULARES EN LAS JUNTAS HORIZONTALES DE LOS ANILLOS DE LA ENVOLVENTE.

5. DIFICULTADES EN EL AJUSTE Y EN EL SOLDEO DE LAS PLACAS DE LA ENVOLVENTE.

LA EXPERIENCIA HA DEMOSTRADO QUE EL TIEMPO CONSUMIDO EN REVISAR Y CORREGIR EL ENRASE DE UNA CIMENTACIÓN FUERA DE NIVEL, PUEDE EVITAR SERIOS PROBLEMAS DURANTE LA CONSTRUCCIÓN, PRUEBAS Y OPERACIÓN DE UN TANQUE. COMPLEMENTANDO LO ANTERIOR SE RECOMIENDA TOMAR LECTURAS PERIÓDICAS DE NIVELACIÓN Y LLEVAR UN REGISTRO DE LOS ASENTAMIENTOS QUE HUBIERA, ASÍ COMO CUALQUIER OTRO PROBLEMA EN LA CIMENTACIÓN QUE PUDIERA AFECTAR LA OPERACIÓN DE UN TANQUE DURANTE SU VIDA ÚTIL.

1.4.1 TOLERANCIA DE NIVEL EN LOS ANILLOS DE CIMENTACIÓN.

CUANDO SE DISEÑAN ANILLOS DE CONCRETO PARA RECIBIR LA ENVOLVENTE, SE CONSIDERA A NIVEL LA CARA SUPERIOR O ENRASE DE DICHO ANILLO AÚN CUANDO HAYA UNA DIFERENCIA DE ± 3 MM. (± 1/8") EN UNA LONGITUD DE CIRCUNFERENCIA DE 9.00 M. (30') TOMADA ARBITRARIAMENTE EN CUALQUIER PARTE DE LA MISMA Y CON UN DESNIVEL DE ± 6 MM. (± 1/4") DESDE UN PUNTO TOMADO COMO REFERENCIA, EN TODA LA CIRCUNFERENCIA.

SI EL ANILLO ES DE PIEDRA O GRAVA, LAS TOLERANCIAS ADMISIBLES SON LAS SIGUIENTES: ± 3 MM. (± 1/8") EN 3.00 M. (10') DE LONGITUD DE CUALQUIER PARTE DEL ANILLO Y ± 13 MM. (1/2") EN TODA LA CIRCUNFERENCIA DESDE UN PUNTO DE REFERENCIA.

1.4.2 REVISIÓN DEL NIVEL DE LOS ANILLOS DE CIMENTACIÓN.

PARA AMBOS TIPOS DE ANILLOS Y AUXILIADO CON LOS TRAZOS DESCRITOS EN EL PÁRRAFO 1.3.2.3 PROCÉDASE A HACER LA REVISIÓN COMO SIGUE: COLOCAR UN NIVEL EN EL CENTRO DE LA BASE DEL TANQUE. COMENZANDO CON LA LOCALIZACIÓN DE LA PRIMERA JUNTA VERTICAL A PARTIR DEL NORTE CONVENCIONAL Ó 0°, TÓMENSE LECTURAS DEL NIVEL DEL ANILLO EN CADA MARCA (JUNTAS VERTICALES Y A MEDIA PLACA) MOVIÉNDOSE DE DERECHA A IZQUIERDA. CADA GRUPO DE LECTURAS DEBERÁ REGISTRARSE EN UNA FORMA ADECUADA COMO LA QUE SE ANEXA (CUADRO 1.4.2) Y SE ENVIARÁN A LA SUPERINTENDENCIA LOCAL DE CONSTRUCCIÓN, CONSERVANDO EL SUPERVISOR UNA COPIA. ESTAS OPERACIONES TOPOGRÁFICAS SE HARÁN ANTES QUE SE TIENDA EL FONDO Y SERVIRÁN PARA QUE REVISIONES POSTERIORES, DURANTE EL MONTAJE DE LA ENVOLVENTE Y DESPUÉS QUE EL TANQUE HA PASADO POR LAS PRUEBAS DE RIGOR, SE COMPAREN CON LA PRIMERA, A FIN DE DETECTAR POSIBLES ASENTAMIENTOS DE LA CIMENTACIÓN.

SI EN LA PRIMERA NIVELACIÓN RESULTA UN ANILLO DE CIMENTACIÓN FUERA DE LAS TOLERANCIAS ADMISIBLES, NOTIFÍQUESE INMEDIATAMENTE A LA SUPTCIA. LOCAL DE COSNTRUCCIÓN ANTES DE TENDER EL FONDO PARA RECIBIR INSTRUCCIONES DE COMO PROCEDER. EL CONTRATISTA DE LA CIMENTACIÓN DESARROLLARÁ SUS MAYORES ESFUERZOS PARA CONSTRUIR SUS ANILLOS DENTRO DE LAS TOLERANCIAS INDICADAS. SIN EMBARGO, EN MUCHOS CASOS NO SE LOGRARÁ ÉSTA EXACTITUD POR LO QUE LA SUPTCIA. LOCAL TIENE LA OPCIÓN DE RECOMENDAR EL USO DE CALZAS CON LÁMINAS DELGADAS DE ACERO, EN CUYO CASO EL RIMERO DE ÉSTAS DEBERÁ ESTAR A NIVEL CON UNA TOLERANCIA DE ± 1.5 MM. (± 1/16"). LA BASE DEBERÁ ESTAR A NIVEL CON EL ENRASE, DEL ANILLO O CON EL EMPAQUE DE LÁMINAS DE LAS CALZAS PARA QUE SE TENGA UN APOYO EFECTIVO DE LAS PLACAS DEL FONDO (FIGURA 1.4.2).

HOJA ____ DE ____

SUPERINTENDENCIA LOCAL DE CONSTRUCCIÓN ______________________

NIVELACION DE LA CIMENTACION

TANQUE ________ DE ________ BLS.

PUNTO	LECTURA	DIFERENCIA		DISTANCIA AL ANTERIOR	CUMPLE ESPECIF.	OBSERVACIONES
		PUNTO MÁS ALTO	PUNTO MÁS BAJO			
1						
2						
3						
4						
5						
6						
7						
.						
.						
.						
66						
67						
68						

NOTAS:

1.- LAS LECTURAS FUERON HECHAS SOBRE PUNTOS SITUADOS EN LA CIRUNFE-- RENCIA CORRESPONDIENTE A LA ENVOLVENTE DEL TANQUE, A CADA 5° QUE EQUIVALEN A ± ______ MM. ENTRE SÍ Y SON TAMBIÉN LOS PUNTOS DE -- LECTURA PARA MEDICIÓN DE REDONDEZ Y VERTICALIDAD DE LA ENVOLVENTE.

2.- CUANDO SE DETECTEN PUNTOS FUERA DE TOLERANCIA, SE HARÁN LAS CO-- RRECCIONES NECESARIAS ANTES DE SOMETER LA CIMENTACIÓN TERMINADA A REVISIÓN DE LAS OPERATIVAS PARA SU RECEPCIÓN.

3.- FIRMAS DEL RESIDENTE DE LA CONTRATISTA Y DEL SUPERVISOR DE PEMEX.

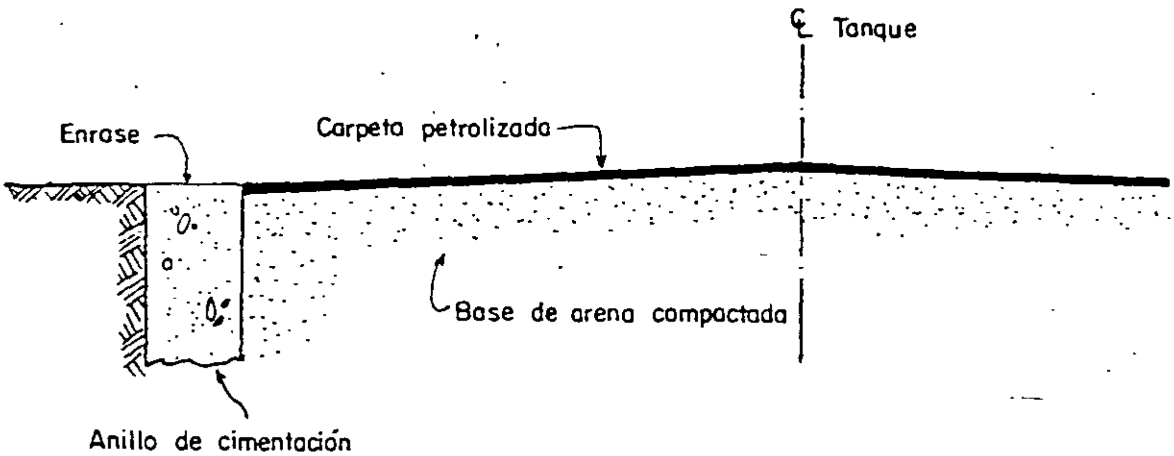


FIG. 1.4.2

1.4.3. <u>REQUERIMIENTOS DE HORIZONTALIDAD DE LA ENVOLVENTE</u>.

LA ORILLA SUPERIOR DE CADA ANILLO DE LA ENVOLVENTE <u>DEBERA</u> -- ESTAR A NIVEL CON UNA TOLERANCIA DE ± 3 MM. (± 1/8") EN UNA LONGITUD DE 9.00 M. (30') EN CUALQUIER PARTE DEL PERÍMETRO - DEL TANQUE Y UNA TOLERANCIA DE ± 6 MM. (± 1/4") EN LA CIRCUNFERENCIA TOTAL DESDE UN PUNTO DE REFERENCIA. ESTAS TOLERANCIAS SON APLICABLES A CUALQUIER TIPO DE CIMENTACIÓN ADOPTADO. SIN EMBARGO, UNA ENVOLVENTE DESPLANTADA SOBRE UN ANILLO DE - PIEDRA O GRAVA, CASI SIEMPRE TENDRÁ QUE SER RENIVELADA PARA ALCANZAR LOS CRITERIOS ACEPTADOS DE NIVEL.

1.4.4 <u>VERTICALIDAD</u>.

LA MÁXIMA DESVIACIÓN DE LA VERTICAL DESDE LA PARTE MÁS ALTA DE LA ENVOLVENTE A UN PUNTO SITUADO A 300 MM. ARRIBA DEL FONDO, NO DEBERÁ EXCEDER DE 1/200 DE LA ALTURA TOTAL H DE LA

ENVOLVENTE; LA DESVIACIÓN EN CADA ANILLO, SERÁ PROPORCIONAL A LA MÁXIMA. POR EJEMPLO: EN LOS TANQUES CON 6 ANILLOS DE -- 2438 MM. (8') DE ANCHO CADA UNO, LA ALTURA TOTAL H VALDRÁ -- 14,628 MM. (48'). LA DESVIACIÓN TOTAL SERÁ DE 76 MMM. (3") - EN NÚMEROS REDONDOS Y EN CADA ANILLO, LA TOLERANCIA SE INCREMENTARÁ 12.5 MM. (1/2") COMO MÁXIMO. (VÉASE LA FIGURA 1.4.4)

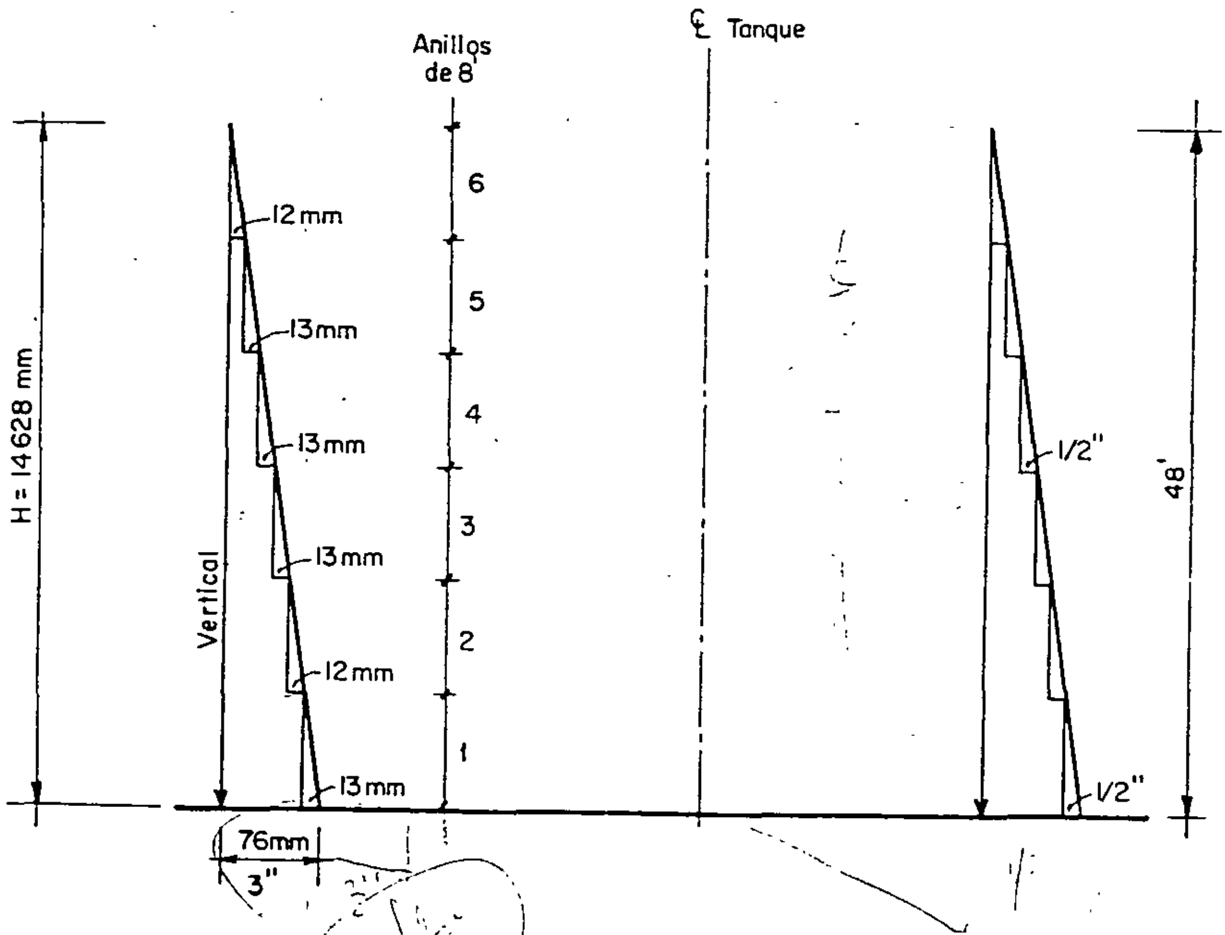


FIG. 1.4.4

LA DESVIACIÓN CON RESPECTO A LA VERTICAL EN CUALQUIER PLACA DE LA ENVOLVENTE NO EXCEDERÁ DE LOS VALORES ESPECIFICADOS DE LAS TOLERANCIAS DE LAMINACIÓN EN LAS TABLAS 14 Ó 15 DE LA ESPECIFICACIÓN A6 DE ASTM O DE LAS TABLAS 10 Ó 13 DE LA ESPECIFICACIÓN A20 TAMBIÉN DE ASTM, SIENDO APLICABLE CUALQUIERA DE ELLAS.

1.4.5 REDONDEZ

LOS RADIOS DE LA ENVOLVENTE MEDIDOS A 300 MM. (1') ARRIBA DEL FONDO, NO EXCEDERÁN DE LAS TOLERANCIAS INDICADAS EN LA TABLA 1.4.5. VÉASE LA SECCIÓN 5, PÁRRAFO 5.5.3 DEL API 650.

TABLA 1.4.5

DIÁMETRO DE TANQUE	TOLERANCIA EN EL RADIO
HASTA 12 METROS (40')	± 13 MM. (± 1/2")
DE 12 A 45 METROS (40' A 150')	± 19 MM. (± 3/4")
DE 45 A 76 METROS (150' A 250')	± 25 MM. (± 1")
MAYOR DE 76 METROS (MAYOR DE 250')	± 32 MM. (± 1 1/4")

1.4.6 "PEAKING" (DISTORSIÓN VERTICAL)

LA TOLERANCIA POR "PEAKING" EN LA ENVOLVENTE, SERÁ DE 13 MM. MEDIDA CON UNA CERCHA DE MADERA DE 900 MM. (36") DE LONGITUD, CURVADA AL RADIO EXTERIOR DEL TANQUE (SECCIÓN 5, PÁRRAFO 5.5.3 DEL API).

1.4.7 "BANDING" (DISTORSIÓN HORIZONTAL).

LA TOLERANCIA POR "BANDING" EN LA ENVOLVENTE SERÁ DE 13 MM. (1/2") MEDIDA CON UNA CERCHA DE MADERA RECTA DE 900 MM. (36") DE LONGITUD (SECCIÓN 5, PÁRRAFO 5.5.4 DEL API).

1.4.8 NIVELACIÓN DE LOS ANILLOS DE LA ENVOLVENTE.

LLEVAR REGISTROS ADECUADOS DE LAS LECTURAS DE NIVELACIÓN DE LA ENVOLVENTE, DESPUÉS QUE CADA UNO DE LOS PRIMEROS TRES -- ANILLOS HA SIDO MONTADO. SI HA OCURRIDO UN ASENTAMIENTO DIFERENCIAL MIENTRAS SE ESTÁ MONTANDO EL SEGUNDO Y EL TERCER ANILLO, CONTINUAR REVISANDOLOS HASTA QUE DOS ANILLOS CONSECUTIVOS NO REGISTREN HUNDIMIENTOS DIFERENCIALES. ASENTAR -- LECTURAS ANTES Y DESPUÉS DE CADA RE-NIVELACIÓN. TAMBIÉN REGISTRAR LOS DIÁMETROS DE TANQUES DE TECHO FLOTANTE EN TODOS LOS ANILLOS QUE REQUIERAN LECTURAS DE NIVEL. VÉASE LA TABLA 1.4.8 PARA LAS DIFERENCIAS ADMISIBLES EN DICHOS DIÁMETROS.

TABLA 1.4.8

DIÁMETRO DEL TANQUE M - (PIES)	DIFERENCIA ADMISIBLE DIAM. MAX. A DIAM. MINIM. MM. - PULG.
0-12 (0-40)	25 - (1)
12-45 (40-150)	38 - (1 1/2)
45-76 (150-250)	51 - (2)
MAYOR DE 76 (MAYOR DE 250)	64 - (2 1/2)

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ES MUY IMPORTANTE ESTABLECER EN EL PRIMER ANILLO DE LA ENVOLVENTE EXACTOS Y BIEN DEFINIDOS PUNTOS DE REFERENCIA EN CADA JUNTA VERTICAL Y A LA MITAD DE CADA PLACA DEL ANILLO. ESTO SE EJECUTA FACILMENTE USANDO PEDAZOS DE CINTAS MÉTRICAS (FLEXÓMETROS), ADHIRIENDOLOS Y LOCALIZANDOLOS EXACTAMENTE A UNA DISTANCIA CONVENIENTE DE LA ORILLA SUPERIOR DEL ANILLO (VÉASE FIGURA 1.4.8). COLOCAR LOS TRAMOS DE CINTA METÁLICA A -- UNA DISTANCIA DE 300 MM. DE CADA JUNTA VERTICAL Y OTRA A LA MITAD DE CADA PLACA, ASEGURANDOSE QUE ESTÉN ALINEADOS PER-- PENDICULARMENTE A SU ORILLA HORIZONTAL.

NOTA : Todas las cintas métricas adhesivas estarán a la misma distancia abajo de la orilla superior de las placas de la envolvente con una tolerancia de ± 1.5 mm (1/16")

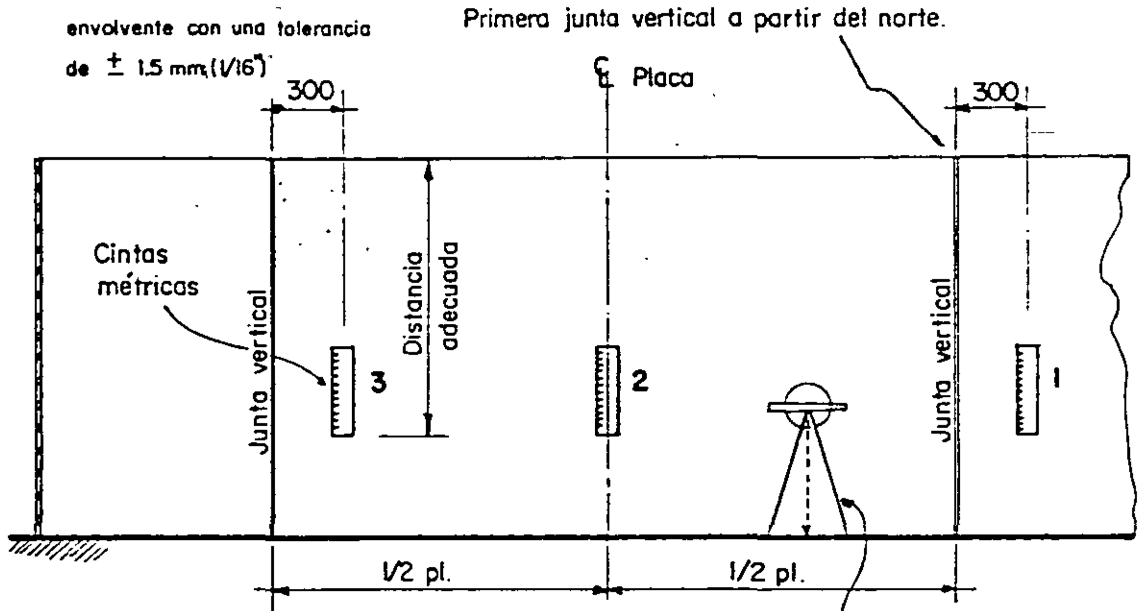


FIG. 1.4.8

DESPUÉS QUE LA JUNTA A ESCUADRA ENTRE LA PLACA ANULAR DEL - FONDO Y LA ENVOLVENTE HA SIDO UNIDA, REVISAR LA HORIZONTALIDAD DEL PRIMER ANILLO. SI SE REQUIERE RE-NIVELAR, ENGANCHAR EL EXTREMO SUPERIOR DE LA ENVOLVENTE CON EQUIPO DE LEVANTA-MIENTO APROPIADO Y ELEVAR LA ENVOLVENTE Y EL FONDO LO NECE-SARIO PARA INSERTAR Y AJUSTAR CALZAS DE LAINAS.

CUANDO SE REVISE EL NIVEL DE OTROS ANILLOS DE LA ENVOLVENTE, VERIFICAR EL DE LOS ANILLOS SUPERIORES ARRIBA DE LAS JUNTAS VERTICALES Y EN LA MITAD DE LA PLACA DEL ANILLO INFERIOR. SI ES NECESARIO RE-NIVELAR EL TANQUE, Y LA ENVOLVENTE ES TAN - ALTA QUE EL EQUIPO DE LEVANTAMIENTO NO PUEDE ALZARLA SE RE- QUERIRÁ USAR GATOS PARA REALIZAR ESTA OPERACIÓN.

1.5 EQUIPOS DE MEDICIÓN Y MEDICIONES.

1.5.1 CINTAS DE MEDIR.

USAR UNICAMENTE CINTAS DE ACERO PARA EFECTUAR MEDICIONES. ES- TÁ DEMOSTRADO QUE LAS CINTAS DE GÉNERO O DE FIBRA DE VIDRIO NO SON SEGURAS POR LA ÍNDOLE DE LAS MEDICIONES QUE SE REALI- ZAN DURANTE EL MONTAJE DE UN TANQUE. LAS CINTAS METÁLICAS - SE CALIBRAN COMUNMENTE A 4.5 KG. (10 LB.) DE TENSIÓN CUANDO ESTÁN APOYADAS EN TODA SU EXTENSIÓN. POR LO TANTO, CUANDO - SE EFECTÚAN MEDICIONES CON LA CINTA TENDIDA EN EL FONDO O - SUSPENDIDA VERTICALMENTE ADOSADA A LA PARED DE LA ENVOLVEN- TE, DEBERÁ SER ATIRANTADA CON LA TENSIÓN ANTES MENCIONADA. SIN EMBARGO, SI LA CINTA ESTÁ APOYADA SOLAMENTE POR SUS EX- TREMOS, DEBERÁ AUMENTARSE LA TENSIÓN PARA REDUCIR LA FLECHA QUE SE FORMA. A 15 METROS (50') EL TIRÓN REQUERIDO ES DE -- 6 KG. (13 LB.); A 30 M. (100') ES DE 14 KG. (30 LB.) Y A - - 46 M. (150') DE 25 KG. (55 LB.). SE REQUIEREN ESTOS VALORES PARA LOGRAR MEDICIONES EXACTAS. SI SE MIDEN RADIOS PARA RE- VISAR REDONDECES DE LOS ANILLOS DE LA ENVOLVENTE DE UN TAN- QUE, ES MUY IMPORTANTE QUE LA TENSIÓN REQUERIDA SEA LA MIS- MA, CADA VEZ QUE SE EFECTÚA UNA MEDICIÓN Y PARA LOGRAR ÉSTO ES NECESARIO USAR UN DINAMÓMETRO PARA GARANTIZAR LA IGUALDAD DE DICHAS TENSIONES Y A LAS ESPECIFICADAS DE ACUERDO CON LA DISTANCIA POR MEDIR.

1.5.2 MEDICIÓN DE LA REDONDEZ DE LA ENVOLVENTE.

LA MEDICIÓN DIRECTA DEL RADIO DE LOS ANILLOS DE LA ENVOLVENTE PARA FINES DE REVISIÓN DE LA REDONDEZ DE LA MISMA, DÁ RESULTADOS SATISFACTORIOS EN TANQUES HASTA ALREDEDOR DE 45.00 M. (150') DE DIÁMETRO. PARA RADIOS MAYORES HAY DOS PROCEDIMIENTOS DE MEDICIÓN PARA OBTENER RADIOS REALES.

1. EL PRIMER MÉTODO CONSISTE EN TRAZAR UN CÍRCULO DE REFERENCIA EN EL FONDO CON UN RADIO X Y USAR UNA PLOMADA CON ALAMBRE CUERDA DE PIANO CON UNA MEDIDA Y, FIJA EN EL EXTREMO SUPERIOR DEL ANILLO CORRESPONDIENTE; VER FIGURA (1.5.2A). MEDIR LA DISTANCIA Z EN TODA LA PERIFERIA, CADA 5° A PARTIR DEL ORIGEN Ó NORTE CONVENCIONAL Y SIGUIENDO UN MOVIMIENTO CONTRARIO AL DE LAS MANECILLAS DEL RELOJ. EL RADIO BUSCADO ES IGUAL A LA SUMA X+Y+Z. TEÓRICAMENTE, SI LA REDONDEZ ES PERFECTA, LA DISTANCIA Z SERÁ LA MISMA EN TODAS LAS MEDICIONES Y POR LO TANTO, LA SUMA X+Y+Z SERÁ EL RADIO DEL TANQUE INDICADO EN EL PLANO DEL FONDO. SIN EMBARGO, SI Z VARÍA DE UNA MEDICIÓN A LAS OTRAS, LOS RADIOS CALCULADOS TAMBIÉN VARÍAN Y EL TANQUE NO ESTÁ REDONDO. COMPARAR CON LAS TOLERANCIAS ADMISIBLES Y SI HAY DISCREPANCIA CORREGIR LA REDONDEZ.

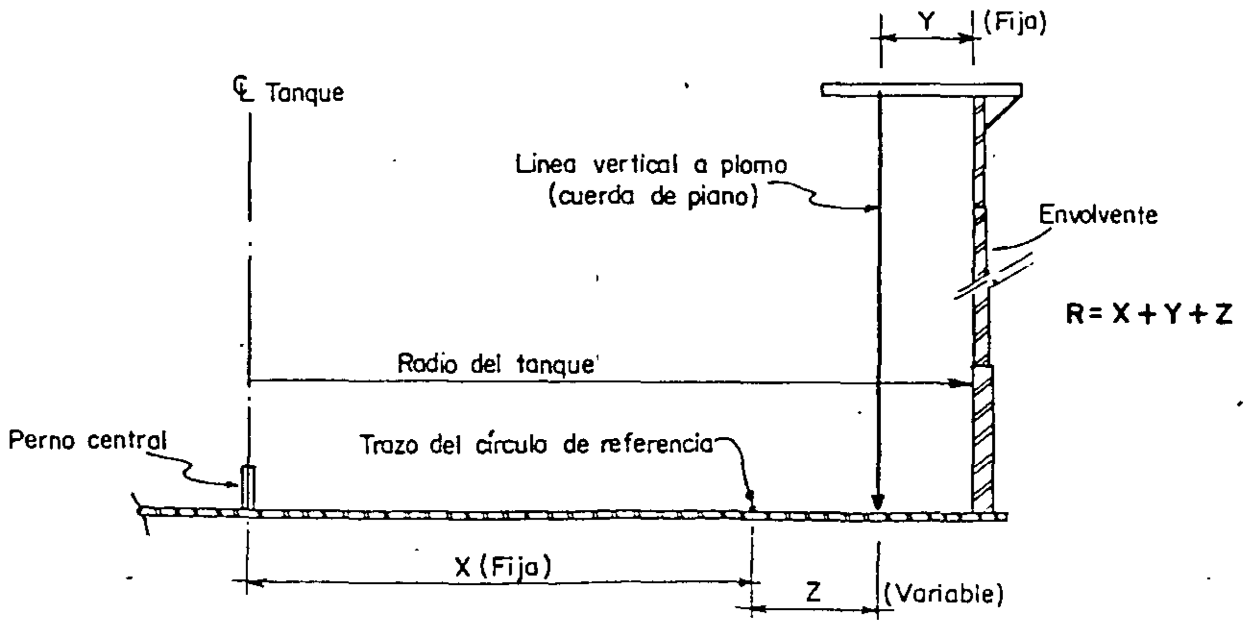


FIG. 1.5.2a

SUMERGIR LA PLOMADA EN UN RECIPIENTE CON AGUA O ACEITE, PARA IMPEDIR CUALQUIER VARIACIÓN DE LA VERTICAL. EN LA MEDICIÓN DE LOS RADIOS DE LOS ANILLOS SUPERIORES CASI SIEMPRE SUCEDE QUE YA SE ESTÁ TRABAJANDO EL DIAFRAGMA DEL TECHO SOBRE EL FONDO, EN CUYO CASO PROCEDER DE ACUERDO CON LA FIGURA 1.5.2B O SEA HÁGANSE LAS MEDICIONES POR EL EXTERIOR DEL TANQUE, PERO SIGUIENDO LAS INDICACIONES CORRESPONDIENTES A LA FIGURA 1.5.2A. AHORA EL RADIO DEL TANQUE SE CALCULA CON LA DIFERENCIA (X-Y)-Z.

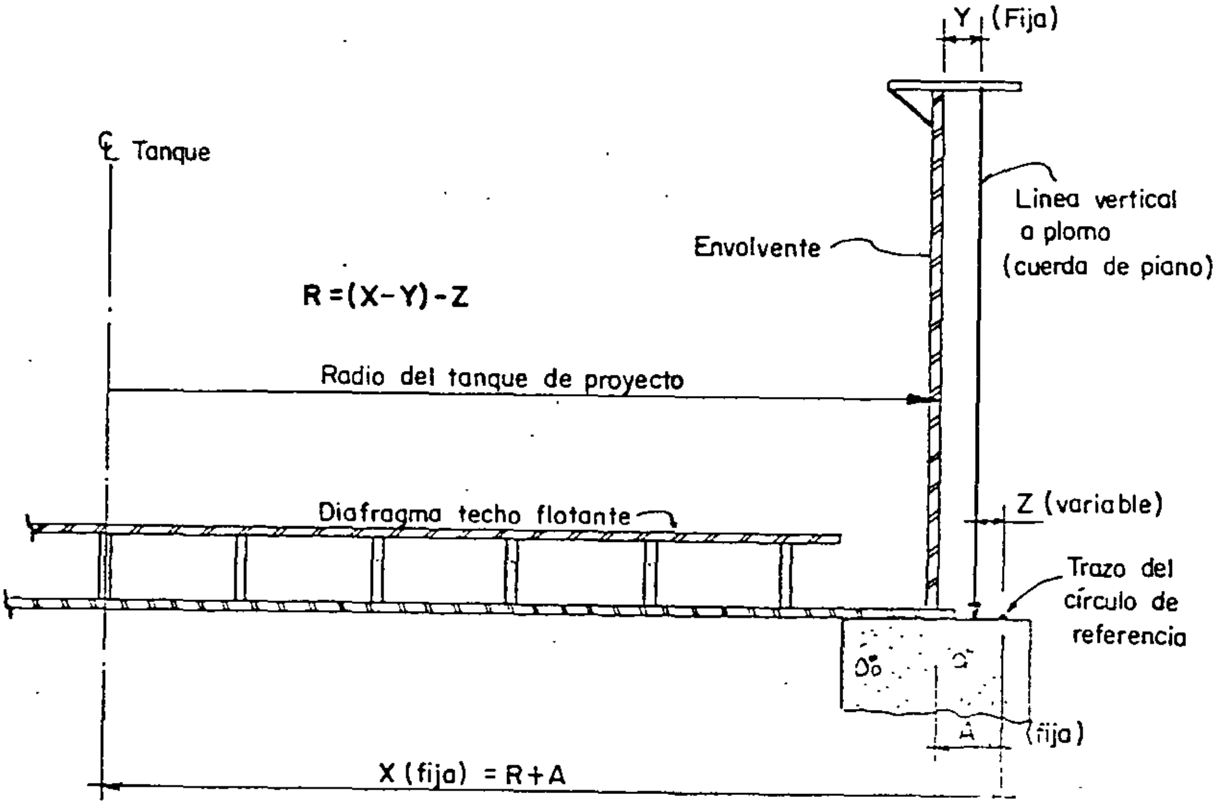


FIG. 1.5.2b

2. EL SEGUNDO PROCEDIMIENTO CONSISTE EN MEDIR CON LA CINTA DE ACERO, RADIOS INCLINADOS DESDE EL PERNO CENTRAL DEL TANQUE A LA ORILLA SUPERIOR DE CADA ANILLO (VÉASE FIGURA

1.5.2c) EN TODA LA PERIFERIA DE LA ENVOLVENTE, CADA 5° A PARTIR DEL ORIGEN Ó N CONVENCIONAL. LLEVAR UN REGISTRO DE MEDICIONES DE CADA RADIO INCLINADO Y COMPARARLAS. SI HAY DISCREPANCIA ENTRE DOS O MÁS MEDICIONES CONSECUTIVAS Y LA DIFERENCIA ENTRE LA MÁS LARGA Y LA MÁS CORTA NO ES ACEPTABLE, CALCULAR EL RADIO REAL INCLINADO Y CORREGIR LA ENVOLVENTE SI ES NECESARIO. CUANDO SE HAGA EL REGISTRO DE MEDICIONES, ANOTAR QUE SE TRATA DE RADIOS INCLINADOS. ESTE PROCEDIMIENTO REQUIERE QUE HAYA LA MISMA TENSIÓN EN LA CINTA EN CADA MEDICIÓN.

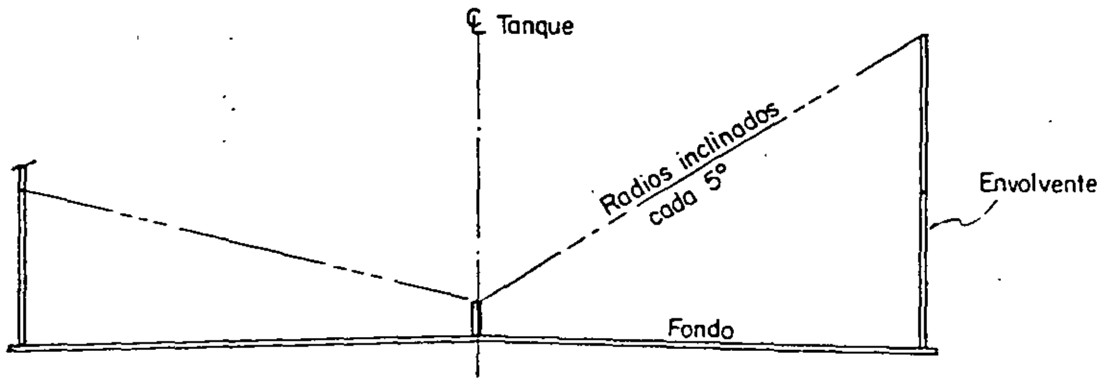


FIG. 1.5.2c

1.5.3. Equipo de Topografía

EL DEPARTAMENTO DE TOPOGRAFÍA DEBERÁ TENER LISTO Y ARREGLADO EL EQUIPO DE MEDICIÓN CON INSTRUMENTOS COMO NIVELES, TRANSITOS Y EQUIPO AUXILIAR, PARA USARSE EN CUALQUIER MOMENTO QUE SE REQUIERA EN LAS OPERACIONES DE NIVELADO, VERTICALIDAD, REDONDEZ, ETC. ESTOS INSTRUMENTOS DEBERÁN SER SIEMPRE

MANEJADOS POR PERSONAL CALIFICADO (TOPÓGRAFOS) Y LOS REGISTROS QUE LLEVEN DE SUS OPERACIONES DEBERÁN SER FIRMADOS POR ELLOS. SIN EMBARGO, EXISTEN ALGUNOS TRABAJOS PRÁCTICOS COMPLEMENTARIOS, AUXILIARES DE LAS OPERACIONES TOPOGRÁFICAS QUE DEBEN SER HECHOS POR EL PERSONAL DE MONTAJE Y DIRIGIDOS POR EL SUPERVISOR.

1.5.3.1 <u>TRÁNSITO Y NIVEL:</u> ESTOS INSTRUMENTOS DEBERÁN ESTAR APOYADOS EN UNA BASE SÓLIDA Y TAN CERCA DEL CENTRO DEL TANQUE COMO SEA POSIBLE. EXISTEN ALGUNOS MÉTODOS PARA INSTALARLOS EN UNA BASE SÓLIDA.

1. UNICAMENTE, <u>CON LA APROBACIÓN DE LA SUPTCIA. LOCAL DE CONSTRUCCIÓN</u>, SE PODRÁ APLICAR EL SIGUIENTE MÉTODO: CORTAR TRES (3) AGUJEROS DE 100 MM. (4") DE DIÁMETROS EN LA PLACA DEL CENTRO PARA DESCUBRIR LA BASE Y APOYAR EN ELLA EL TRÍPODE DEL INSTRUMENTO. DEBERÁ DISPONERSE DE PLACA DE LA MISMA ESPECIFICACIÓN PARA HACER TRES PARCHES CIRCULARES DE 150 MM. (6") DE DIÁMETRO PARA TAPAR LOS ORIFICIOS TRASLAPANDO Y SOLDANDO DESPUÉS DE TERMINAR LA OPERACIÓN TOPOGRÁFICA. HACER A ÉSTAS SOLDADURAS LAS MISMAS PRUEBAS QUE AL RESTO DEL FONDO.

2. PUNTEAR TRES (3) TUERCAS LISAS EN EL FONDO PARA SOSTENER EL TRÍPODE DEL INSTRUMENTO Y HACER UN ENTRAMADO TRIANGULAR DE MADERA (TABLONES USADOS EN LOS ANDAMIOS) ALREDEDOR DEL MISMO PARA AISLARLO (VER FIG. 1.5.3.1). COMO MOVIMIENTO EN EL ENTRAMADO PUEDE AÚN DESNIVELAR EL APARATO, HAY QUE ASEGURARSE DE REVISAR EL NIVEL DE LA BURBUJA ANTES DE CADA LECTURA.

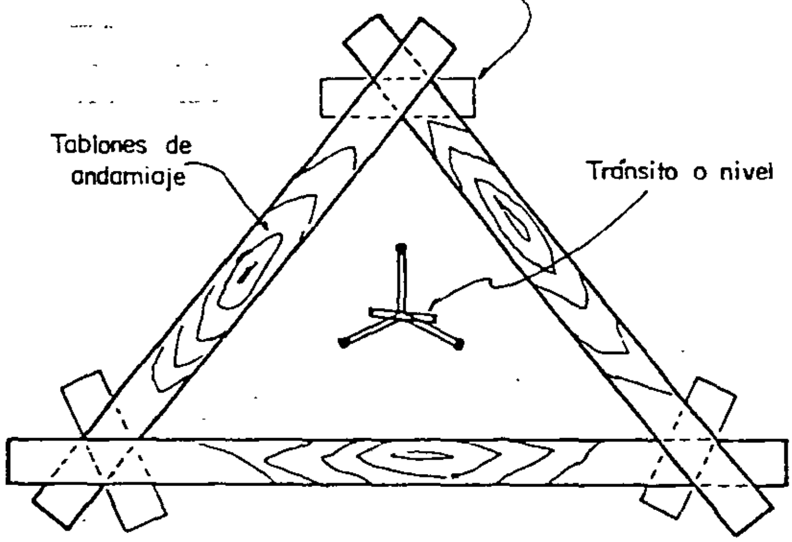


FIG. 1.5.3.1

3. TRABAJAR UNA POLIGONAL FUERA DEL TANQUE.

1.5.3.2 RECOMENDACIONES ADICIONALES: ES MUY IMPORTANTE QUE LA -- BURBUJA DEL TELESCOPIO ESTÉ PERFECTAMENTE CENTRADA EN CADA OBSERVACIÓN. SI LA BURBUJA ESTÁ DESCENTRADA UNA DIVISIÓN DEL TUBO, LA LECTURA ESTARÁ ALTA O BAJA 25 MM. A - - 45.00 METROS DE DISTANCIA EN LA MAYORÍA DE LOS INSTRUMENTOS, (VÉASE FIG. 1.5.3.2). POR LO TANTO, PARA OBTENER LECTURAS REALES CON UNA TOLERANCIA DE ± 3 MM., LA BURBUJA DEBE ESTAR DESCENTRADA CUANDO MÁS, LA OCTAVA PARTE DE UNA - DIVISIÓN.

LOS RAYOS DEL SOL DIRECTOS SOBRE UN INSTRUMENTO PUEDE - - DESCENTRAR LA BURBUJA. SERÁ NECESARIO INSTALAR UNA SOMBRILLA DE PROTECCIÓN, MIENTRAS SE TOMAN LECTURAS.

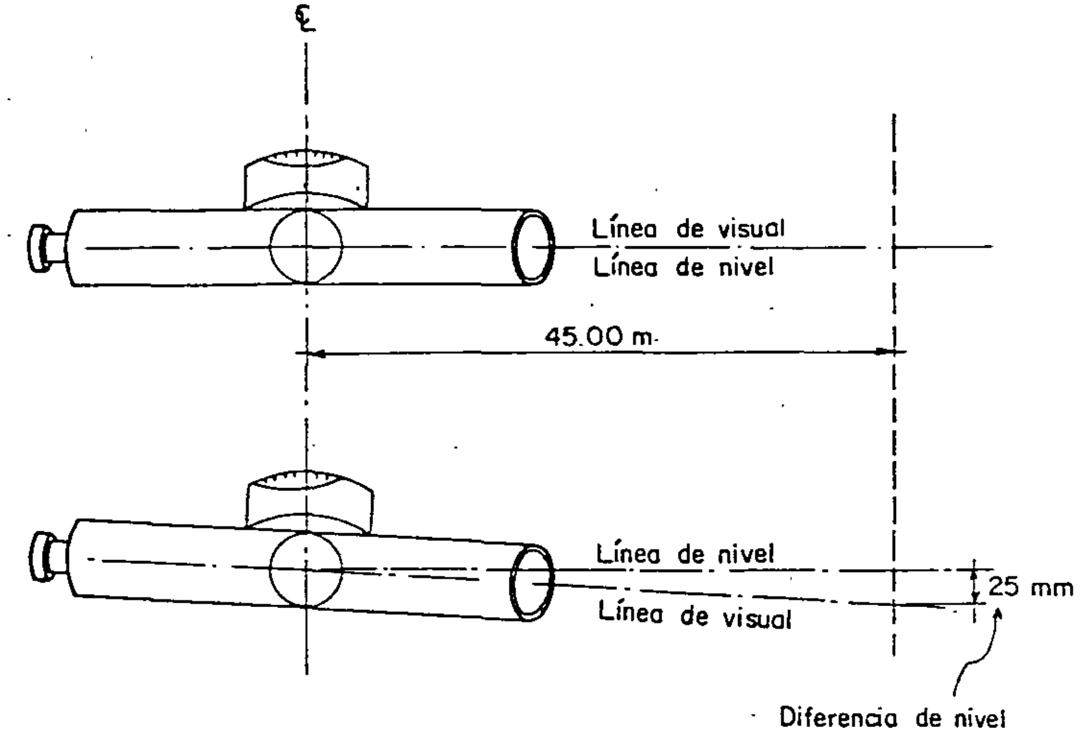


FIG. 1.5.3.2

EN TANQUES DE GRAN DIÁMETRO, DONDE ES NECESARIO HACER UN -- GRAN NÚMERO DE LECTURAS A GRAN DISTANCIA SE REQUIEREN INS-- TRUMENTOS MÁS SOFISTICADOS COMO NIVELES AUTONIVELABLES, INSTRUMENTOS A BASE DE RAYOS LASER, ETC.

DESPUÉS DE TOMAR UNA SERIE DE LECTURAS DE NIVEL, REVISAR -- SIEMPRE REGRESANDO AL PUNTO DE PARTIDA Y PARA ESTAR SEGUROS QUE EL INSTRUMENTO NO SE HA MOVIDO DEBEN COINCIDIR LAS LECTURAS. EN CASO CONTRARIO, REVISAR EL APOYO DEL TRÍPODE Y REPETIR LAS LECTURAS HASTA QUE COINCIDAN LA FINAL CON LA INICIAL.

2.0 FONDO. GENERALIDADES.

DISTRIBUIR Y TENDER LAS PLACAS DEL FONDO DEL TANQUE CON EL EQUIPO DISPONIBLE: GRÚA, PLUMA, MONTACARGA, ETC., LAS PLACAS SE TENDERÁN EN SU LUGAR SIGUIENDO LA SECUENCIA MARCADA EN EL PLANO DE MONTAJE, EMPEZANDO DEL CENTRO HACIA LA PERIFÉRIA, DEPENDIENDO DE LA DIRECCIÓN DEL TRASLAPE. PUNTEAR LAS PLACAS ENTRE SÍ NO MÁS DE LO REQUERIDO PARA SOSTENERLAS EN SU LUGAR.

EL CONTROL DE LA CONTRACCIÓN DE LAS PLACAS ES MUY IMPORTANTE EN LA SOLDADURA DEL FONDO. EN TANQUES MUY GRANDES, ES MÁS CRÍTICO DICHO CONTROL. UNICAMENTE UN ESTRICTO APEGO A LOS PROCEDIMIENTOS EXPUESTOS EN ÉSTE MANUAL, HARÁN MÍNIMOS LOS PROBLEMAS DE LA CONTRACCIÓN.

LA LIMPIEZA E INSPECCIÓN DE LA SOLDADURA DEL FONDO DEBERÁ HACERSE SIMULTANEAMENTE CON EL AVANCE DEL SOLDEO. PREFERIBLEMENTE, LO QUE SE SUELDA EN UN DÍA, DEBERÁ INSPECCIONARSE Y PROBARSE EL MISMO DÍA. MARCAR CON PINTURA EL AVANCE DE ÉSTAS OPERACIONES. VÉASE LA DESCRIPCIÓN DE LAS PRUEBAS DEL FONDO EN EL CAPÍTULO CORRESPONDIENTE.

2.1 MONTAJE DEL FONDO Y SECUENCIA DE LA SOLDADURA.

2.1.1 FONDOS CON PLACAS ANULARES, SOLDADAS A TOPE CON BISEL EN V Y LÁMINAS DE RESPALDO: MONTAR EL FONDO DE TANQUES DE ACUERDO CON LAS INSTRUCCIONES DADAS EN EL ORDEN INDICADO EN LOS SIGUIENTES PÁRRAFOS:

1. COLOCAR LA PLACA CORRESPONDIENTE AL CENTRO DEL TANQUE Y TRANSPORTAR A LA MISMA DICHO CENTRO, PREVIAMENTE LOCALIZADO (VEÁSE PÁRRAFO 1.3.2.1) HACIENDO COINCIDIR LA INTERSECCIÓN DE LOS EJES N-S Y E-W CON LA INTERSECCIÓN DE LAS DIAGONALES DE LA PLACA (FIG. 2.1.1A). SOLDAR EN EL NUEVO CENTRO, UN PERNO DE 13 MM. (1/2") DE DIÁMETRO Y 100 MM. (4") DE LONGITUD. CONSERVAR ESTA IMPORTANTE MARCA PUES ES UN AUXILIAR PARA TRAZOS Y MEDICIONES POSTERIORES.

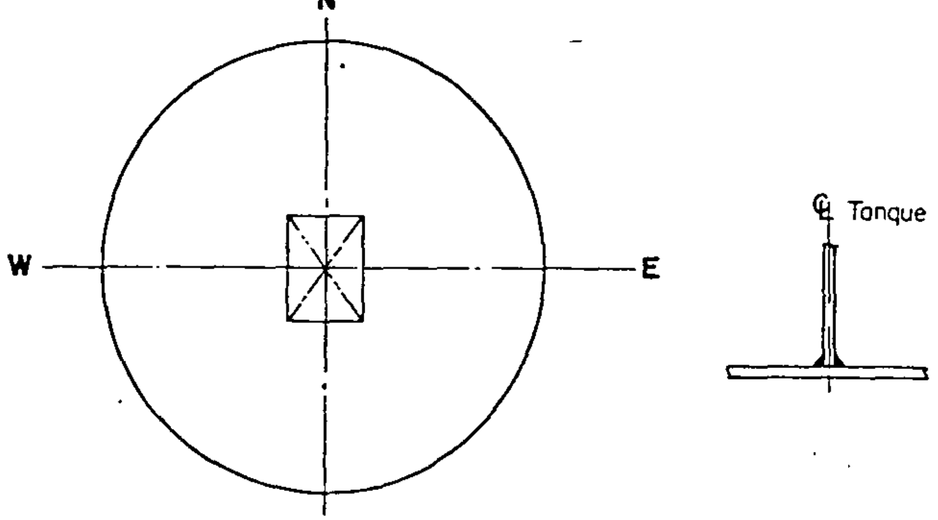


FIG. 2.1.1a

2. TENDER Y AJUSTAR LAS PLACAS ANULARES. A FIN DE OBTENER UNA SEPARACIÓN APROPIADA ENTRE PLACA Y PLACA, USAR UN RADIO AJUSTADO QUE NO ES OTRO, QUE EL INDICADO EN LOS PLANOS PARA LA PERIFÉRIA DE LAS PLACAS, AUMENTADO ALGUNOS MILÍMETROS, SEGÚN LA TABLA SIGUIENTE 2.1.1. TOMANDO COMO

TABLA 2.1.1

Número de placas anulares	13	19	25	32	38	44	50
Aumento al radio del plano en mm.	6	10	13	16	19	22	25

NOTA : Con menos de 13 placas , incrementar el radio 6 mm.

BASE EL CÍRCULO DE REFERENCIA TRAZADO SOBRE EL ANILLO - DE CIMENTACIÓN QUE SE DESCRIBE EN EL PÁRRAFO 1.3.2.3 Y CON EL AUXILIO DE UN ESCANTILLÓN TENDER LAS PLACAS EN - SU LUGAR. (VÉASE FIG. 2.1.1B).

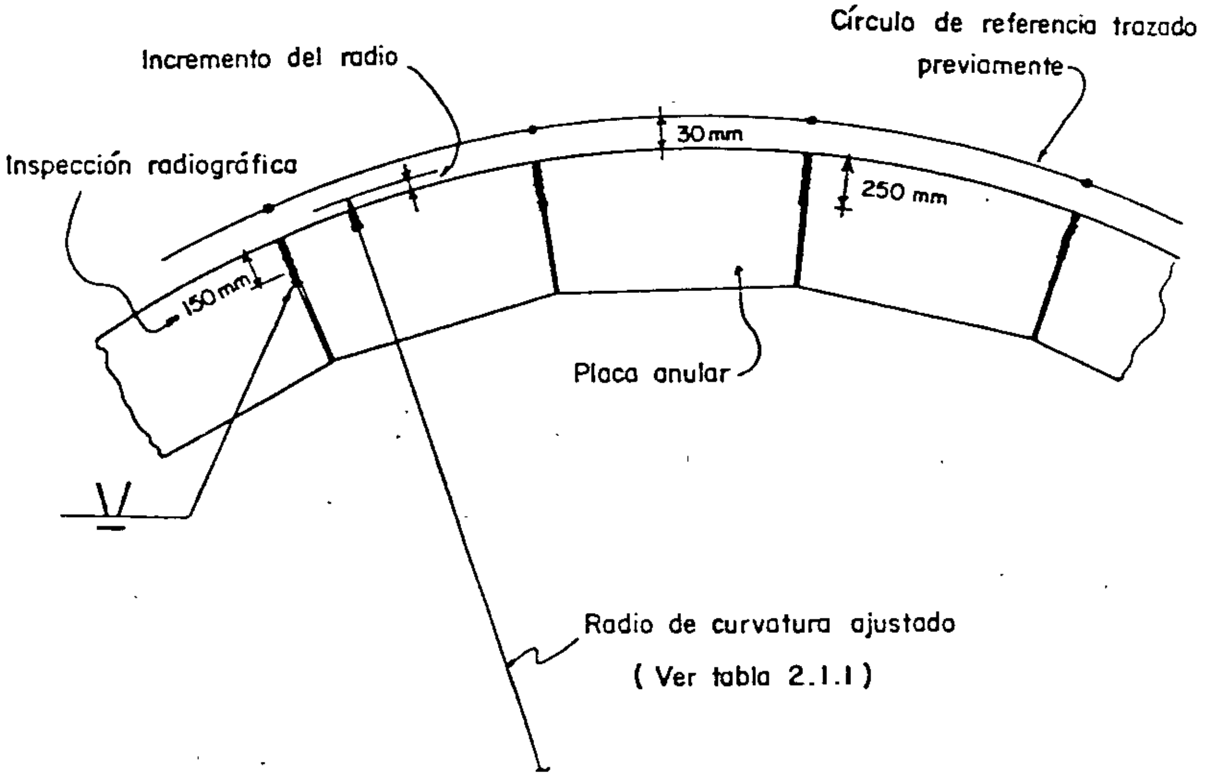


FIG. 2.1.1b

3. INICIAR EL MONTAJE DE LAS PLACAS RECTANGULARES TRASLAPADAS DEL FONDO, SIGUIENDO LAS INSTRUCCIONES DE LOS PÁRRAFOS 2.1.2 Y 2.1.3 Y DE ACUERDO CON SU COLOCACIÓN Y LA SECUENCIA MARCADA EN EL PLANO RESPECTIVO.

4. SOLDAR LOS 250 MM. (10") DEL EXTREMO EXTERIOR DE TODAS - LAS JUNTAS RADIALES DE LAS PLACAS ANULARES, ESMERILARLAS

E INSPECCIONAR LA SOLDADURA CON RADIOGRAFÍAS O PARTICULA MAGNÉTICA DE LOS 150 MM. (6") EXTREMOS, POR EL CONTRATISTA DE LA INSPECCIÓN RADIOGRÁFICA (VER FIG. 2.1.1B).

5. MONTAR EL PRIMER ANILLO DE LA ENVOLVENTE, DE ACUERDO CON LAS INSTRUCCIONES DE LA SECCIÓN 3.0 Y SOLDAR LAS JUNTAS VERTICALES.

6. FIJAR LA JUNTA ENTRE FONDO Y ENVOLVENTE, SEGÚN INSTRUCCIONES DEL PÁRRAFO 3.5 (FIG. 2.1.1C). EN LOS TANQUES DE GRAN CAPACIDAD, LAS PLACAS DEL PRIMER ANILLO SON TAN GRUESAS QUE LOS PUNZONES O CUÑAS DE AJUSTE ENTRE LA ENVOLVENTE Y LAS TUERCAS PUNTEADAS EN LAS PLACAS ANULARES,

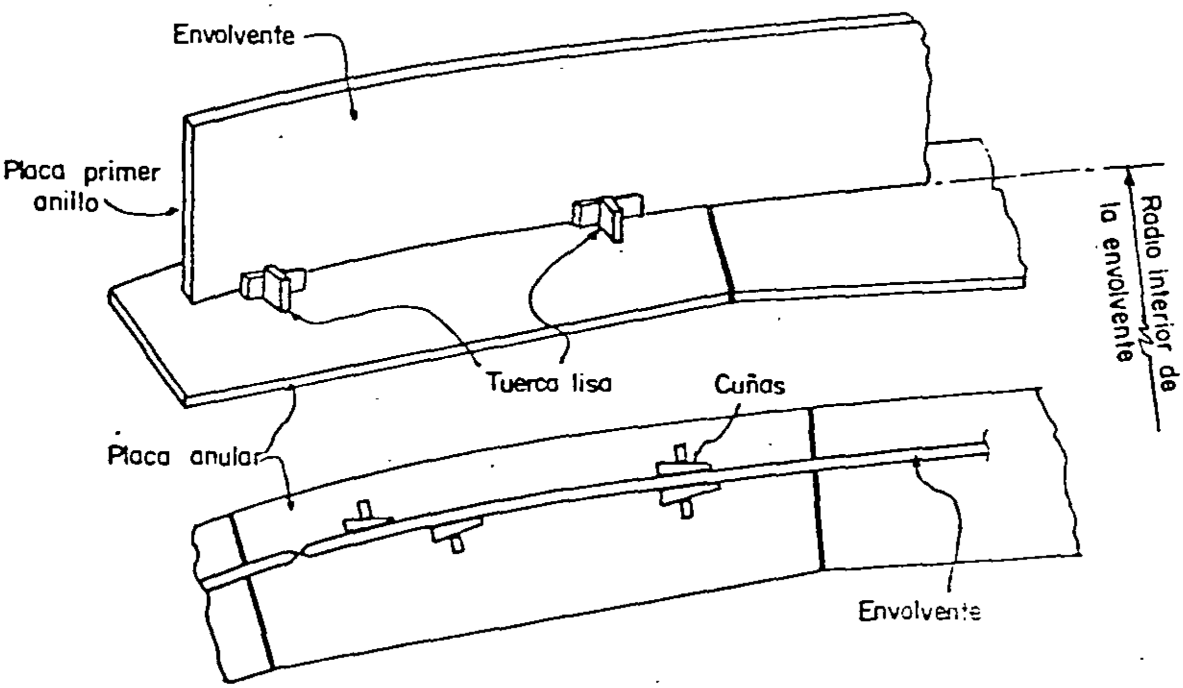


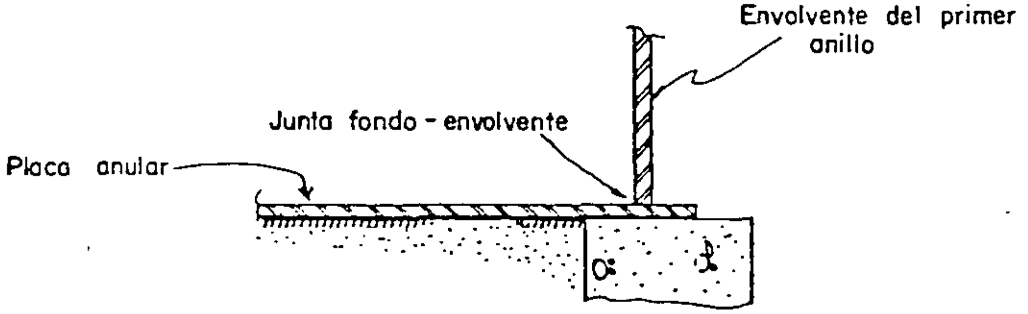
FIG. 2.1.1c

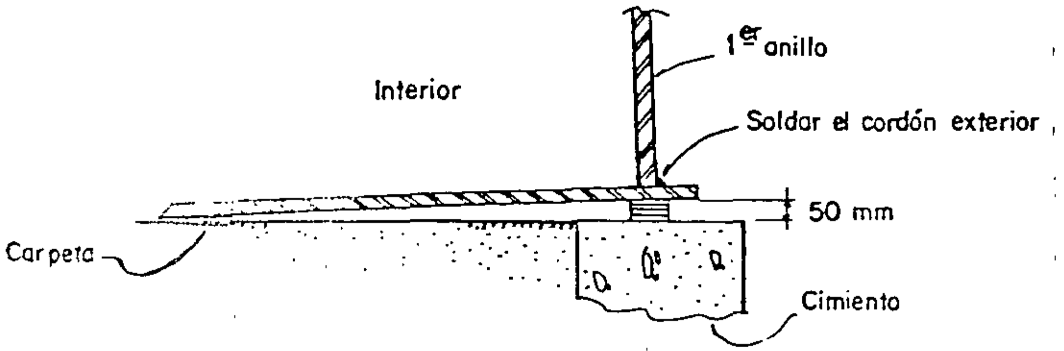
DEFORMAN ÉSTAS EN LUGAR DE REDONDEAR LA ENVOLVENTE. PUNTEAR LAS PLACAS IRREGULARES A LAS ANULARES PARA AGREGAR RESISTENCIA CUANDO SEA NECESARIO.

7. SOLDAR LA JUNTA CIRCULAR ENTRE LA PLACA ANULAR Y EL PRIMER ANILLO DE LA ENVOLVENTE CON LAS INDICACIONES CONTENIDAS EN EL PÁRRAFO 3.5. ESTA SOLDADURA ORIGINA QUE EL LADO INTERIOR (LADO RECTO) DE LAS PLACAS ANULARES TIENDA A LEVANTARSE DEBIDO A LA CONTRACCIÓN. UNO O TODOS DE LOS SIGUIENTES CINCO MÉTODOS DEBERÁN SER USADOS PARA CONTROLAR ESTA DEFORMACIÓN:

   A. ASEGURARSE SIEMPRE QUE EL TRAMO NO SOLDADO DE LAS JUNTAS RADIALES (FIG. 2.1.1b) SE PUEDA MOVER LIBREMENTE.

   B. ANTES DE SOLDAR LA JUNTA FONDO-ENVOLVENTE, COLOCAR TEMPORALMENTE UN EMPAQUE CON LAINAS DE MÁS O MENOS 50 MM. (2") DE ALTURA, ABAJO DEL FONDO Y DE LA ENVOLVENTE DE MODO QUE LA PLACA ANULAR SE INCLINE (FIGURA 2.1.1D) HACIA EL INTERIOR DEL TANQUE. QUITAR EL EMPAQUE DESPUÉS QUE SE HA SOLDADO LA JUNTA.





c. PUNTEAR CANALES (DE LAS USADAS COMO RIGIDIZANTES) ENTRE LA ENVOLVENTE Y LAS PLACAS ANULARES PARA QUE TRABAJEN COMO TORNAPUNTAS (FIG. 2.1.1E).

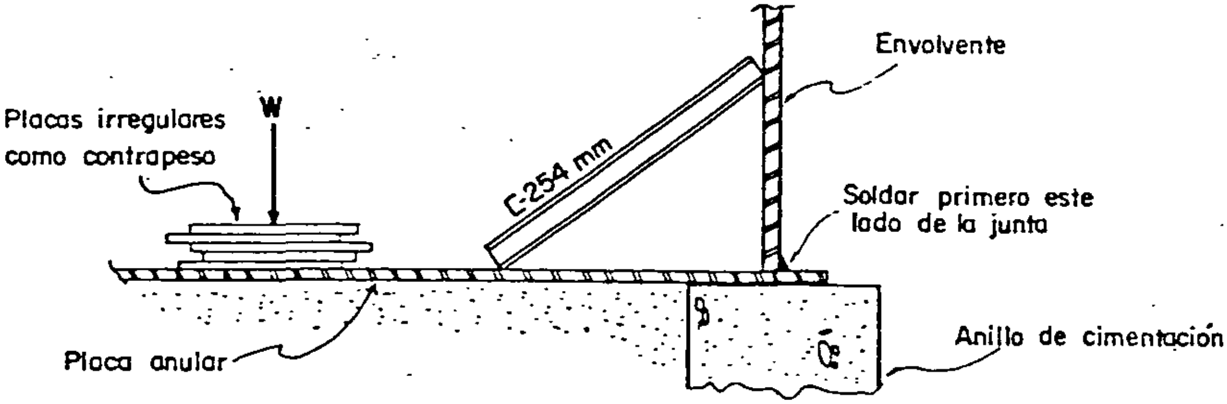


FIG. 2.1.1e

D. TENDER PLACAS IRREGULARES SOBRE LAS ANULARES PARA QUE SIRVAN COMO CONTRA-PESO Y AYUDEN EN ÉSTA FORMA A EVITAR QUE LAS PLACAS ANULARES SE LEVANTEN (FIG. 2.1.JE).

E. SOLDAR PRIMERO EL CORDÓN EXTERIOR DE LA JUNTA FONDO-ENVOLVENTE (SOLDADURA DE FILETE) PARA QUE LAS PLACAS ANULARES DEFORMADAS TIENDAN A VOLVER A SU POSICIÓN HORIZONTAL.

8. CON ARCO-AIRE CORTAR LAS JUNTAS RADIALES NO SOLDADAS ENTRE LAS PLACAS ANULARES ABRIENDOLAS A LA SEPARACIÓN APROPIADA. TERMINAR DE SOLDAR ÉSTAS JUNTAS SIN INTERRUPCIÓN Y BOTAR LA LÁMINA DE RESPALDO. EXAMINAR EL PRIMER PASO - (FONDEO) DE ACUERDO CON EL CONTRATISTA DE INSPECCIÓN RADIOGRÁFICA. SI LAS PLACAS IRREGULARES HAN SIDO TENDIDAS ANTES DE SOLDAR LAS PLACAS ANULARES, ASEGURARSE DE LEVANTARLAS POR LA ORILLA PARA SOLDAR COMPLETAMENTE LAS JUNTAS DE LAS PLACAS ANULARES.

9. FIJAR Y SOLDAR LAS PLACAS IRREGULARES A LAS ANULARES. CUANDO SE FIJE ÉSTA JUNTA, ASEGURARSE DE MANTENER LA MÍNIMA DISTANCIA ENTRE LA ENVOLVENTE Y LA PLACA IRREGULAR COMO SE INDICA EN EL PLANO DE MONTAJE. SOLDAR LAS INTERSECCIONES COMO JUNTAS DE TRES (3) PLACAS TAL COMO ESTÁ DETALLADO EN EL PÁRRAFO 2.2.2 JUNTA B.

10. FIJAR Y SOLDAR LAS PLACAS IRREGULARES UNAS A OTRAS. CUANDO HAYAN SIDO SOLDADAS TODAS LAS PLACAS RECTANGULARES, SOLDAR LAS IRREGULARES A AQUELLAS. EL USO DE CANDADOS ENTRE ÉSTAS COSTURAS AYUDARÁ A MANTENER LAS PLACAS ANULARES, PLANAS.

    NOTA: EN ALGUNOS PROYECTOS DE TANQUES SE DISEÑAN LAS PLACAS ANULARES CON JUNTAS A TOPE DOBLE V. EN ÉSTE CASO, LA SECUENCIA DE MONTAJE DE LAS PLACAS, ES SEMEJANTE A LA INDICADA EN EL PÁRRAFO 2.1.1 PERO EL PROCEDIMIENTO DE SOLDEO ES DIFERENTE. EN JUNTAS DOBLE V SE SUELDA PRIMERO EL CORDÓN INFERIOR, LUEGO CON ARCO-AIRE SE SANEA EL FONDEO POR EL LADO CONTRARIO Y SE SUELDA EL CORDÓN SUPERIOR EN POSICIÓN.

2.1.2 TENDIDO DEL FONDO CON PLACAS TRASLAPADAS RECTANGULARES.

EL ARREGLO DEL TENDIDO DE LAS PLACAS DEL FONDO, PUEDE ADOPTAR VARIAS FORMAS. A CONTINUACIÓN SE DESCRIBEN TRES DE LOS TIPOS MÁS COMUNES SELECCIONADOS POR LOS MÁS IMPORTANTES DISEÑADORES DE TANQUES:

1. FONDOS CON LAS PLACAS FORMANDO HILERAS LONGITUDINALES Y

FILAS TRANSVERSALES. LAS PLACAS PERIFÉRICAS DE CIERRE - SON ANULARES E IRREGULARES (FIGS. 2.1.3.1A Y 2.1.3.2A). ESTE ARREGLO ES PARA TANQUES DE GRAN CAPACIDAD (DE 100 A 500,000 BLS.)

2. FONDOS CON LAS PLACAS RECTANGULARES DISPUESTAS SOLAMENTE EN HILERAS LONGITUDINALES CON PLACAS ANULARES EN LA PERIFERIA Y PLACAS IRREGULARES TRANSVERSALES (FIG. 2.1.3.1B PARA TANQUES DE MEDIANA CAPACIDAD (55 A 100,000 BLS.).

3. FONDOS TIPO PLATAFORMA CON TODAS LAS PLACAS RECTANGULARES E IRREGULARES EN UN SOLO SENTIDO Y SIN PLACAS ANULARES. ESTE ARREGLO SE USA EN TANQUES DE MEDIANA A BAJA CAPACIDAD (55,000 A 500 BLS.) VÉASE FIG. 2.1.3.1C.

EN EL ARREGLO DE PLACAS DESCRITO EN EL PÁRRAFO 1 PARA USARSE EN LOS GRANDES TANQUES DE ALMACENAMIENTO CON TECHO FLOTANTE, LAS FILAS DE PLACAS TRANSVERSALES DEBERÁN TRASLAPARSE POR ENCIMA DE LAS HILERAS LONGITUDINALES ADYACENTES, A MENOS QUE - LO PROHIBA ALGUNA ESPECIFICACIÓN PARTICULAR DEL USUARIO. EN LO QUE SIGUE, SE DESARROLLA UN MÉTODO PARA TENDER LAS HILERAS Y LAS FILAS TRANSVERSALES PARA LOGRAR UN TRASLAPE APROPIADO. (VÉASE LA FIG. 2.1.2). EL EMPLEO DE ÉSTE MÉTODO ELIMINARÁ MUCHOS TRASLAPES DEL TIPO DE LA INTERSECCIÓN B (FIG. 2.2.2 PÁG. 21).

1. TENDER LA HILERA CENTRAL DE PLACAS RECTANGULARES A UNO Y OTRO LADOS DE LA PLACA CENTRAL PREVIAMENTE COLOCADA Y MEDIANTE UN HILO A REVENTÓN, MANTENER UN EXTREMO RECTO.

2. MARCAR EN EL OTRO EXTREMO DE LA HILERA, 50 MM. SI EL - -

TRASLAPE VA A SER DE 25 MM. (1" MÍNIMO) Ó 76 MM. SI EL PLANO DE MONTAJE MARCA UN TRASLAPE DE 38 MM. (1 1/2" MÁXIMO).

3. TENDER LA PRIMERA FILA DE PLACAS TRANSVERSALES DE MODO QUE SE TRASLAPEN LOS 50 O LOS 76 MM. MARCADOS EN LA PRIMERA HILERA, PUNTEANDOLA LIGERAMENTE A ÉSTA.

4. TENDER LA SIGUIENTE HILERA DE PLACAS A TOPE CON LAS DE LA FILA TRANSVERSAL.

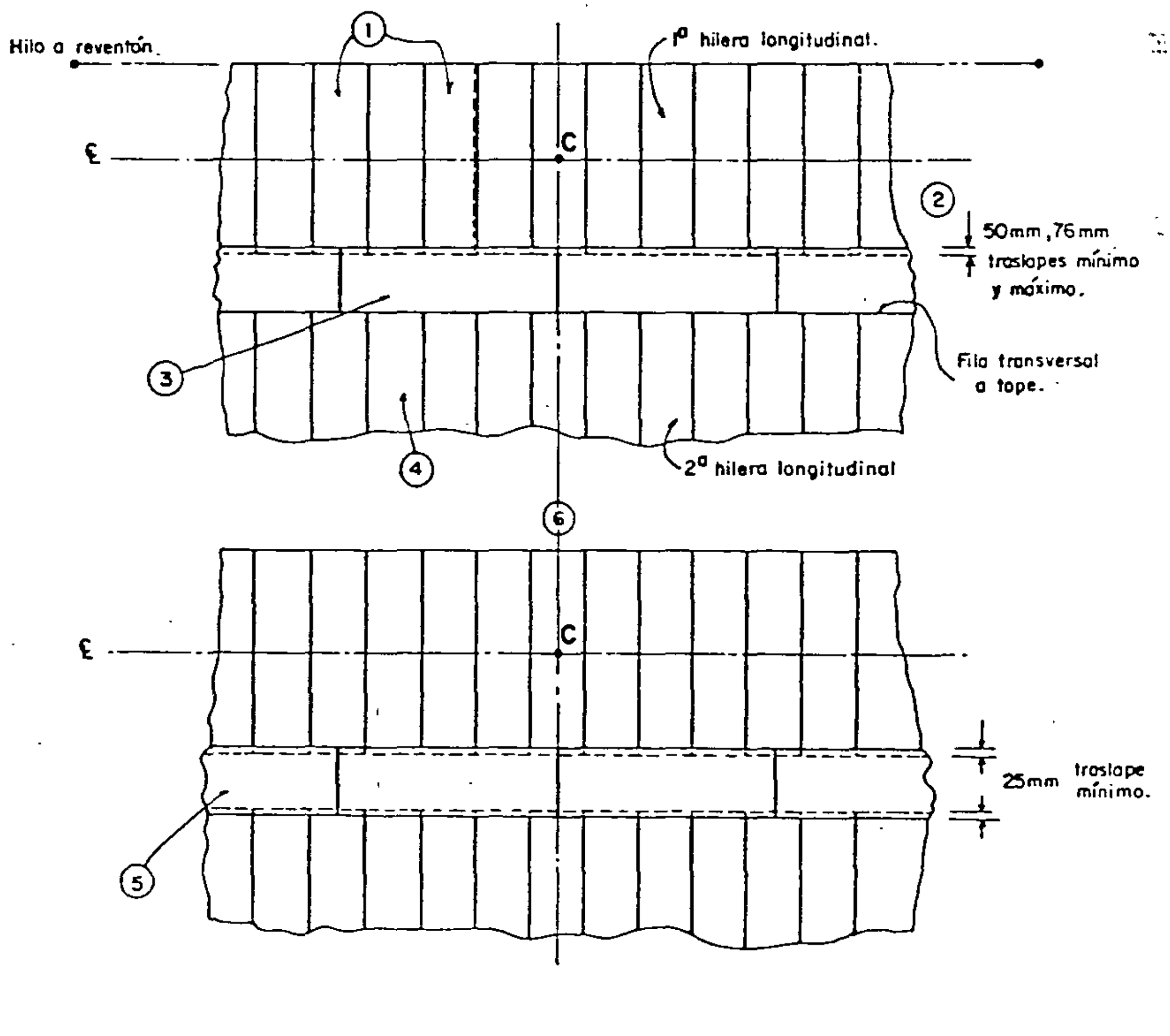


FIG. 2.1.2

5. DESTRUIR LOS PUNTOS DE SOLDADURA Y EMPUJAR CON BARRETAS LAS PLACAS DE LA FILA TRANSVERSAL POR ARRIBA DE LA SEGUNDA HILERA HASTA OBTENER UN TRASLAPE DE AMBAS HILERAS DE 25 A 38 MM. SEGÚN EL CASO.

6. REPETIR LAS OPERACIONES ANTERIORES EN EL LADO OPUESTO - SIMÉTRICO.

2.1.3 SECUENCIA DE SOLDEO EN FONDOS CON PLACAS TRASLAPADAS.

EN LA INTRODUCCIÓN A LA SECCIÓN 2.0 SE HACE NOTAR LA IMPORTANCIA QUE SE LE DEBE ASIGNAR AL CONTROL DE LA CONTRACCIÓN DE PLACAS DEBIDO A LA SOLDADURA, SOBRE TODO EN LOS GRANDES FONDOS DE 30, 60 Y 90 METROS DE DIÁMETRO. DE AQUÍ LA IMPORTANCIA QUE REVISTE EL ADOPTAR EN LOS PROCEDIMIENTOS DE LA - SOLDADURA DE FONDOS UNA SECUENCIA DETERMINADA PREVIAMENTE - CALIFICADA. A CONTINUACIÓN SE DESCRIBEN TÉCNICAS DE SOLDEO DE CBI, PDM Y DEL AUTOR PARA LOS TIPOS DE FONDOS MENCIONADOS EN EL PÁRRAFO 2.1.2.

2.1.3.1 TÉCNICA CBI: VEÁNSE LAS FIGURAS 2.1.3.1A, B Y C DONDE SE -- MUESTRA CON NÚMEROS ENCERRADOS EN CÍRCULOS, EL ORDEN DE LA SECUENCIA A SEGUIR. EL SOLDEO PUEDE INICIARSE TAN PRONTO COMO LAS PLACAS RECTANGULARES SON COLOCADAS Y FIJADAS EN SU - LUGAR CON UN MÍNIMO DE PUNTOS DE SOLDADURA.

1 SOLDAR LAS JUNTAS TRASLADADAS A LO LARGO DE LAS PLACAS RECTANGULARES DE CADA HILERA Y LAS DE LAS FILAS TRANSVERSALES A LO ANCHO CON COSTURAS EN UN SOLO SENTIDO Y -

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR : Ing. I. J. L. APROBADO POR : Ing. J. H B.	FECHA IV-86	HOJA 11 DE 24
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

NOTAS.
a.- Fíjense las placas con la menor cantidad de puntos de soldadura.
b.- Usese la técnica de soldeo en retroceso en todas las costuras.
c.- Las costuras 8 pueden hacerse en cualquier momento pero siempre antes que las 8a y 8b sean punteadas o soldadas.

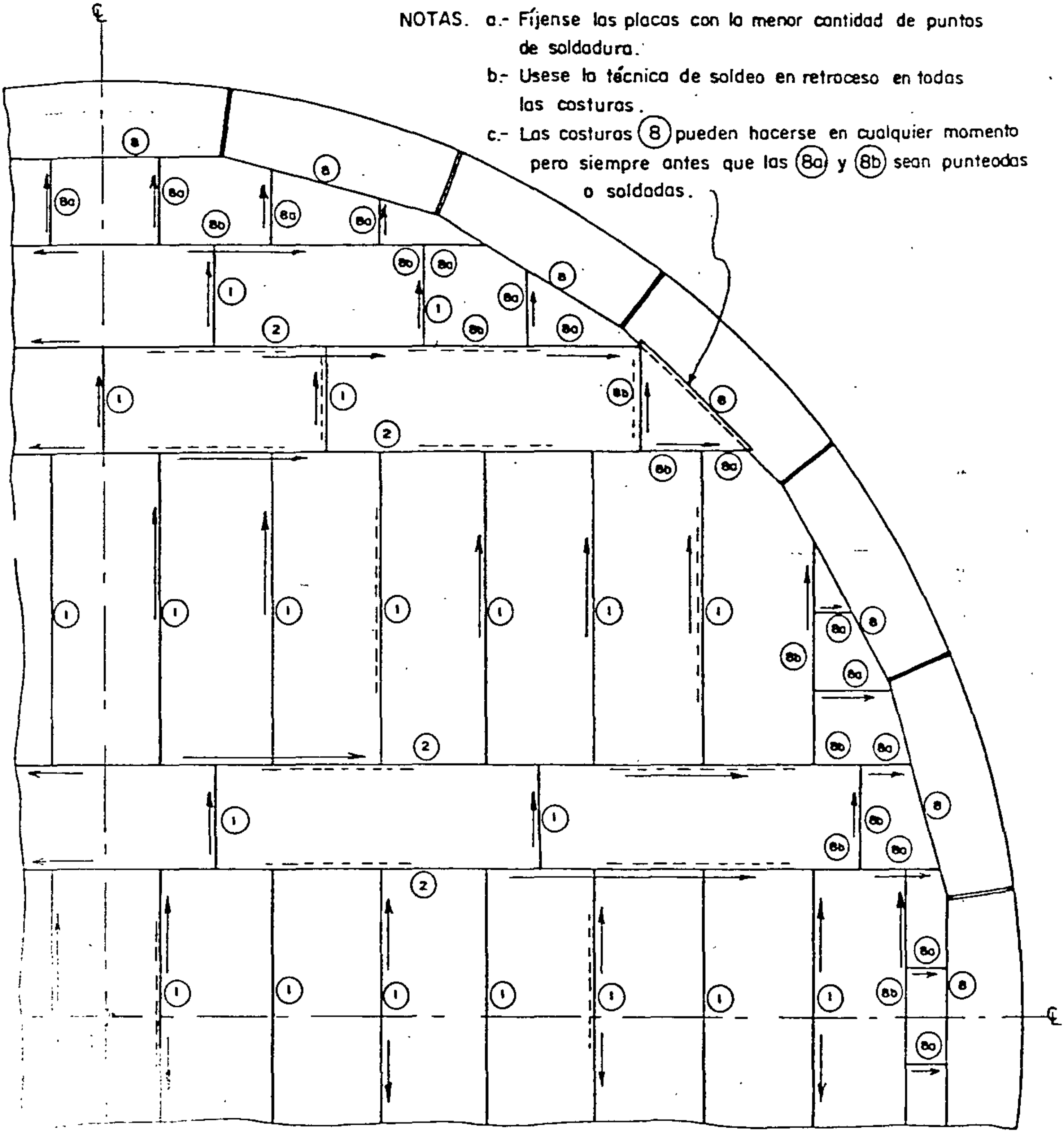


FIG. 2.1.3.1A

PEMEX	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR : Ing. I. J. L.	FECHA	HOJA
	APROBADO POR : Ing. J. H. B.	IV-86	12 DE 24
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

NOTAS. a.- Fíjense las placas con el menor número de puntos de soldadura.

b.- Usar la técnica de soldeo en retroceso en todas las costuras.

c.- Las costuras 8 pueden hacerse en cualquier momento, pero siempre antes que las 8a y las 8b sean punteadas ó soldadas.

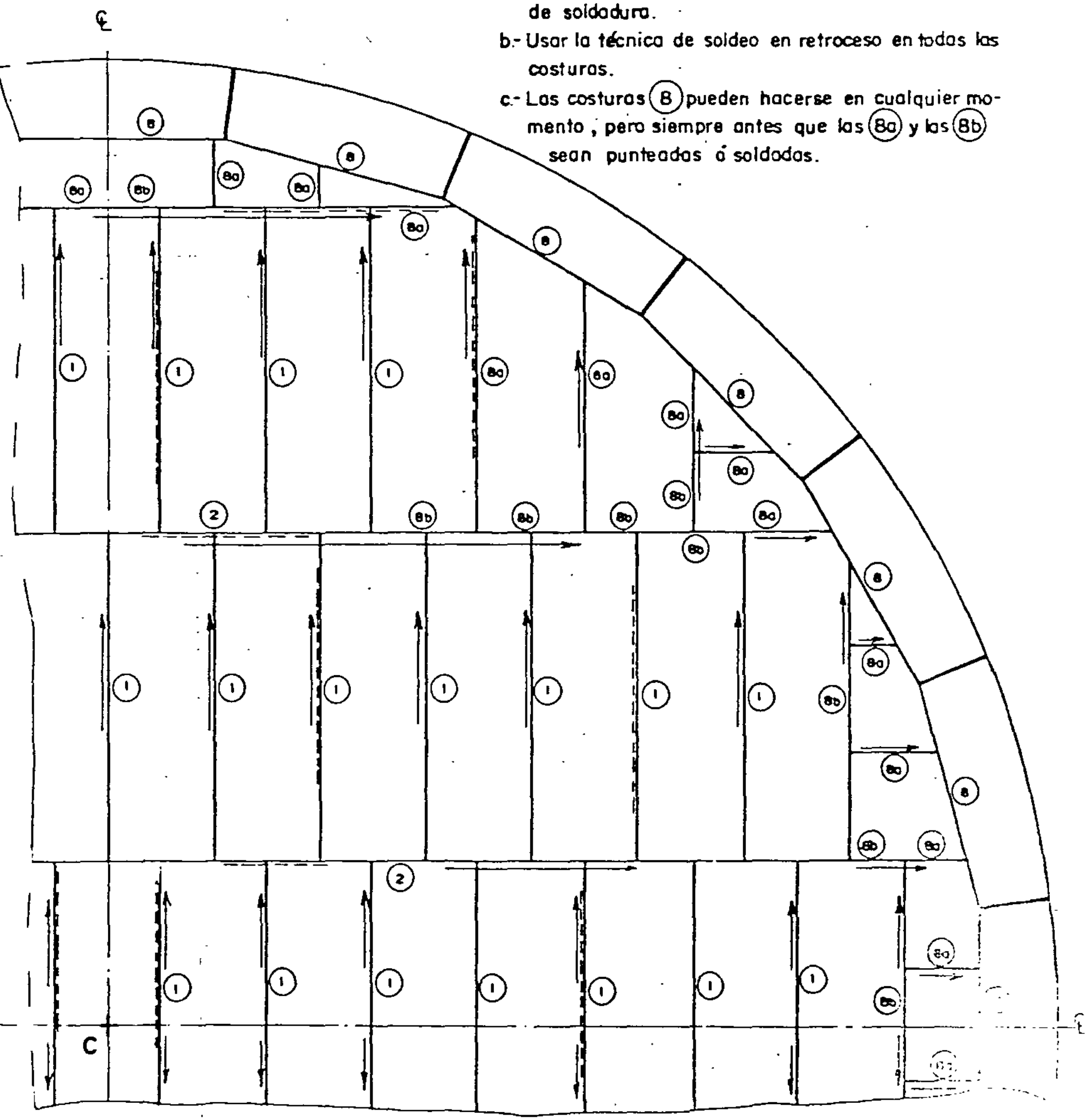
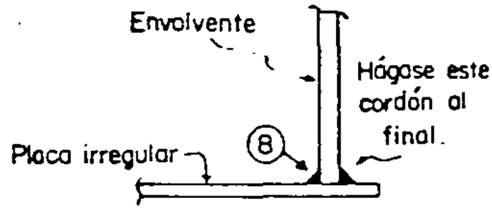


FIG. 2.1.3.18

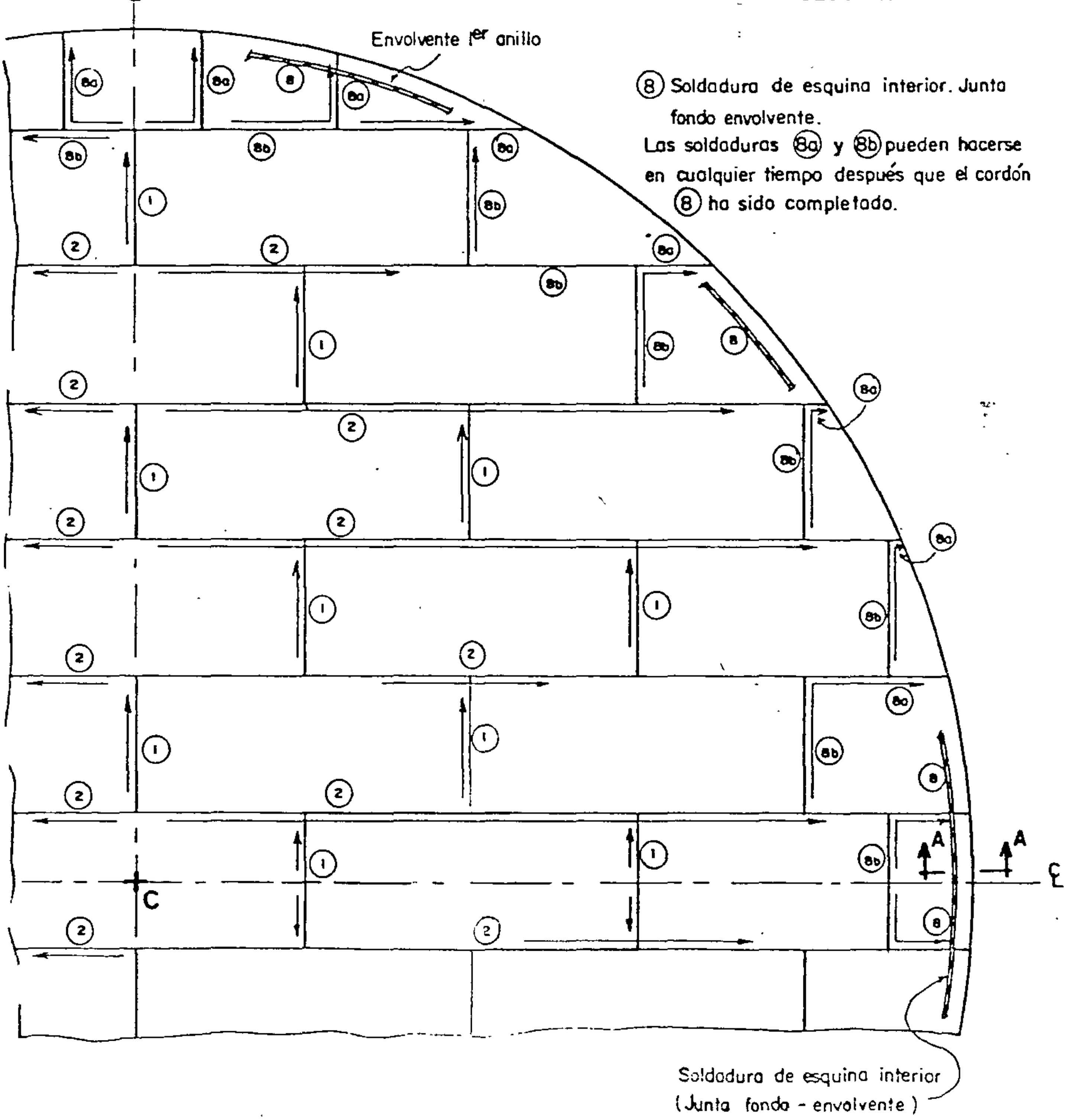
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES. TECHO FLOTANTE	HECHO POR : Ing. I. J. L. / APROBADO POR : Ing. J. H B.	FECHA IV-86	HOJA 13 DE 24
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

NOTAS. a.- Fíjense las placas con un mínimo de puntos de soldadura.

b.- Usese la técnica de soldeo en retroceso en todas las costuras.



SECCION A-A



(8) Soldadura de esquina interior. Junta fondo envolvente.

Las soldaduras (8a) y (8b) pueden hacerse en cualquier tiempo después que el cordón (8) ha sido completado.

DEL CENTRO HACIA LA PERIFÉRIA. LAS JUNTAS DEBERÁN ESTAR LIBRES DE PUNTOS DE SOLDADURA CUANDO SE SUELDEN. USAR -- ELECTRODOS E-6010 (AWS) A MENOS QUE SE ESPECIFIQUE OTRO EN LOS PLANOS DE MONTAJE.

NOTA: PARA TENER EN CUENTA LA CONTRACCIÓN EN TANQUES CON DIÁMETROS MAYORES DE 60 METROS (200'). SOLDAR JUNTAS ALTERNADAMENTE AL MISMO TIEMPO QUE SE ELIMINAN LOS PUNTOS EN LAS JUNTAS NO SOLDADAS AÚN.

2) SOLDAR LAS JUNTAS ENTRE HILERAS Y FILAS EN FORMA ININTERRUMPIDA Y SIEMPRE DEL CENTRO HACIA LA PERIFÉRIA.

3) AJUSTAR LOS TRASLAPES BAYONETEADOS EN LAS ESQUINAS DE LAS PLACAS IRREGULARES. (VÉANSE LOS PÁRRAFOS 2.2.1 Y 2.2.2 - PARA INSTRUCCIONES DE AJUSTE Y SOLDEO DE TRASLAPES EN -- TRES DIRECCIONES).

4) MONTAR EL PRIMER ANILLO DE LA ENVOLVENTE SIGUIENDO LAS - INSTRUCCIONES DADAS EN LA SECCIÓN 3.0 Y SOLDAR LAS JUN-- TAS VERTICALES.

5) AJUSTAR Y SOLDAR UNA DE LAS JUNTAS CIRCUNFERENCIALES ENTRE FONDO Y ENVOLVENTE (SOLDADURA DE ESQUINA) SEGÚN INSTRUCCIONES DEL PÁRRAFO 3.5.

6) SOLDAR O PUNTEAR LAS PLACAS IRREGULARES ENTRE SÍ, ANTES DE MONTAR EL SEGUNDO ANILLO. ESTO IMPEDIRÁ QUE SE INCLINEN Y SE ABRAN ÉSTAS PLACAS, DEBIDO A ASENTAMIENTOS DEL DEL FONDO APOYADO EN BASES DE ARENA.

(7) SOLDAR EL SEGUNDO CORDÓN CIRCUNFERENCIAL DE LA JUNTA - - FONDO-ENVOLVENTE (SOLDADURA DE ESQUINA). ESTO SE PUEDE - HACER POSTERIORMENTE EN EL MOMENTO OPORTUNO.

(8) SOLDAR LAS PLACAS IRREGULARES ENTRE SÍ Y A LAS RECTANGULARES. ESTAS COSTURAS PUEDEN REALIZARSE EN CUALQUIER MOMENTO Y DESPUÉS QUE SE HAN COMPLETADO LAS CIRCUNFERENCIALES DE ESQUINA (JUNTA FONDO-ENVOLVENTE) Y LAS VERTICALES DEL PRIMER ANILLO. EL SOLDEO ENTRE PLACAS RECTANGULARES E IRREGULARES PUEDE LLEVARSE A CABO AL MISMO TIEMPO QUE ÉSTAS SE ESTÁN SOLDANDO ENTRE SÍ, O PODRÁ SOLDARSE UN -- CORDÓN SIN INTERRUPCIÓN, INICIANDOLO EN LAS JUNTAS ENTRE IRREGULARES Y RECTANGULARES Y CONTINUANDOLO HASTA REMA-- TARLO ENTRE LA IRREGULAR CORRESPONDIENTE Y SU ADYACENTE. (VÉASE FIG. 2.1.3.1A, B Y C).

2.1.3.2. TÉCNICA DE SOLDEO PDM. EL ARREGLO DEL TENDIDO DE PLACAS CON HILERAS LONGITUDINALES Y FILAS TRANSVERSALES, EL MÁS COMUNMENTE USADO EN TANQUES DE GRAN DIÁMETRO, SE MUESTRA EN LA -- FIG. 2.1.3.2A PARA EL FONDO DEL TANQUE DE 500,000 BLS. PARA EVITAR GRANDES DEFORMACIONES, SEGUIR LA SECUENCIA DE LA SOLDADURA, MARCADA EN LAS JUNTAS CON NÚMEROS PROGRESIVOS Y RESPETAR LA DIRECCIÓN DEL AVANCE DEL SOLDEO, MARCADO CON FLECHAS.

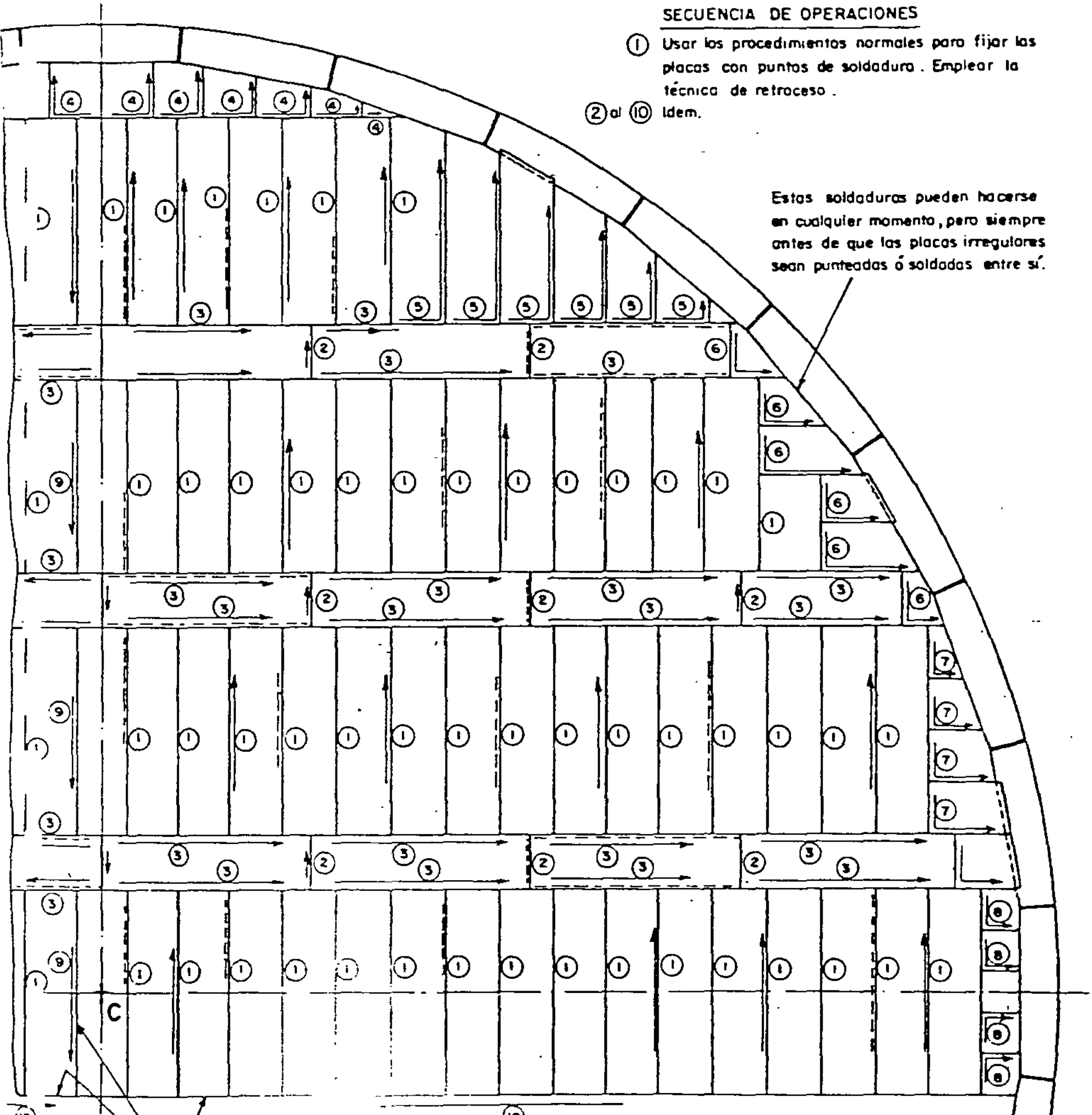
LA FIG. 2.1.3.2B, ENSEÑA EL ARREGLO DE PLACAS, LA SECUENCIA Y LA DIRECCIÓN DE SOLDEO, MEDIANTE NÚMEROS Y FLECHAS, EN FONDOS DE TANQUES DE MEDIANA Y BAJA CAPACIDAD (NORMALMENTE DE - TECHO FIJO). ADEMÁS SE ESTÁN INDICANDO LOS PUNTOS DONDE DEBE INICIARSE LA ÚLTIMA SOLDADURA MARCADA CON LÍNEAS MÁS GRUESAS. COMO COMPLEMENTO A LO ANTERIOR, SÍGANSE LAS INSTRUCCIONES DADAS A CONTINUACIÓN:

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR : Ing. I J. L. / APROBADO POR : Ing. J. H B.	FECHA IV-86	HOJA 16 DE 24
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

SECUENCIA DE OPERACIONES

(1) Usar los procedimientos normales para fijar las placas con puntos de soldadura. Emplear la técnica de retroceso.

(2) al (10) Idem.



NOTAS. a.- Fíjense las placas con el menor número de puntos de soldadura.

b.- Usese la técnica de retroceso en todas las costuras.

c.- Las soldaduras (4) se harán en cualquier tiempo después que la soldadura de esquina interior ha sido completada.

d.- El cordón marcado con línea gruesa se hará al final.

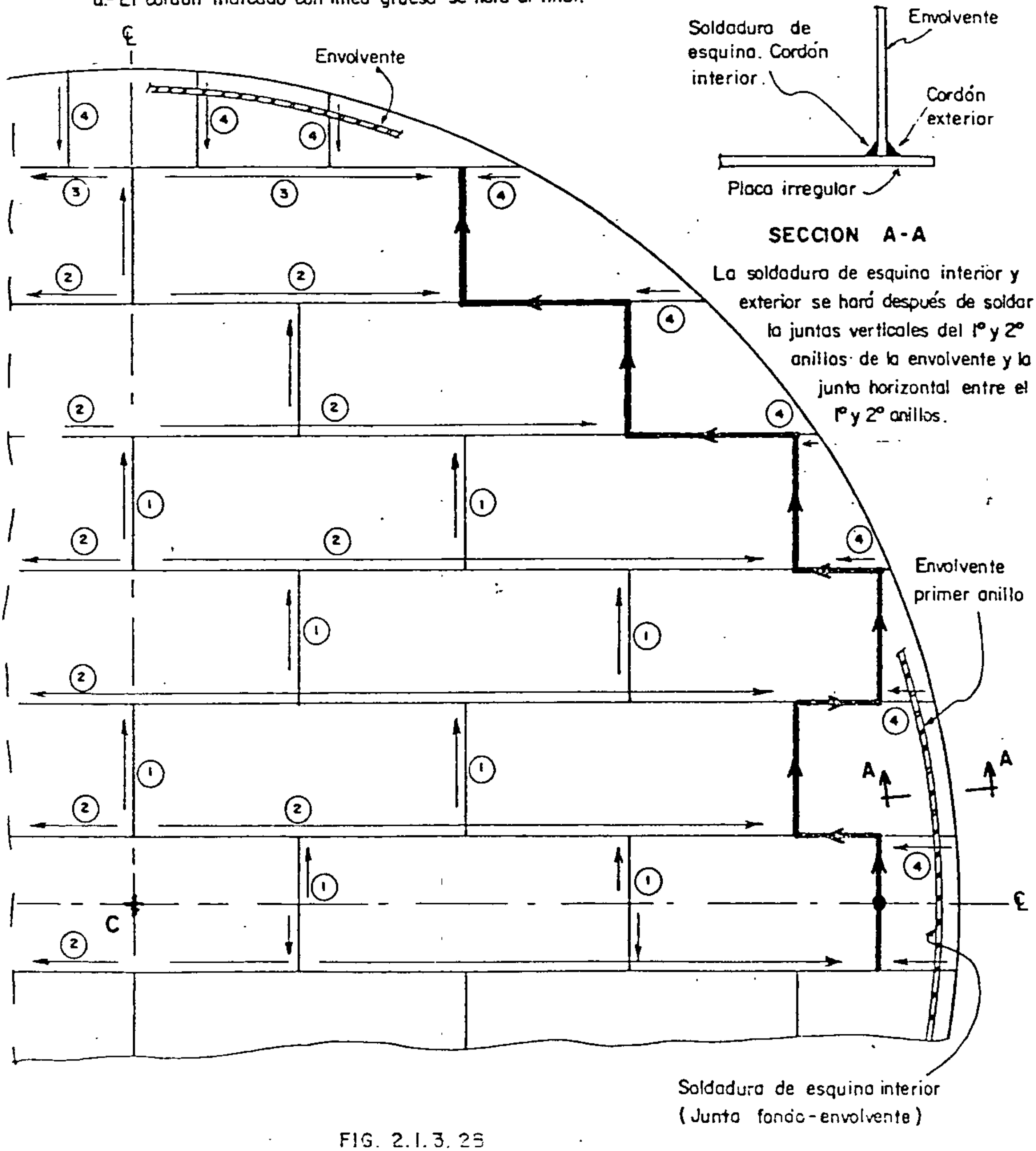


FIG. 2.1.3.25

1. LAS COSTURAS ENTRE PLACAS RECTANGULARES MARCADAS 1 Y 2 PODRÁN SOLDARSE SIN RELACIONARLAS CON LAS SOLDADURAS 4 ENTRE PLACAS IRREGULARES.

2. LAS COSTURAS 4 DE LAS PLACAS IRREGULARES SE SOLDARÁN EN CUALQUIER TIEMPO DESPUÉS QUE EL CORDÓN INTERIOR DE LA JUNTA FONDO-ENVOLVENTE HA SIDO SOLDADO COMPLETAMENTE.

3. LAS COSTURAS 3 DE LAS PLACAS RECTANGULARES DEBERÁN SOLDARSE ANTES QUE LAS JUNTAS ENTRE LAS IRREGULARES Y LAS RECTANGULARES MARCADAS CON LÍNEAS MÁS GRUESAS, SEAN SOLDADAS. LAS DEMÁS SOLDADURAS 1 2 Y 4 SE COMPLETARÁN ANTES QUE SE SUELDA LA 3.

2.2 AJUSTE Y SOLDEO DE ESQUINAS BAYONETEADAS.

2.2.1 DOBLADO DE ESQUINAS BAJO LA ENVOLVENTE: CUANDO EL DISEÑO DE UN FONDO NO CONTEMPLA PLACAS ANULARES, LA ENVOLVENTE SE APOYARÁ EN PLACAS IRREGULARES TRASLAPADAS, PERO PARA MANTENER NIVELADA TODA LA PERIFÉRIA PARA ASENTAR LA ENVOLVENTE, ES NECESARIO MODIFICAR EL TRASLAPE EN LAS ESQUINAS EXTREMAS EXTERIORES, FORMANDO UN CONJUNTO MACHI-HEMBRADO. ESTO SE HARÁ ANTES QUE SE MONTE EL PRIMER ANILLO DE LA ENVOLVENTE. ASEGURARSE QUE LAS PLACAS ESTÁN TRASLAPADAS APROPIADAMENTE ANTES DE HACER EL DOBLADO EN FORMA DE BAYONETA.

SUÉLDESE EL LADO EXTERIOR DE FUERA HACIA EL CENTRO, OMITIR 150 MM. (6") Y SOLDAR 100 MM. (4") MÁS COMO SE INDICA EN LA FIG. 2.2.1A. USAR ELECTRODOS E-6010 Ó E-7018 DE 5 MM. (3/16") O DE MENOR DIÁMETRO PARA SOLDAR LAS ESQUINAS DOBLADAS. NO SE USE EL ELECTRODO E-6012. EN ALGUNOS DISEÑOS ESPECIALES, LAS -

PLACAS IRREGULARES ALCANZAN UN ESPESOR MAYOR DE 8 MM. (5/16") EN CUYO CASO LA ZONA DE DOBLEZ DEBE CALENTARSE A UN ROJO CEREZA ANTES DE GOLPEAR LAS PLACAS PARA EL DOBLADO.

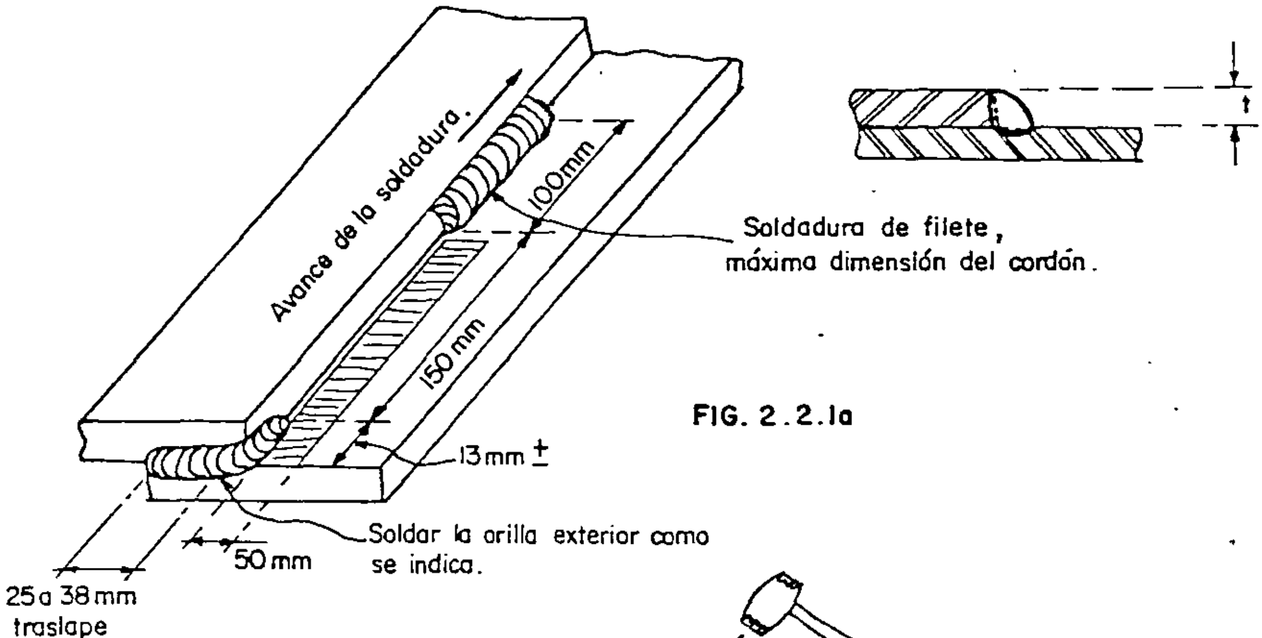


FIG. 2.2.1a

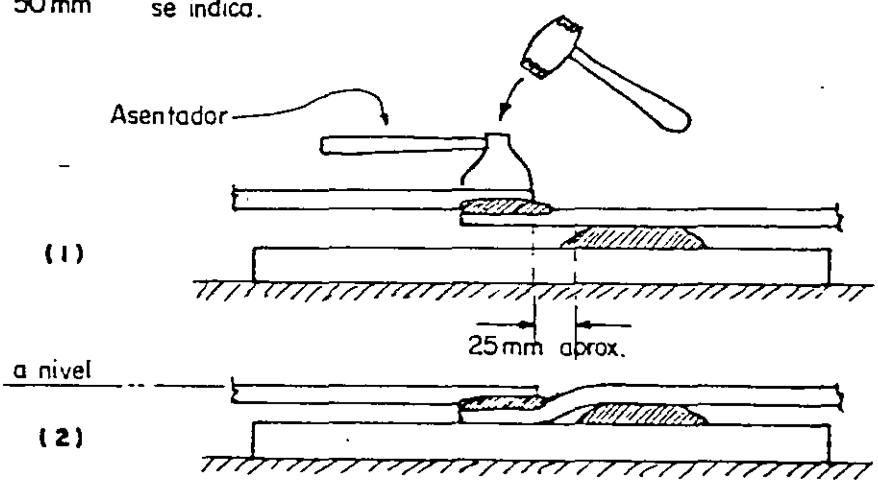


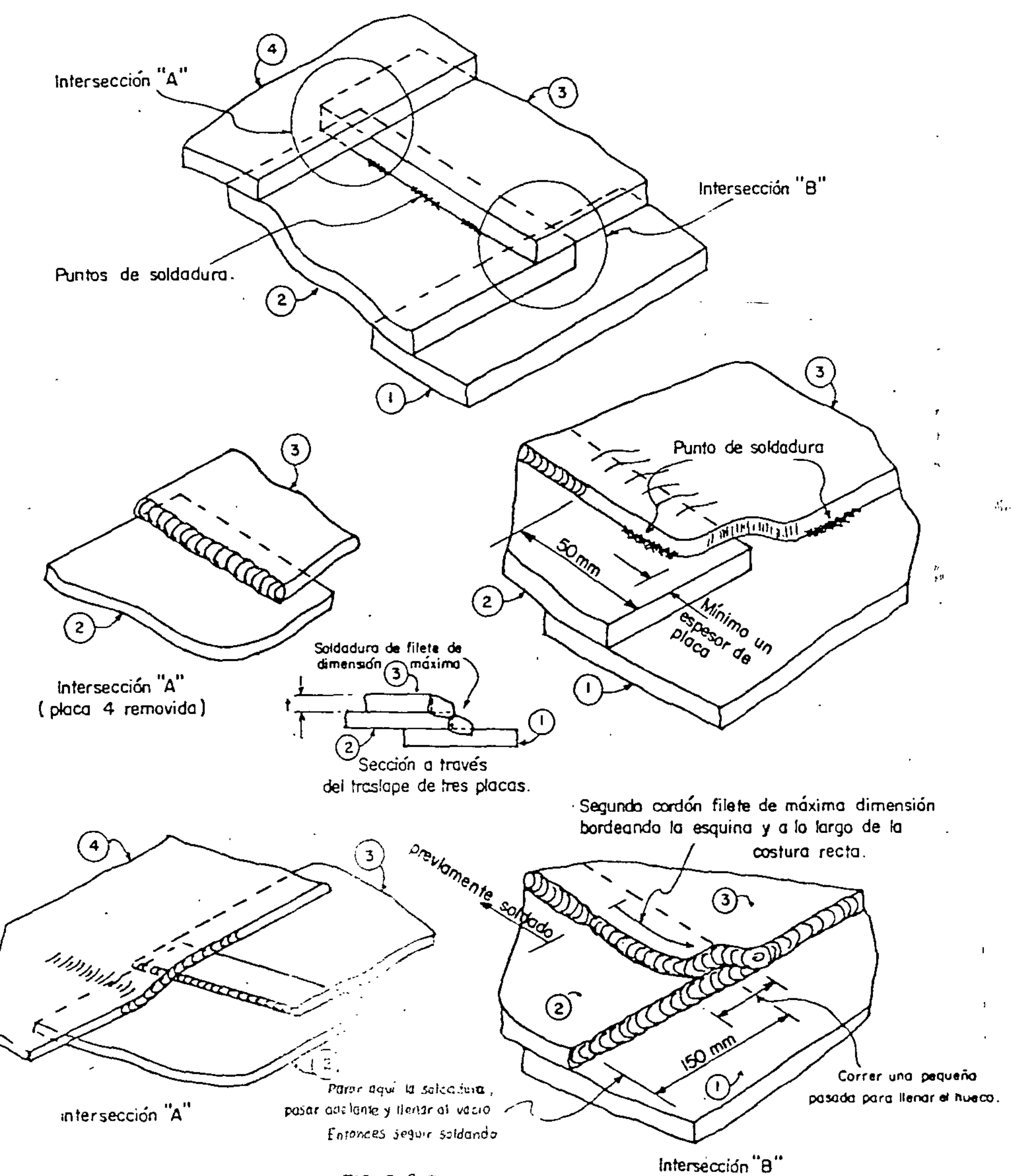
FIG. 2.2.1b

COLOCAR UNA PLACA DE ASIENTO PROVISIONAL CON UN BORDO COMO LA MOSTRADA EN LA FIG. 2.2.1B (1) Y GOLPEAR EL TRASLAPE HACIA ABAJO HASTA QUE LAS CARAS SUPERIORES DE DOS PLACAS IRRE-

GULARES ADYACENTES ESTÉN A NIVEL COMO SE VE EN LA FIGURA - - 2.2.1B (2). EMPEZAR GOLPEANDO DESDE LA ORILLA O LADO EXTERIOR, HACIENDO UN TRASLAPE HERMÉTICO Y CONTINUAR EL GOLPE HACIA -- EL CENTRO ALREDEDOR DE 130 MM. (5"). COMPLETAR LA SOLDADURA EN EL ÁREA DOBLADA, USANDO EL REQUERIDO NÚMERO DE PASADAS -- (DOS MÍNIMAS) PARA HACER UN TRASLAPE SOLDADO COMPLETO Y RETI RAR LA PLACA PROVISIONAL DE ASIENTO PARA QUE LAS IRREGULARES SE APOYEN EN EL ANILLO DE CIMENTACIÓN.

2.2.2 DOBLECES EN TRASLAPES DE TRES PLACAS. CUANDO LAS HILERAS Y - FILAS DE PLACAS RECTANGULARES SE SUELDAN ENTRE SÍ, SE REQUIE RE HACER DOBLECES EN LA INTERSECCIÓN DE TRES PLACAS Y LUEGO SOLDAR. PREVIAMENTE DEBERÁ LIMPIARSE EL ÁREA DE PUNTOS DE -- SOLDADURA. LA FIG. 2.2.2 MUESTRA LAS DOS INTERSECCIONES TÍPI CAS A Y B QUE REQUIEREN UNA ATENCIÓN ESPECIAL PARA SOLUCIONAR SUS TRASLAPES Y FIJAR LOS PROCEDIMIENTOS DE SOLDADURA MÁS -- ADECUADOS. LAS PLACAS SON NUMERADAS PARA SU IDENTIFICACIÓN. LOS NÚMEROS CORRESPONDEN A LA SECUENCIA DE ENSAMBLADO.

EN LA INTERSECCIÓN "A", LA PORCIÓN DE LAS PLACAS 2 Y 3 CUBIER TAS POR LA PLACA 4 DEBE SER SOLDADA MIENTRAS LA JUNTA COMPLE TA ES PUNTEADA ANTES DE TENDER LA 4. SI ÉSTA YA HA SIDO COLO- CADA EN SU LUGAR, DEBERÁ CORRERSE MEDIANTE PALANQUEO, DE MO- DO QUE LA PORCIÓN BAJO LA PLACA SEA DESCUBIERTA Y PUEDA SOL-- DARSE. SI ÉSTA PARTE DE LA JUNTA NO FUÉ SOLDADA DURANTE EL EN SAMBLADO, LA PLACA 4 DEBE REMOVERSE PARA QUE EL SOLDADOR PUE- DA SOLDAR TODO EL CORDÓN. DÓBLECE LA PLACA 4 EN FORMA DE BAYO NETA PARA QUE ASIENTE SOBRE LA 2 Y SUÉLDESE LA JUNTA FORMADA POR 4 3 Y 2 DE UN MODO CONTINUO SIN INTERRUMPIRLA EN EL TRAS LAPE PARA IMPEDIR QUE SE PRODUZCA UN CORTE O MUESCA QUE PODRÍA ORIGINAR UNA GRIETA QUE PUDIERA PROLONGARSE A LO LARGO DE LA COSTURA DESPUES, CUANDO EL TANQUE ESTÉ EN SERVICIO



Intersección "A"
Intersección "B"
Puntos de soldadura.
Punto de soldadura
50 mm
Mínimo un espesor de placa
Intersección "A"
(placa 4 removida)
Soldadura de filete de dimensión máxima
t
Sección a través del traslape de tres placas.
Segunda cordón filete de máxima dimensión bordeando la esquina y a lo largo de la costura recta.
Previamente soldado
150 mm
Correr una pequeña pasada para llenar el hueco.
Parar aquí la soldadura, pasar adelante y llenar al vacío
Entonces seguir soldando
intersección "A"
Intersección "B"

FIG. 2.2.2

CUANDO SE SUELDE EL CORDÓN ENTRE LAS PLACAS (3) Y (2) (INTERSECCIÓN B) PARAR LA SOLDADURA MÁS O MENOS 50 MM. (2") AL EXTREMO DE LA JUNTA. ESTOS ÚLTIMOS 50 MM. SE SOLDARÁN - - CUANDO SE HAGA LA COSTURA DE LA PLACA (2) A LA (1)

LA ESQUINA DE LA PLACA SUPERIOR (PLACA 3) EN LA INTERSECCIÓN B SE RECORTARÁ CON ARCO-AIRE O CINCEL, COMO SE MUESTRA EN LA FIGURA DE MODO QUE LA PLACA (2) SOBRE SALGA POR - LO MENOS, UN ESPESOR DE PLACA FUERA DE LA PLACA (3) ESTO - DEJA ESPACIO PARA UNA SOLDADURA DE FILETE COMPLETA DE LA PLACA (3) A LA (2)

CUANDO SE AJUSTAN LAS COSTURAS DE LAS PLACAS (3) Y (2) A LA (1) DEBERÁ HACERSE UN DOBLEZ EN FORMA DE BAYONETA PARA MINIMIZAR EL VACÍO EN EL TRASLAPE. COMUNMENTE SE GOLPEA LA PLACA CON UN MARTILLO PARA AYUDAR A CERRAR LA ABERTURA. EN TAN-- QUES QUE OPERAN A BAJAS TEMPERATURAS (CRIOGÉNICOS) NO SE - PERMITE EL MARTILLEO DEL MATERIAL. EL VACÍO DEBERÁ RELLE-- NARSE CON METAL DE SOLDADURA.

SUÉLDESE EL TRASLAPE DE TRES PLACAS DEPOSITANDO EL PRIMER CORDÓN SIN INTERRUPCIÓN A LO LARGO DE LA JUNTA ENTRE LAS - PLACAS (2) Y (1) Y AVANZANDO HACIA EL TRASLAPE. SI HAY UN VACÍO EN ÉSTE, DETENER EL CORDÓN DE SOLDADURA A 150 MM. (6") DE DISTANCIA DEL TRASLAPE, SALTARSE Y RELLENAR LA ABERTURA -- CON SOLDADURA; REGRESARSE LOS 150 MM. Y SOLDAR UN CORDÓN - CONTINUO, Y DETENERSE HASTA UNA DISTANCIA DE MÁS O MENOS - 150 MM. (6") ADELANTE DEL TRASLAPE.

EN SEGUIDA, COMPLETAR LA SOLDADURA DE LA PLACA SUPERIOR (3), EMPEZANDO EN EL EXTREMO DONDE SE DETUVO EL CORDÓN TRANSVERSAL Y CONTINUAR ÉSTE ALREDEDOR DE LA ESQUINA DE LA PLACA (3)

CON UNA SOLDADURA DE FILETE CORDÓN COMPLETO Y TERMINARLO ADELANTE DEL TRASLAPE HASTA LIGARLO CON EL CORDÓN LONGITUDINAL PREVIAMENTE SOLDADO.

PARA COMPRENDER MEJOR LAS EXPLICACIONES DADAS EN EL TEXTO, ES CONVENIENTE TENER ENFRENTE LA FIGURA 2.2.2. COMPLETA.

2.3 PLACAS DE APOYO

ESTAS PLACAS SE USAN A VECES COLOCÁNDOLAS ENTRE LA ANULAR Y EL ANILLO DE CIMENTACIÓN, BAJO LA ENVOLVENTE PARA TRANSFERIR LAS CARGAS DE LA MISMA A LA CIMENTACIÓN. NO SE SUELDEN LAS PLACAS UNA A LA OTRA. AL SOLDAR LA PLACA DE APOYO

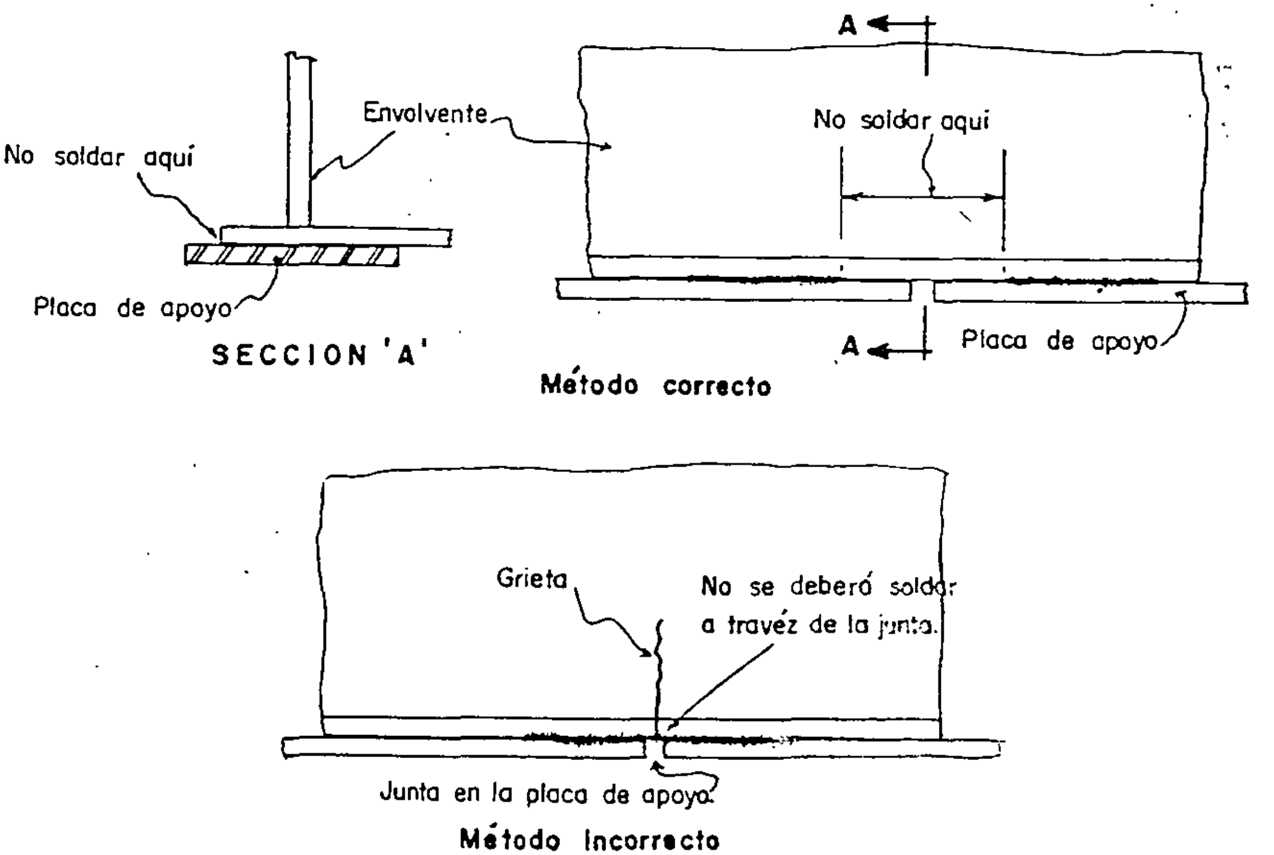


FIG. 2.3

A LA ANULAR DEL FONDO, DÉJESE SIN SOLDAR UNA PARTE EN LA JUNTA DE SEPARACIÓN DE LAS PLACAS DE APOYO. LA FIGURA -- 2.3 INDICA LAS ÁREAS DONDE DEBE OMITIRSE LA SOLDADURA, - PARA EVITAR LA FORMACIÓN DE GRIETAS EN LA ENVOLVENTE - - CUANDO SE SUELDE DE CORRIDO.

3.0 ERECCION DE LA ENVOLVENTE. GENERALIDADES

LAS OPERACIONES DE UNA CONSTRUCCIÓN CONTINUA DEPENDEN DE UN PERSONAL BIEN ORGANIZADO. EL SISTEMA DE MOVIMIENTOS EN ESPIRAL EN LA ENVOLVENTE, SE HA ENCONTRADO QUE ES MUY EFICIENTE. ACOMODAR LA MÁQUINA DE SOLDAR AUTOMÁTICA Y EL EQUIPO DE MONTAJE DE MODO QUE SIGAN SIEMPRE EL MOVIMIENTO EN ESPIRAL EN SENTIDO CONTRARIO A LAS MANECILLAS DEL RELOJ, ES BUENA TÁCTICA. IGUALMENTE ÉSTO MISMO DEBERÍA HACERSE, SI EL PROCEDIMIENTO DE SOLDADURA ES MANUAL.

A CONTINUACIÓN SE INDICA UN MÉTODO GENERAL DE MONTAJE. LAS INSTRUCCIONES DETALLADAS SE EXPONEN MÁS ADELANTE EN LOS PÁRRAFOS 3.1. AL 3.7 INCLUSIVE.

1. MONTAR EL ANILLO NÚMERO 1.

2. FIJAR Y SOLDAR LAS JUNTAS VERTICALES DEL ANILLO No. 1 (EXCEPTO LAS VERTICALES DE LAS PLACAS CORRESPONDIENTES A LAS PUERTAS DE LIMPIEZA).

3. AJUSTAR Y SOLDAR LA JUNTA CIRCUNFERENCIAL ENTRE LAS PLACAS DEL PRIMER ANILLO DE LA ENVOLVENTE Y LAS ANULARES O IRREGULARES DEL FONDO (SOLDADURA DE ESQUINA).

4. MONTAR DOS (2) PLACAS DEL SEGUNDO ANILLO.

5. AJUSTAR, FIJAR Y SOLDAR LA JUNTA VERTICAL EN ESTAS DOS PLACAS.

6. MONTAR LA MÁQUINA DE SOLDAR AUTOMÁTICA Y REVISAR SU ALINEACIÓN (OMITIR ESTE PASO, SI SE VA A SOLDAR MANUALMENTE)

7. CONTINUAR LA ERECCIÓN DEL SEGUNDO ANILLO AJUSTADO, FIJANDO Y SOLDANDO SUS JUNTAS VERTICALES.

8. AJUSTAR Y SOLDAR LA JUNTA HORIZONTAL ENTRE EL PRIMERO Y EL SEGUNDO ANILLO.

9. MONTAR LOS ANILLOS RESTANTES: 3, 4, ETC., SIGUIENDO LA MISMA SECUENCIA. SOLDAR SIEMPRE LAS JUNTAS VERTICALES ANTES QUE LAS HORIZONTALES. VÉASE LA FIGURA 3.0 DONDE SE MUESTRA LA SECUENCIA QUE SE SIGUE EN EL SOLDEO DE LAS JUNTAS DE LA ENVOLVENTE.

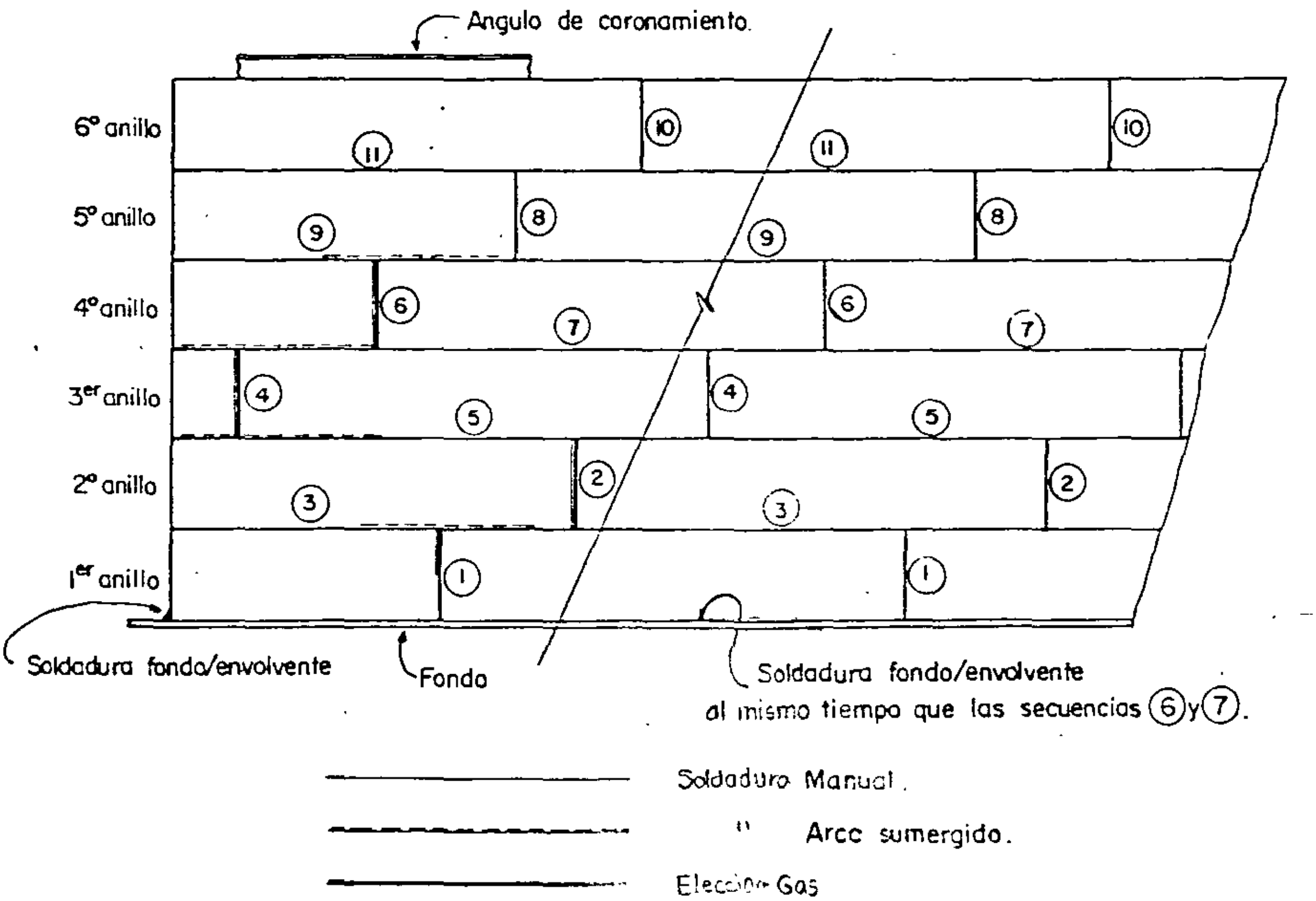


FIG. 3.0

3.1 TRAZOS PREVIOS AL MONTAJE DEL PRIMER ANILLO.

3.1.1 CENTRO Y EJES DEL TANQUE. REVISAR, PARA ASEGURARSE QUE EL CENTRO HA SIDO EXACTAMENTE TRANSFERIDO DE LA BASE A LA PLACA CENTRAL DEL FONDO. CUANDO LAS PLACAS ANULARES (O LAS IRREGULARES) SE HAN TENDIDO Y AJUSTADO, TRANFERIR LOS EJES N-S Y E-W MARCADOS EN EL ANILLO DE CIMENTACIÓN, A DICHAS PLACAS. USAR TRÁNSITO O UN HILO A REVENTÓN PARA EFECTUAR ESTA OPERACIÓN. MARCAR LOS EJES CON UNA SERIE DE PUNTOS TRAZADOS RADIALMENTE HACIA EL CENTRO DEL TANQUE DESDE LA ORILLA EXTERIOR DEL FONDO, MÁS O MENOS UNA DISTANCIA DE 150 MM (6"). ESTO HARÁ VISIBLES LOS EJES DESDE EL EXTERIOR Y EL INTERIOR DE LA ENVOLVENTE. PINTAR ÉSTAS MARCAS PARA QUE SIEMPRE SEA FÁCIL LOCALIZARLAS.

3.1.2 TRAZOS AUXILIARES PARA EL MONTAJE DE LA ENVOLVENTE.

1. ENGANCHAR LA ARGOLLA EXTREMA DE UNA CINTA METÁLICA DE MEDIR, EN EL PERNO SOLDADO EN LA PLACA CENTRAL DEL FONDO Y TRAZAR TRES CÍRCULOS CONCÉNTRICOS DE REFERENCIA: EL PRIMERO, CON UN RADIO AL MEDIO ESPESOR DE LAS PLACAS DEL PRIMER ANILLO DE LA ENVOLVENTE, EL SEGUNDO CON EL RADIO INTERIOR DEL TANQUE TOMADO DE LOS PLANOS DE MONTAJE Y EL ÚLTIMO CON UN RADIO DE 25 MM. MENOR QUE EL SEGUNDO (VÉASE FIG. 3.1.2A). INCREMENTAR A LOS VALORES ANTERIORES EL CORRESPONDIENTE AL RADIO DEL PERNO Y EL EXTREMO DE LA CINTA.

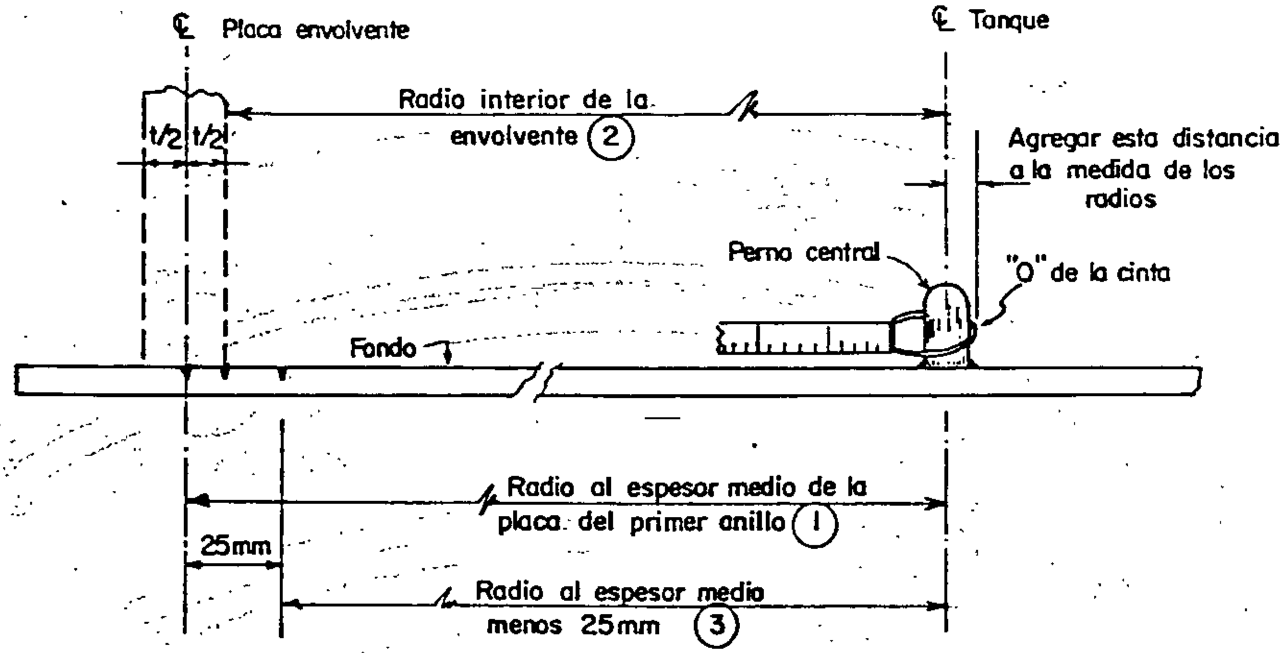


FIG. 3.1.2A

2. EMPEZANDO EN EL PUNTO DONDE SE INICIA EL MONTAJE DE LA ENVOLVENTE, TRAZAR LOS EXTREMOS DE LAS CUERDAS DE TODAS Y CADA UNA DE LAS PLACAS DEL PRIMER ANILLO, TRABAJANDO INDEPENDIENTEMENTE LAS DOS MEDIAS CIRCUNFERENCIAS EN DIRECCIONES OPUESTAS, PARA REDUCIR EL ERROR ACUMULATIVO. LA LONGITUD DE CADA CUERDA SE MEDIRÁ SOBRE EL CÍRCULO QUE CORRESPONDE AL MEDIO ESPESOR DE LAS PLACAS (EJE DEL ANILLO) VÉASE LA FIGURA 3.1.2B. SI LAS LOCALIZACIONES FINALES EN CADA DIRECCIÓN NO COINCIDEN, DIVIDIR EL ERROR ENTRE EL NÚMERO DE CUERDAS, INCREMENTAR SU LONGITUD CON EL COCIENTE QUE RESULTE Y TRAZARLAS NUEVAMENTE. REPETIR ÉSTA OPERACIÓN HASTA QUE NO HAYA ERROR. MARCAR CON PUNTO Y MARTILLO LOS TRAZOS EXTREMOS DE CADA CUERDA, PROLONGAR LAS MARCAS RADIALMENTE HACIA EL INTERIOR DE LA ENVOLVENTE UNOS 100 MM. Y HACIA EL EXTERIOR, 50 MM. Y PINTARLAS PARA LOCALIZARLAS RÁPIDAMENTE. ESTOS TRAZOS SON MUY IM--

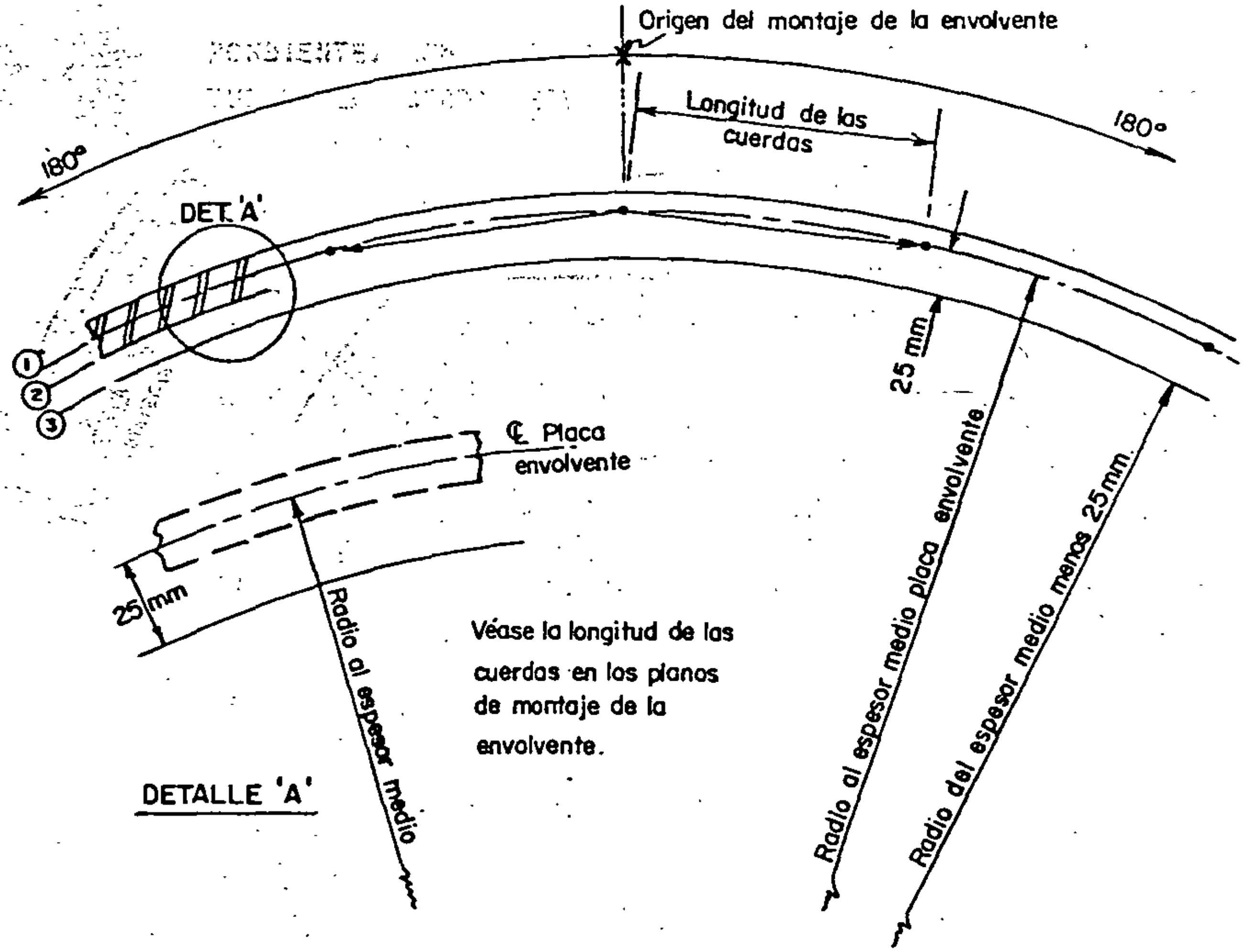


FIG. 3.1.2B

PORTANTES PUÉS MARCAN EL EJE DE LAS JUNTAS VERTICALES Y SIRVEN POR LO TANTO, PARA LOCALIZAR EXACTAMENTE LOS EXTREMOS DE CADA PLACA DEL PRIMER ANILLO DE LA ENVOLVENTE.

UN MÉTODO PRÁCTICO Y RÁPIDO PARA LLEVAR A CABO EL TRABAJO DE TRAZOS DE CUERDAS DESCRITOS EN EL PÁRRAFO ANTERIOR, ES EL DESARROLLADO MEDIANTE EL EMPLEO DE DOS CINTAS DE MEDIR (VÉASE FIG. 3.1.2C). MIENTRAS QUE CON UNA SE ESTÁ MIDIENDO EL RADIO DEL MÉDIO ESPESOR SOBRE EL CÍRCULO CORRES

PONDIENTE, CON LA OTRA SE MIDE AL MISMO TIEMPO LA LONGITUD DE LA CUERDA DESDE EL TRAZO ANTERIOR. LA INTERSECCIÓN

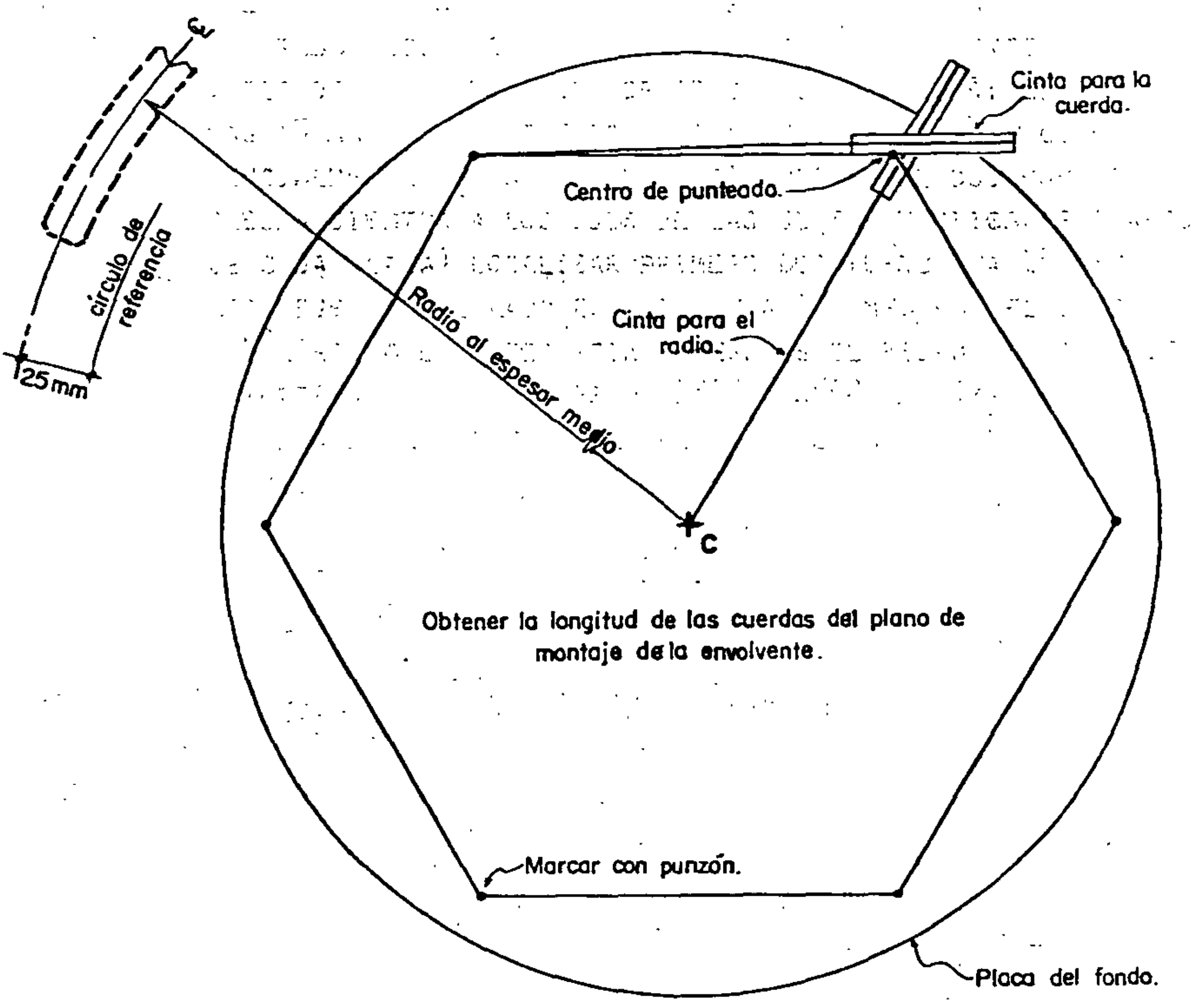


FIG. 3.1.2c

DE LAS DOS CINTAS DÁ EL EJE DE LA JUNTA VERTICAL Y SE MARCA CON EL PUNTO, PROSIGUIENDO EN ESTA FORMA LA OPERACIÓN HASTA COMPLETAR MEDIA CIRCUNFERENCIA. SE DEBE TRABAJAR SIMULTANEAMENTE Y EN LA MISMA FORMA LA OTRA MITAD.

3. PUNTEAR POR PARES EN LAS PLACAS ANULARES O IRREGULARES -- DEL FONDO, UNA SERIE DE TUERCAS LISAS DE 50 X 50 X 25 MM. SEPARADAS DEL EJE DE LA ENVOLVENTE HACIA EL EXTERIOR Y EL INTERIOR, UN MEDIO ESPESOR DE LA PLACA DEL PRIMER ANILLO MÁS 13 MM. EN EL SENTIDO RADIAL (FIG. 3.1.2D) Y CIRCUNFERENCIALMENTE EN CADA ARCO DE CÍRCULO ENTRE DOS MARCAS CORRESPONDIENTES A LOS EJES DE LAS JUNTAS VERTICALES (LARGO DE CADA PLACA) LOCALIZAR PRIMERO DOS TUERCAS A 150 MM. DE CADA EJE POR EL LADO EXTERIOR Y A 600 MM. POR EL INTERIOR DEL CÍRCULO DE REFERENCIA Y DESPUÉS EL RESTO DE LA SERIE A INTERVALOS NO MENORES DE 1800 A 2500 MM. (MIENTRAS MÁS DELGADA ES LA PLACA DE LA ENVOLVENTE, MENOR SERÁ EL ESPACIAMIENTO).

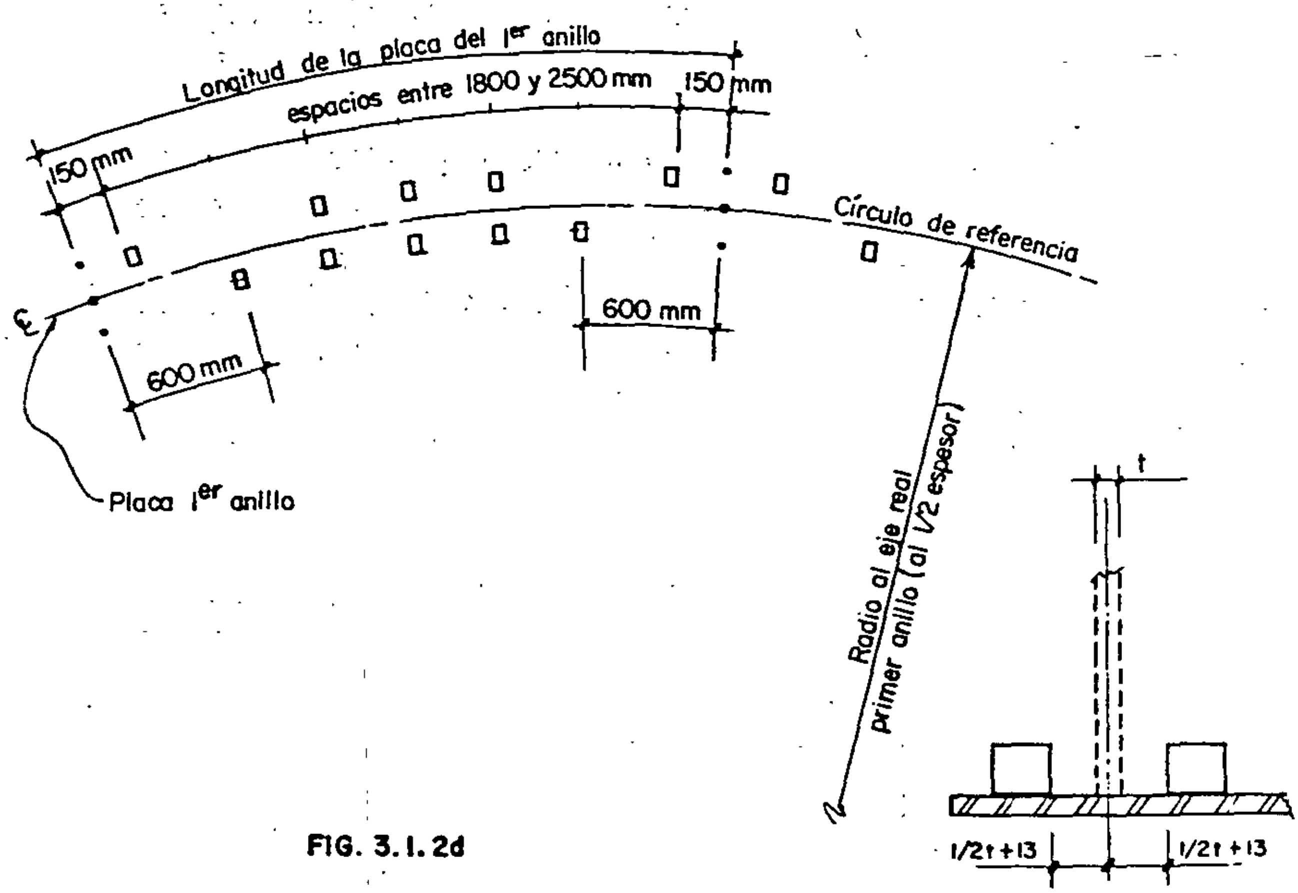


FIG. 3.1.2d

EXISTE UNA VARIACIÓN EN LA LOCALIZACIÓN DE LA TUERCA INTERIOR; CONSISTE EN PUNTEARLA EN EL CÍRCULO TRAZADO CON EL RADIO INTERIOR DE LA ENVOLVENTE O SEA QUE QUEDA ADOSADA A LA CARA INTERIOR DEL ANILLO. ESTAS TUERCAS SON DESPRENDIDAS SI ES NECESARIO MOVER LA ENVOLVENTE HACIA ADENTRO.

3.2 ERECCIÓN DEL PRIMER ANILLO.

CUANDO LOS PLANOS DE MONTAJE INDICAN QUE VARIOS ANILLOS TIENEN LAS MISMAS DIMENSIONES PERO QUE LAS PLACAS ESTÁN MARCADAS CON EL NÚMERO DEL ANILLO CORRESPONDIENTE O TIENEN UNA MARCA ESPECIAL, DEBERÁN SER ORDENADAS POR GRUPOS Y MONTADAS CON LA MARCA DE MONTAJE INDICADA EN EL PLANO RESPECTIVO. AÚN SUPONIENDO QUE NO SE TIENE UN REPORTE DE DISCREPANCIAS, ES CONVENIENTE REVISAR DIMENSIONES PUESTO QUE PUEDE HABER UN ANILLO MÁS ANGOSTO QUE LOS OTROS O PUEDE HABER EN EL MISMO UNA PLACA MÁS LARGA O MÁS CORTA.

SOLDAR EN CADA PLACA DE LA ENVOLVENTE, LAS TUERCAS LISAS PARA LOS CANDADOS SUJETADORES CORRESPONDIENTES A LAS JUNTAS VERTICALES Y PARA LOS RIGIDIZANTES EN LAS JUNTAS HORIZONTALES, ASÍ COMO LAS SOLERAS PARA APOYAR LAS MÉNSULAS DEL ANDAMIAJE, TODO ÉSTO ANTES DE MONTARLAS (VÉASE LA FIG. 3.2A).

LA SOLDADURA DE LAS SOLERAS DE SOPORTE PARA LAS MÉNSULAS, DEBE ESTAR LIMPIA DE ESCORIA, INSPECCIONADA Y HECHA POR UN SOLDADOR CALIFICADO. DEBERÁ SER CALIFICADA Y CIRCULADA CON LAS INICIALES DEL INSPECTOR CALIFICADO QUE HIZO LA REVISIÓN.

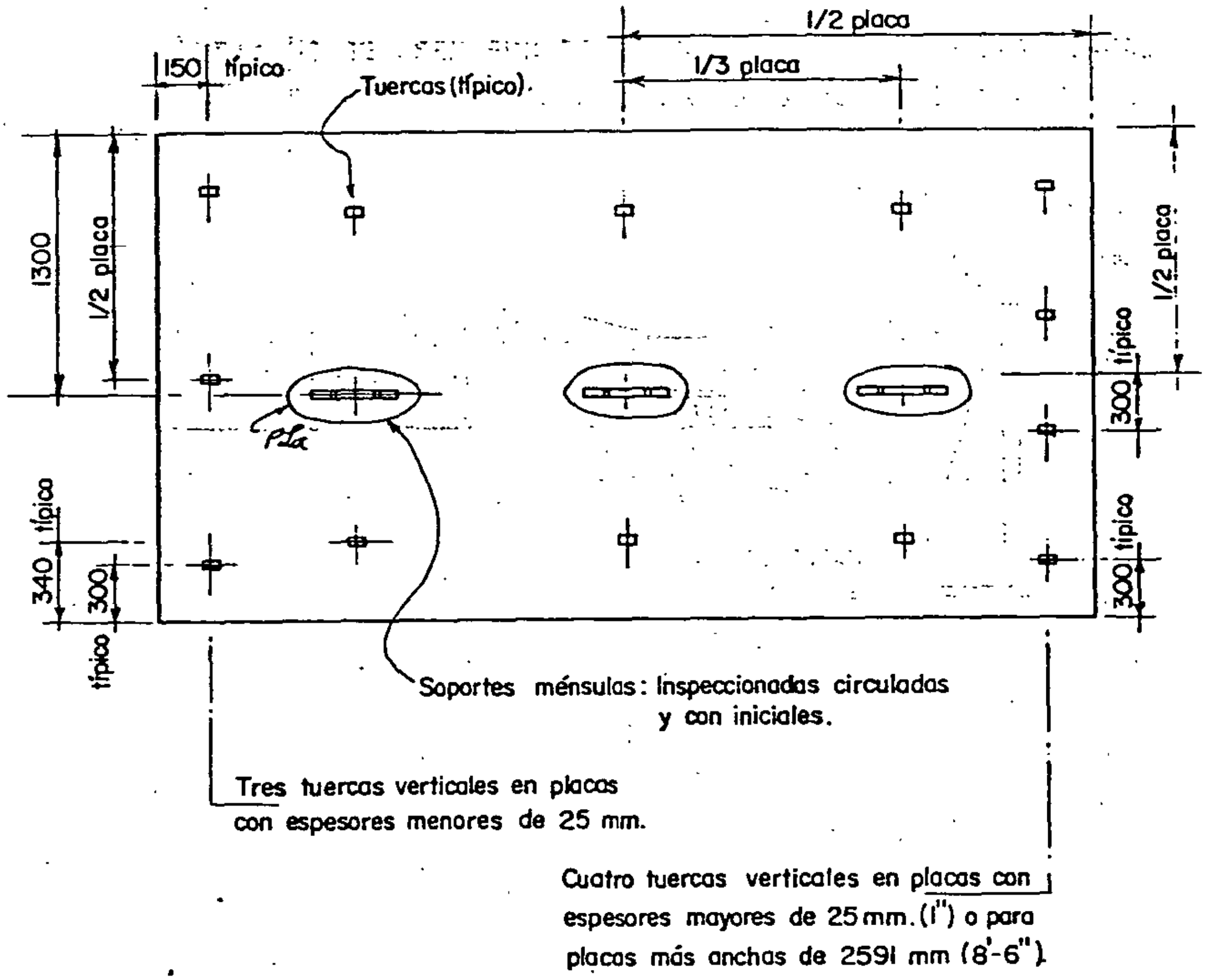


FIG. 3. 2a

MONTAR LAS PLACAS USANDO EL EQUIPO DE LEVANTAMIENTO APROPIADO GRÚA, MONTACARGA, BALANCÍN, PERNOS, ESTROBOS, ETC. (FIGURA -- 3.2B) Y LOS HERRAJES ESPECIFICADOS: CANDADOS, SEPARADORES, -- ETC. (FIG. 3.4.1B y C)

NOTA: NO SE USEN PUNTOS DE SOLDADURA PARA FIJAR PLACAS DE LA ENVOLVENTE UNAS A LAS OTRAS, DURANTE EL MONTAJE DE LAS MISMAS.

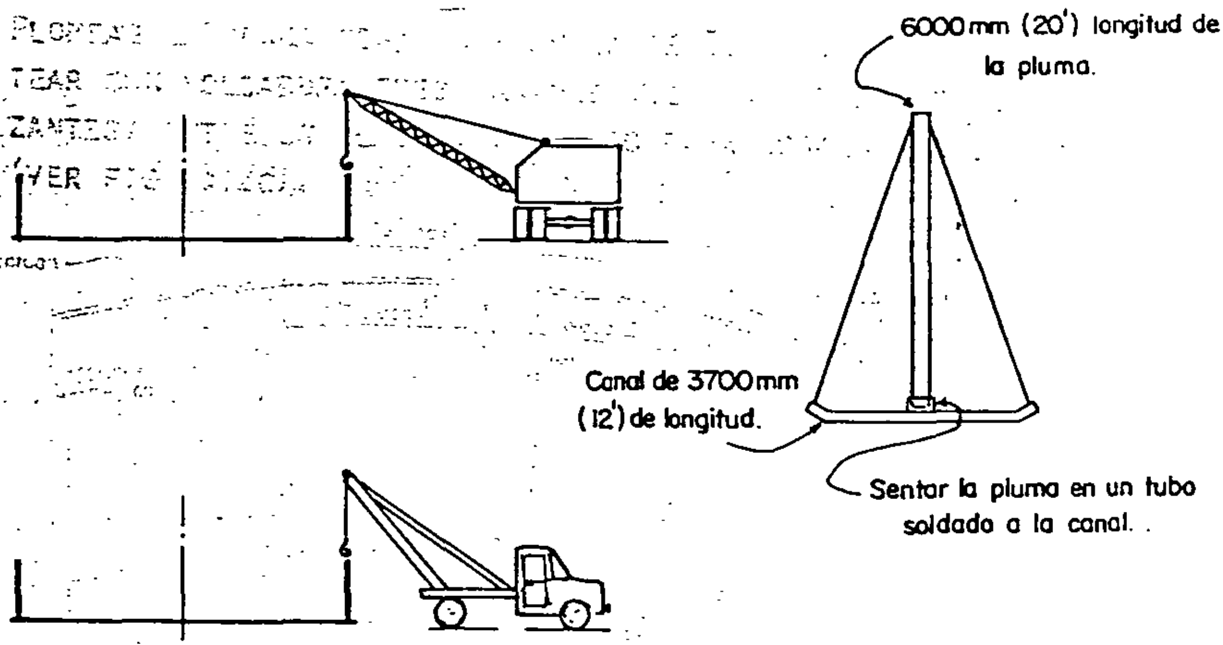


FIG. 3. 2b Equipo para montar el primer anillo.

ENGANCHAR Y TRANSPORTAR A SU LUGAR LA PRIMERA PLACA DE MODO - QUE EL EXTREMO QUE VA A APOYARSE PRIMERO ESTÉ LIGERAMENTE MÁS ELEVADO QUE EL OTRO, PERO LLEVANDO LA PLACA CASI A NIVEL.

SENTAR EL EXTREMO DE LA PLACA EN LA MARCA HECHA PREVIAMENTE - EN EL FONDO QUE INDICA LA LOCALIZACIÓN DE LA JUNTA VERTICAL Y SOSTENERLA. APOYAR TODA LA PLACA EN LA CARA INTERIOR DE LAS - TUERCAS EXTERIORES. MOVER LA PLACA HACIA ADENTRO O HACIA AFUE RA LO NECESARIO PARA SITUAR EL OTRO EXTREMO EN LA MARCA CORRES

PONDIENTE. LAS MARCAS SEÑALADAS CON PUNTOS EN EL FONDO SON -- MUY IMPORTANTES PARA LOCALIZAR PROBLEMAS DE MONTAJE SI HAY -- ERRORES DE FABRICACIÓN.

PLOMEAR LA PLACA CON UNA PLOMADA DE 1.80 M. Ó MÁS LARGA Y PUNTEAR CON SOLDADURA TRES CANALES (DE LAS EMPLEADAS COMO RIGIDIZANTES) ENTRE LA PLACA Y EL FONDO PARA SOSTENERLA EN SU LUGAR (VER FIG. 3.2c).

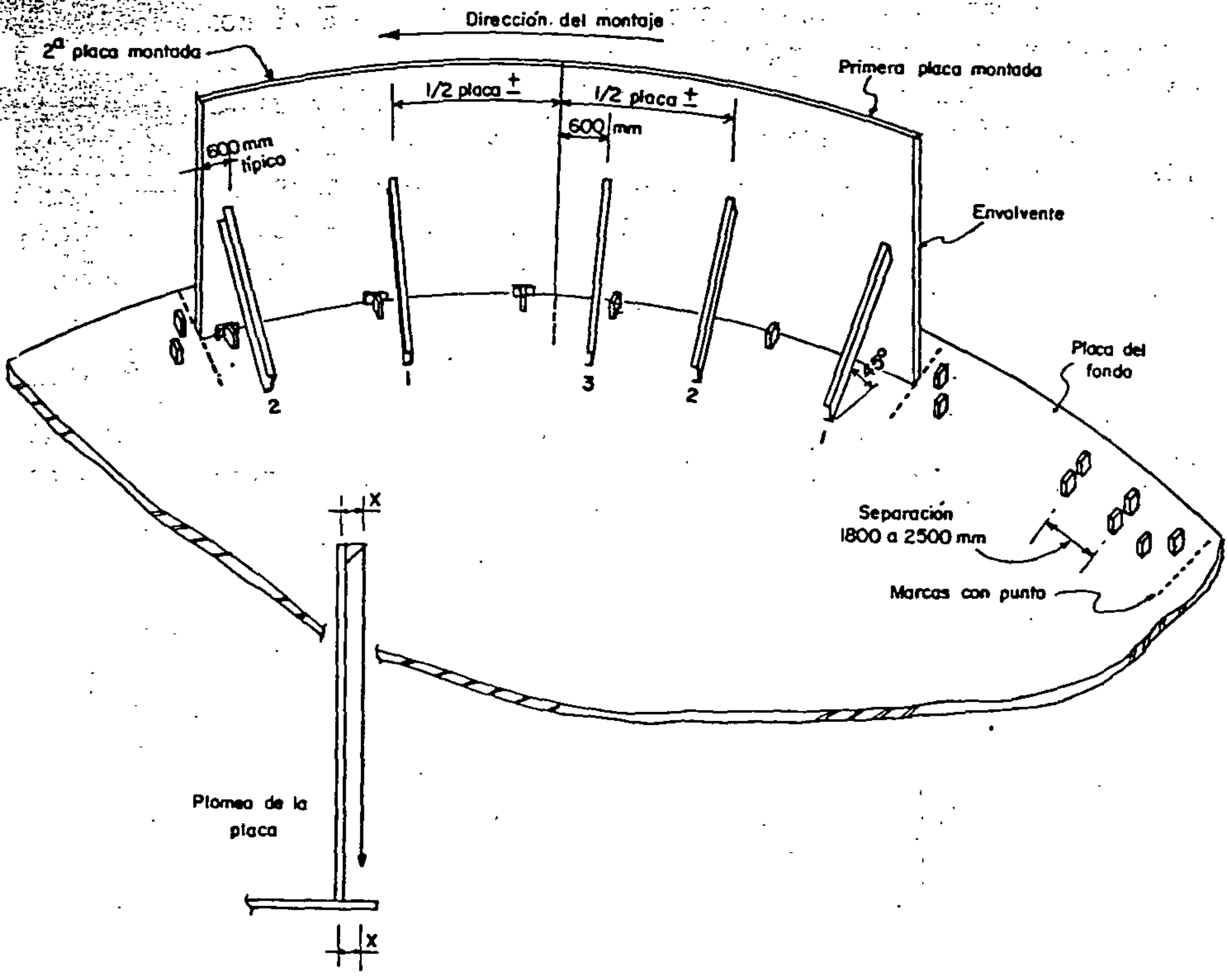


FIG. 3.2c Montaje del primer anillo.

ENGANCHAR Y MONTAR SIN SOLTAR LA SEGUNDA PLACA. HÁGASE COINCIDIR SU ORILLA VERTICAL CON LA DE LA PLACA MONTADA Y FÍJELA A ÉSTA CON UN CANDADO. MOVER LA PLACA HACIA AFUERA O HACIA ADENTRO LO REQUERIDO PARA HACER COINCIDIR EL OTRO EXTREMO CON LA MARCA PUNTEADA EN EL FONDO. FIJAR AMBAS PLACAS CON LOS CANDADOS REQUERIDOS. PARA PLACAS DE 2.44 M. (8') DE ANCHO AFIANZAR LAS PLACAS DE MENOS DE 25 MM. (1") DE ESPESOR CON TRES (3) Ó CUATRO (4) CANDADOS POR JUNTA VERTICAL. PLACAS MÁS GRUESAS Ó CON 2.75 M. (9') O MÁS DE ANCHO, REQUIEREN CUATRO (4) Ó MÁS CANDADOS POR JUNTA VERTICAL. PLOMEAR LA PLACA Y PUNTEAR UNA CANAL A 600 MM. DEL EXTREMO LIBRE Y POR EL LADO INTERIOR Y OTRA A LA MITAD DE LA PLACA PARA SOSTENERLA PLOMEADA. CUANDO LAS PLACAS SE ESTÁN MONTANDO, USAR PUNZONES O CUÑAS (VER FIG. 2.1.1C) ENTRE LAS TUERCAS DEL FONDO Y LAS PLACAS PARA REDONDEAR ÉSTAS ÚLTIMAS Y FIJARLAS EN SU POSICIÓN EXACTA, AUXILIÁNDOSE CON LOS CÍRCULOS DE REFERENCIA 2 Y 3 PREVIAMENTE TRAZADOS (VER FIG. 3.1.2A Y PÁRRAFO 3.1.2).

CONTINUAR MONTANDO PLACAS DEL PRIMER ANILLO DE LA MANERA DESCRITA HASTA CERRARLO. SI EL TANQUE TIENE UNA O MÁS PUERTAS DE LIMPIEZA, VÉASE EL PÁRRAFO SIGUIENTE 3.3 CON EL INSTRUCTIVO PARA LA ERECCIÓN DE LAS PLACAS QUE CONTIENEN ÉSTOS ACCESORIOS.

3.3 PLACAS CON PUERTAS DE LIMPIEZA.

ESTAS PLACAS ASÍ COMO SUS REFUERZOS Y LAS MISMAS PUERTAS DE LIMPIEZA, DEBEN SER DISEÑADAS Y DETALLADAS POR INGENIERÍA DE DISEÑO DE PEMEX PARA QUE POSTERIORMENTE SEAN FABRICADAS EN LOS TALLERES CONTRATADOS. NUNCA DEBERÁN CORTARSE EN EL CAMPO. ANTES DE INICIAR EL MONTAJE DE LA ENVOLVENTE, LA SUPERVISIÓN

REVISARÁ SI EL MATERIAL DE LAS PUERTAS ESTÁ COMPLETO PARA QUE EL MONTAJE DE ÉSTAS NO SE DEJE INCOMPLETO.

PRESENTAR LA, Ó LAS PLACAS EN SU UBICACIÓN CORRECTA COMO SE INDICA EN EL PLANO RESPECTIVO. MANÉJENSE EN LA MISMA FORMA QUE LAS DEMÁS PLACAS DE LOS ANILLOS. DESPUÉS DE HACER COINCIDIR LAS ORILLAS EXTREMAS VERTICALES CON LAS DE LAS PLACAS ADYACENTES, SUJÉTENSE CON CANDADOS Y PLACAS DE SUJECIÓN. NO USAR PLACAS SEPARADORAS. NO DESENGANCHAR EL EQUIPO DE LEVANTAMIENTO, HASTA QUE LOS CANDADOS ESTÉN APRETADOS (VÉASE LA FIG. 3.3A).

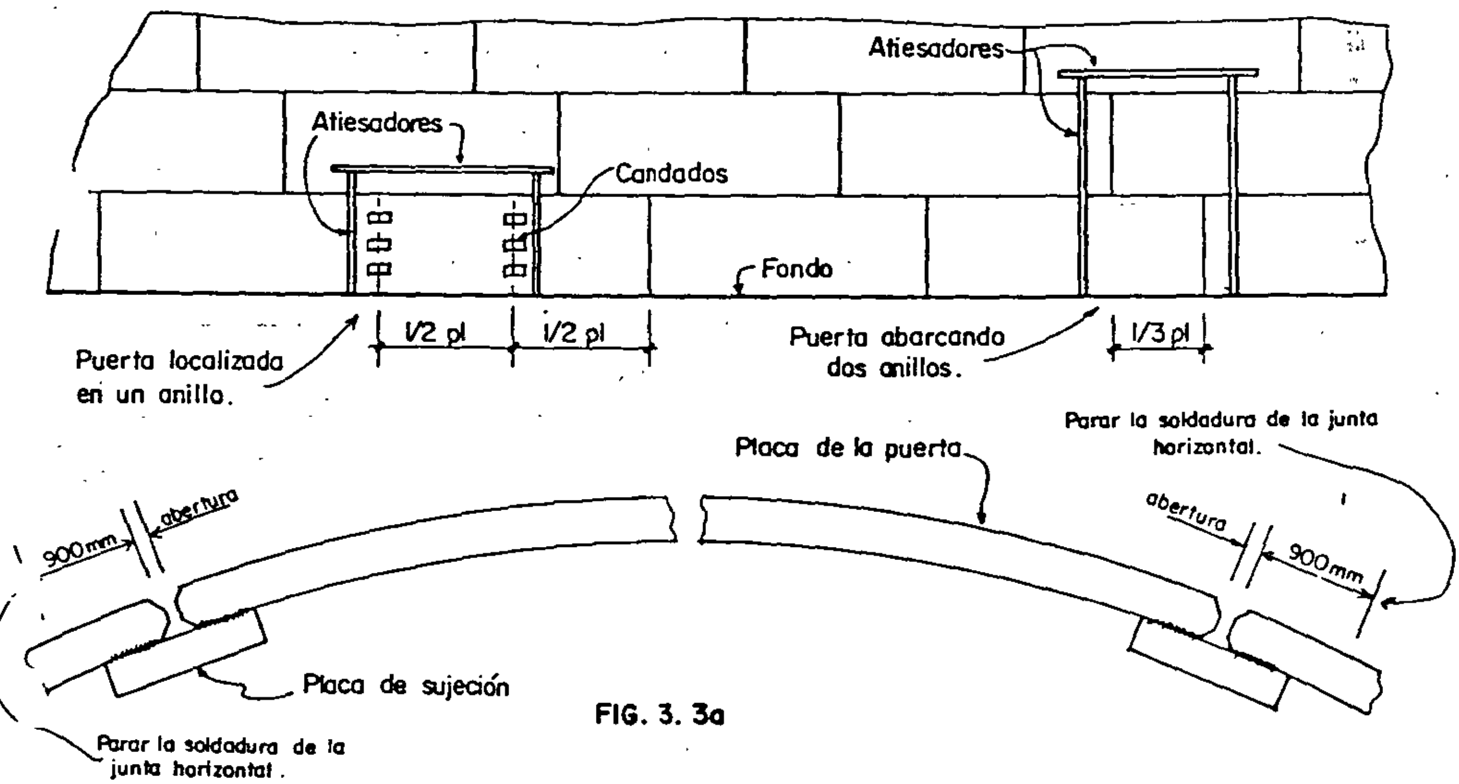


FIG. 3. 3a

UNA VEZ PRESENTADAS, AJUSTADAS Y SUJETADAS LAS PLACAS DE LA PUERTA DE LIMPIEZA, REVISAR LA INSTALACIÓN Y LIBERESE EL EQUIPO DE MONTAJE.

PARA CONTAR CON UNO O MÁS ACCESOS HACIA EL INTERIOR DEL TANQUE, HAY NECESIDAD DE REMOVER LAS PLACAS DE LAS PUERTAS. ESTA REMOCIÓN SE HACE HASTA QUE SEA ABSOLUTAMENTE NECESARIO INTRODUCIR O SACAR DEL TANQUE MATERIALES, EQUIPO Y HERRAMIENTA.

LAS PLACAS DE LAS PUERTAS NO SE QUITARÁN HASTA QUE LAS OPERACIONES SIGUIENTES HAYAN SIDO EJECUTADAS:

1. DOS ANILLOS SUPERIORES, CUANDO MENOS, DEBERÁN ESTAR COMPLETAMENTE SOLDADOS.

2. LA JUNTA CIRCUNFERENCIAL FONDO-ENVOLVENTE Y LA PRIMERA JUNTA HORIZONTAL ENTRE EL PRIMERO Y EL SEGUNDO ANILLO ESTÉN SOLDADAS EXCEPTO 900 MM. MÍNIMOS POR CADA LADO DE LA PLACA (FIG. 3.3B).

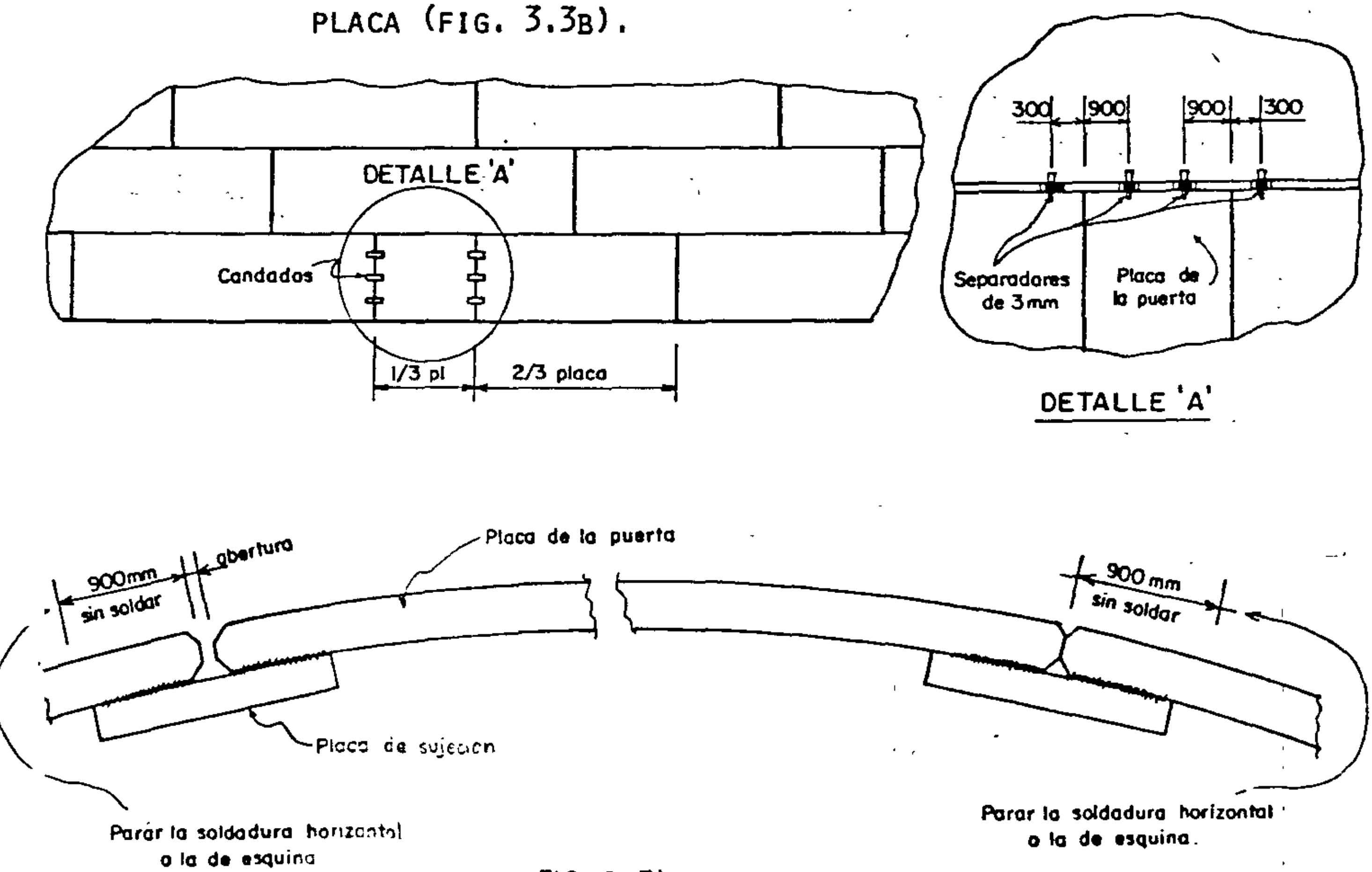


FIG. 3. 3b

3. LA ABERTURA QUE DEJA LA PLACA AL RETIRARLA HA SIDO PERFECTAMENTE ATIESADA CON CANALES DE 3.50 M. DE LONGITUD MÍNIMOS (EL PERFIL DE LA CANAL SERÁ FIJADO POR INGENIERÍA) -- (VÉASE LA FIGURA 3.3C).

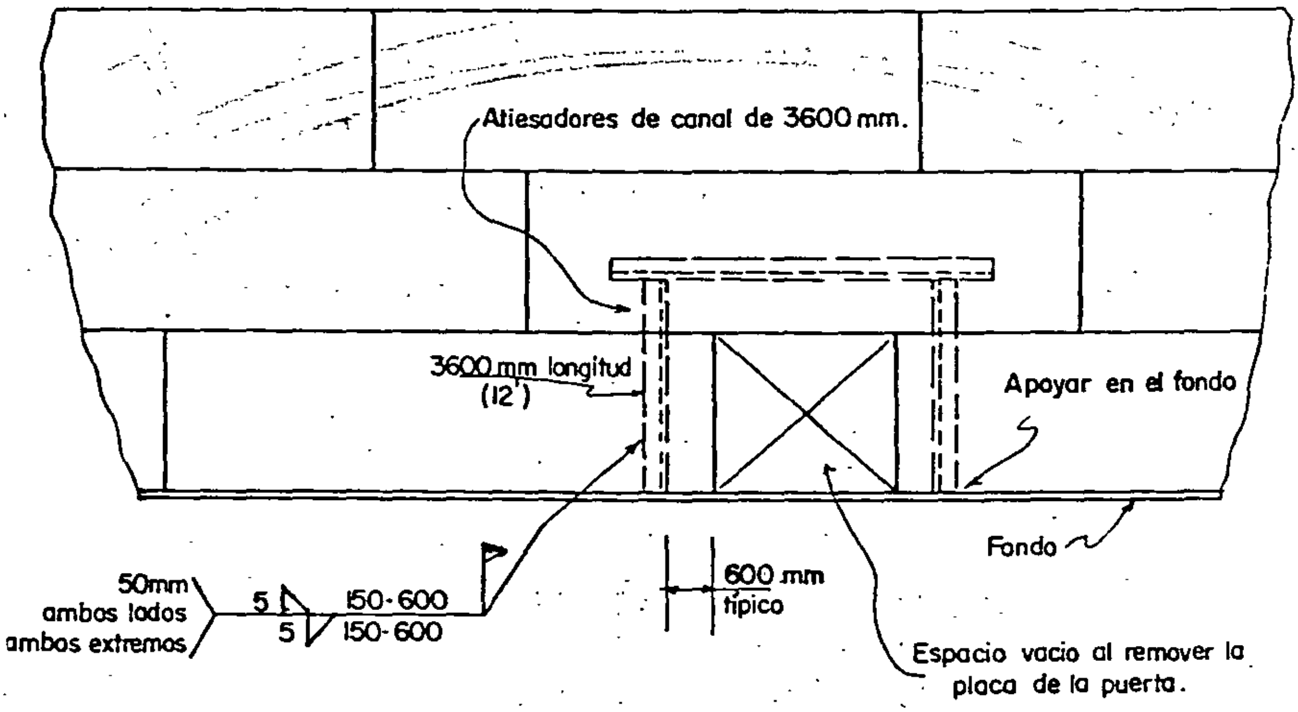


FIG. 3.3c

4. NO EMPEZAR A QUITAR CANDADOS NI PLACAS DE SUJECIÓN, HASTA QUE EL EQUIPO DE IZAJE ESTÉ ENGANCHADO.

CUANDO LAS PLACAS DE LAS PUERTAS SE COLOCAN EN FORMA DEFINITIVA EN SU LUGAR, UNA VEZ QUE SE TERMINÓ EL MONTAJE, FIJARLAS - EN AMBAS JUNTAS VERTICALES EXTREMAS, MEDIANTE PLACAS SEPARADORAS. ESTO PODRÍA ORIGINAR QUE LA PLACA DE LA PUERTA SE PANDEE HACIA AFUERA Y QUE NO QUEDE EN LÍNEA CON LAS OTRAS PLACAS DE LA ENVOLVENTE EN LA JUNTA HORIZONTAL FIG. 3.3D.

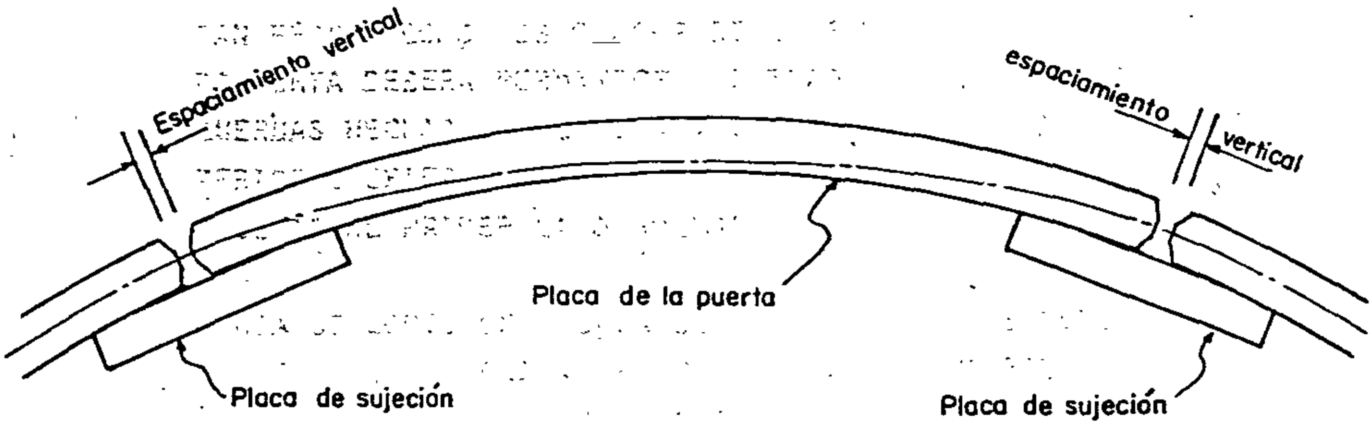


FIG. 3. 3d

CUANDO SE SUELDEN LAS JUNTAS VERTICALES, ÉSTE PANDEO DESAPARECERÁ DEBIDO A LA CONTRACCIÓN DEL METAL DE SOLDADURA. SUÉLDESE LA COSTURA HORIZONTAL DE LA PLACA DE LA PUERTA EN EL FORMA -- USUAL.

SI SE USA EL PROCEDIMIENTO DE SOLDADURA AUTOMÁTICA EN LAS COSTURAS HORIZONTALES, ARRIBA DE LA PLACA DE LA PUERTA DE LIMPIEZA, OBTENER LA ABERTURA Y EL BISEL DE LA JUNTA CON ARCO-AIRE EN LUGAR DE FORZAR LAS PLACAS CON HERRAJES PARA LOGRAR SU SEPARACIÓN.

LA SUPERVISIÓN DEBERÁ ESTAR SIEMPRE PENDIENTE DE PROTEGER AL TRABAJADOR DE OBJETOS QUE PUEDAN CAER CERCA, COLOCANDO TABLONES EN LAS MÉNSULAS ARRIBA DE CADA PUERTA DE LIMPIEZA Y DE -- LOS REGISTROS DE HOMBRE POR FUERA Y POR DENTRO DEL TANQUE.

: 4 UNIÓN Y SOLDEO DE JUNTAS VERTICALES.

EL USO DE HERRAJES EN LAS JUNTAS VERTICALES PUEDE INICIARSE TAN PRONTO COMO DOS PLACAS DE LA ENVOLVENTE SON MONTADAS. CADA JUNTA DEBERÁ PERMANECER CENTRADA SOBRE LA MARCA DE LAS CUERDAS HECHAS EN EL FONDO. LOS HERRAJES PUEDEN COLOCARSE INTERIOR O EXTERIORMENTE PERO SIEMPRE SE COLOCARÁN EN EL LADO OPUESTO AL PRIMER LADO SOLDADO.

NUNCA SE CORTE UNA PLACA DE ENVOLVENTE O SE SUELDE UNA ABERTURA DE RAÍZ MUY ANCHA SIN CONSEGUIR LA AUTORIZACIÓN DE LA SUPTCIA. LOCAL DE CONSTRUCCIÓN.

3. AJUSTE DE JUNTAS VERTICALES: HÁGANSE LOS AJUSTES Y UNIÓN DE LAS JUNTAS VERTICALES SIGUIENDO EL ORDEN INDICADO A CONTINUACIÓN:

1. EMPAREJAR LAS PLACAS EN EL EXTREMO SUPERIOR DE LA JUNTA PARA QUE QUEDEN AL RAS.

2. REVISAR EL EXTREMO INFERIOR DE LAS JUNTAS. SI LOS ANCHOS DE LAS PLACAS VARÍAN EN MÁS DE 3 MM, INVESTIGAR SI HAY ERROR DE FABRICACIÓN ANTES DE FIJAR LA JUNTA. MEDIR EL ANCHO DE AMBAS PLACAS Y NOTIFICAR EL RESULTADO A LA SUPTCIA. LOCAL DE CONSTRUCCIÓN. DEPENDIENDO DE LA LOCALIZACIÓN DEL ERROR, O BIEN, FIJAR LA JUNTA AL RAS EN EL EXTREMO SUPERIOR O INFERIOR, O DIVIDIR EL ERROR. ENTONCES AUMENTAR EL LADO CONVENIENTE DE LA PLACA CON SOLDADURA. CUALQUIER OPERACIÓN DE AJUSTE EN LA PARTE SUPERIOR SE REQUIERE QUE SEA HECHA ANTES QUE LA MÁQUINA AUTOMÁTICA CRUCE EL DESNIVEL.

3. AJUSTAR Y AMARRAR LA JUNTA EMPEZANDO DESDE ARRIBA HASTA - LLEGAR A LA PARTE INFERIOR. INSTALAR SEPARADORES DE LÁMINA Y PUNZONES PARA ASEGURARSE QUE LA ABERTURA DE LA RAÍZ EN LOS BISELES ES LA CORRECTA.

MIENTRAS SE AJUSTA UNA JUNTA, USAR UNA PLOMADA PARA DETERMINAR SI ESTÁ VERTICAL. CUANDO LOS EXTREMOS DE LAS PLACAS ESTÁN MAL FABRICADAS Y ELLAS SON FIJADAS ESTRICTAMENTE A LA SEPARACIÓN APROPIADA PUEDE RESULTAR UNA DE LAS DOS CONDICIONES SIGUIENTES:

A. SI EL EXTREMO FUÉ CORTADO RECTO PERO EN ÁNGULO, LA PLACA SE INCLINA YA SEA HACIA ADENTRO O HACIA AFUERA DE-- PENDIENDO DE LA DIRECCIÓN DEL ERROR. LA PLOMADA DETECTA ÉSTO RÁPIDAMENTE.

B. SI EL EXTREMO DE LA PLACA ES CORTADO CURVÁNDOLO, TOMARÁ LA FORMA DE BARRIL YA SEA, HACIA ADENTRO O HACIA -- AFUERA. NUEVAMENTE USANDO LA PLOMADA COMO REFERENCIA - VERTICAL SE MOSTRARÁ ÉSTA CONDICIÓN.

ES IMPORTANTE QUE LA PLACA ESTÉ DERECHA Y A PLOMO DESPUÉS DE AJUSTADA Y FIJADA CON SUS CORRESPONDIENTES HERRAJES, - LO CUAL SIGNIFICA QUE LA SEPARACIÓN DE LA JUNTA PUEDE VARIAR Y QUE EL BORDE DE LA PLACA DEBE SER AUMENTADO ANTES DE SOLDAR LA JUNTA.

LAS PLACAS CON JUNTAS VERTICALES RECTAS (SIN BISEL) SE -- MONTARÁN SIN SEPARADORES INTERMEDIOS EN LAS MISMAS Y PARA FIJARLAS EN SUS RESPECTIVAS POSICIONES DE TAL MODO QUE NO PUEDAN DESVIARSE, DEBERÁ SOLDARSELE UNA PLACA DE SUJECIÓN -

SOBRE LA JUNTA HORIZONTAL A AMBOS LADOS DE LA VERTICAL Y A 900 MM. DE LA MISMA.

SUJETAR LAS DOS PLACAS DE UN ANILLO CON LAS MENCIONADAS - EN EL PÁRRAFO ANTERIOR PARA EVITAR CUALQUIER DESVIACIÓN, HACE QUE DICHAS PLACAS SEAN FORZADAS HACIA AFUERA DE MODO QUE SOBRESALEN HORIZONTALMENTE DE LAS PLACAS DEL ANILLO - INFERIOR. AL SOLDAR LA JUNTA VERTICAL, LAS PLACAS REGRE-- SAN A SU POSICIÓN ORIGINAL SIN TENER PARTES PLANAS EN UNO U OTRO LADO DE LA JUNTA.

USANDO CANDADOS CRUZANDO LAS JUNTAS VERTICALES, SE ABREN LAS MISMAS HASTA ASEGURAR LAS SEPARACIÓN ADECUADA.

4. REVISAR EL AJUSTE DE LAS JUNTAS VERTICALES CON UNA CERCHA DE MADERA DE UNA LONGITUD MÍNIMA DE 900 MM, CON UN LADO - CURVADO AL RADIO DEL TANQUE Y UNA MUESCA CIRCULAR EN EL CENTRO PARA LIBRAR EL CORDÓN DE SOLDADURA. LA CERCHA PUE- DE USARSE PARA VERIFICAR EL AJUSTE ASÍ COMO PARA REVISAR LA REDONDEZ DEL TANQUE DURANTE EL SOLDEO.

   CUANDO LAS VERTICALES SE ESTÁN AJUSTANDO, SOLDAR PLACAS - DE SUJECIÓN SOBRE LAS JUNTAS A INTÉRVALOS DE 600 MM. SUÉL DESE UNICAMENTE UN LADO DE LAS PLACAS DE SUJECIÓN A LA -- ENVOLVENTE. ESTAS PLACAS DEBERÁN ESTAR INCLINADAS LIGERA- MENTE HACIA ABAJO PARA EVITAR SOCAVADOS (VÉASE FIGURA - - 3.4.1A).

5. VÉANSE LAS FIGURAS 3.4.1B Y 3.4.1.C PARA EL USO DEL EQUI- PO RIGIDIZANTE EN EL AJUSTE DE LAS JUNTAS.

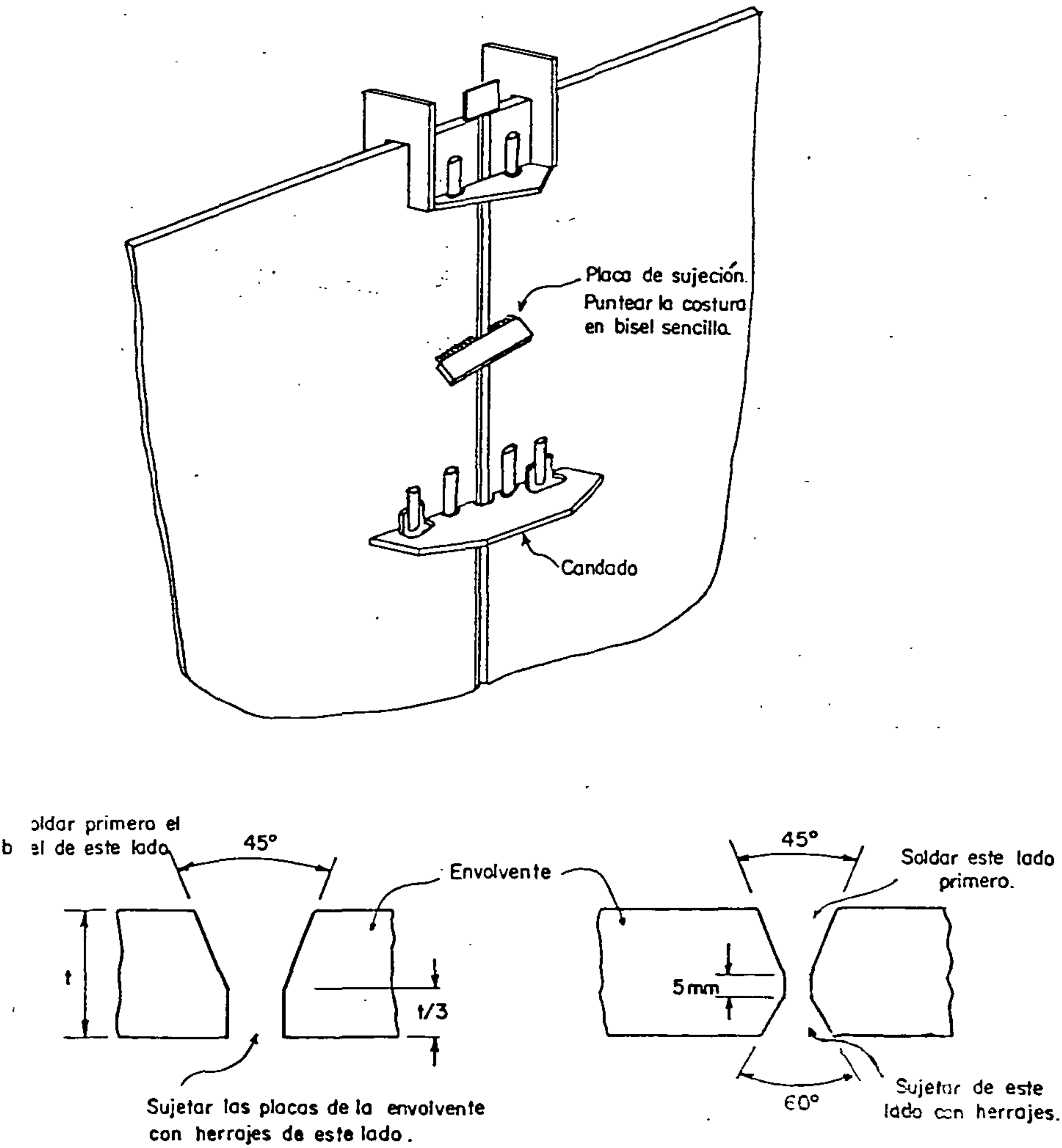
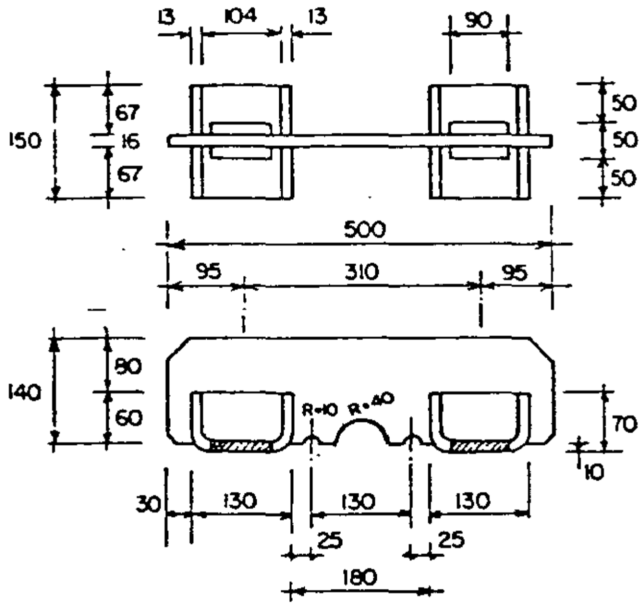
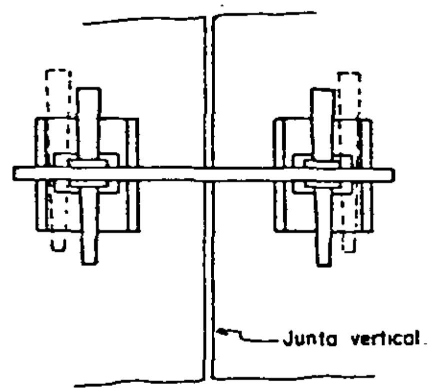
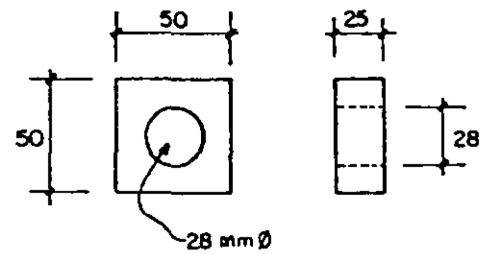

Placa de sujeción.
Puntear la costura
en bisel sencilla.
Candado
oldar primero el
b el de este lado
45°
Envolvente
t
t/3
Sujetar las placas de la envolvente
con herrajes de este lado.
45°
Soldar este lado
primero.
5mm
60°
Sujetar de este
lado con herrajes.

FIG. 3. 4. Ia



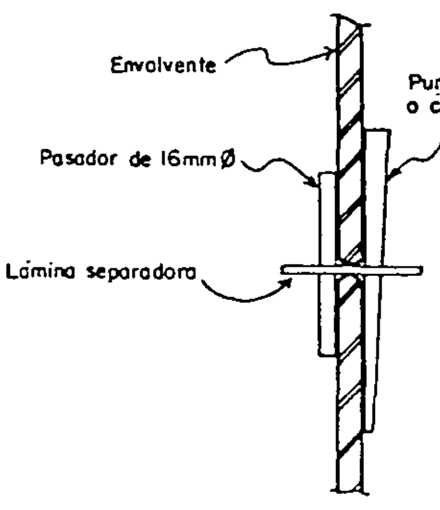
DETALLE DEL CANDADO

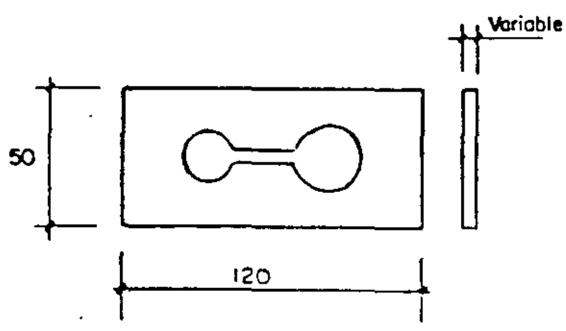




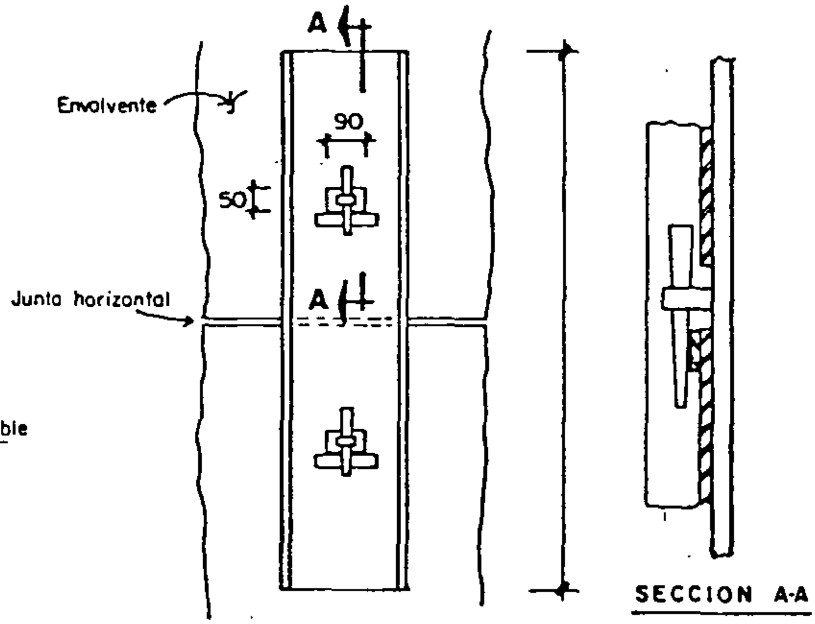
Acotaciones en mm.

DETALLE DE TUERCA LISA





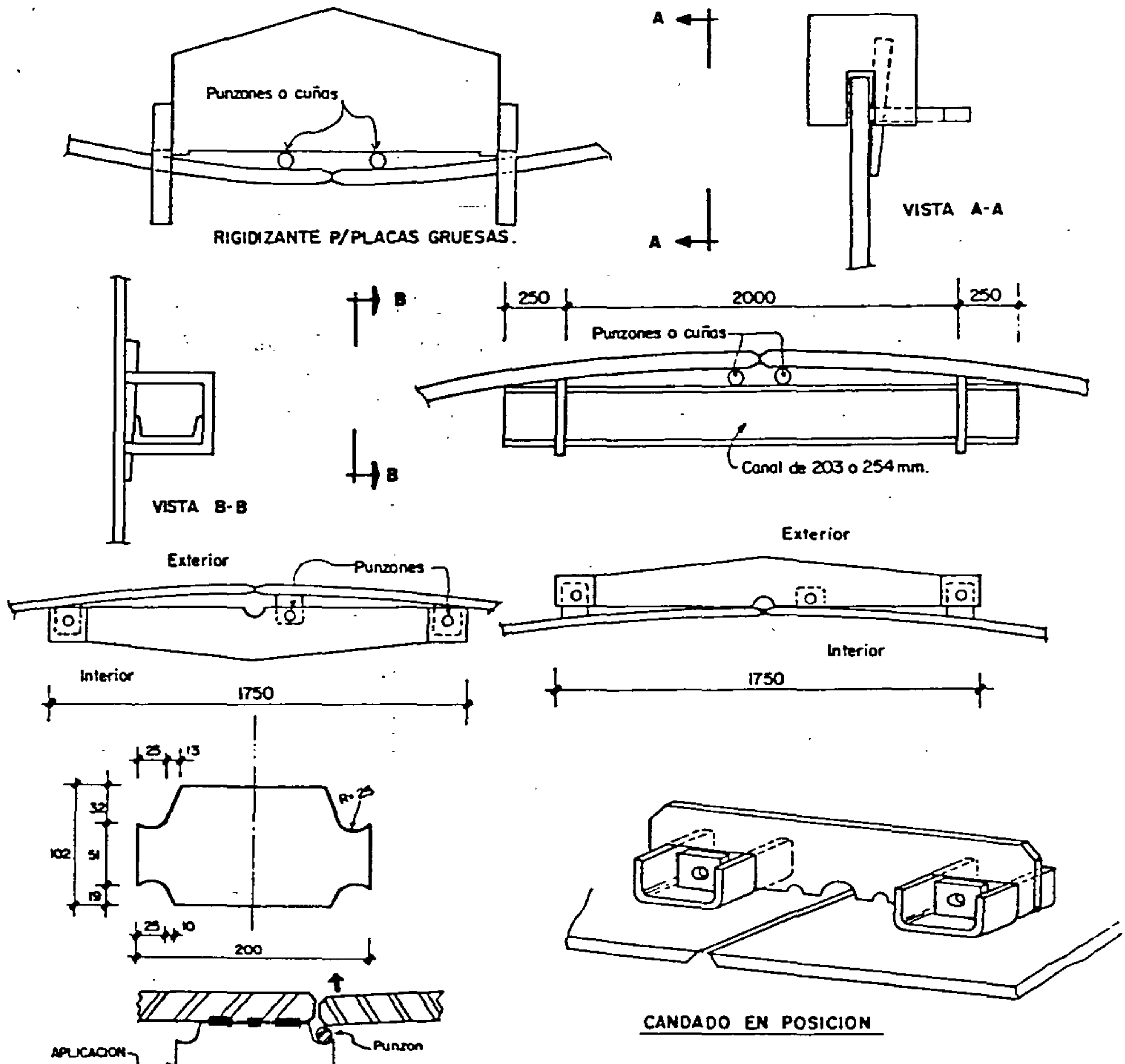
SEPARADOR PARA JUNTAS HORIZONTALES



SECCION A-A

CANDADO DE CANAL
PARA JUNTAS HORIZONTALES

FIG. 3.4.1b



PLACA DE AJUSTE TIPO 2 espesor min. 19 mm.

NOTAS:

1.- Dejar los rigidizantes en su lugar hasta que sea completada la soldadura.

2.- No se permite martillar para remover distorsiones en material de placas de 10 mm o mas de espesor.

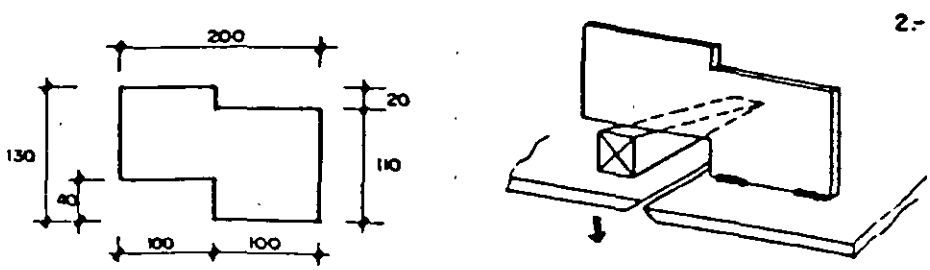


FIG. 3. 4. Ic

3.4.2 SOLDEO DE LAS JUNTAS VERTICALES.

SOLDAR LAS VERTICALES DE ACUERDO CON EL PROCEDIMIENTO DE SOLDADURA INDICADO (VER SECCIÓN "A") ASÍ COMO CON EL ELECTRODO - SELECCIONADO TANTO PARA SOLDADURA MANUAL COMO PARA LA AUTOMÁTICA. PRIMERO, SOLDAR COMPLETO EL LADO DE LA JUNTA QUE NO TIENE HERRAJES SOBRE ELLA. SOLDANDO EL OTRO LADO PRIMERO Y SALTANDO SOBRE LOS HERRAJES, PUEDE ORIGINAR GRIETAS Y FUSIÓN INCOMPLETA CUANDO SE SUELDAN LAS ÁREAS OMITIDAS. LOS CANDADOS Y LOS DEMÁS HERRAJES PUEDEN SER REMOVIDOS DESPUÉS QUE SE HA SOLDADO COMPLETAMENTE EL LADO LIBRE. PUEDE DEJARSE SI ES NECESARIO EL RIGIDIZANTE EXTREMO PARA MANTENER UNA CURVATURA CORRECTA. SI ÉSTA NO SE ADQUIERE EN LA VERTICAL CUANDO SE HA TERMINADO EL SOLDEO, DEBERÁ CORREGIRSE LA JUNTA. UNA MODERADA CANTIDAD DE MARTILLEO PUEDE DAR LA FORMA, PERO NO MARTILLAR EN - PLACAS DE 10 MM. O MÁS DE ESPESOR. VACÍAR LA SOLDADURA CON -- ARCO-AIRE Y RESOLDAR, NO CORREGIR SOLAMENTE EL EXTREMO. LA VERTICAL ENTERA DEBE ESTAR CORRECTA.

3.5 SOLDADURA EN LA JUNTA CIRCUNFERENCIAL FONDO-ENVOLVENTE.

ESTA SOLDADURA PUEDE SER TRABAJADA EN EL MOMENTO QUE SE QUIERA DESPUÉS QUE EL PRIMER ANILLO DE LA ENVOLVENTE HA SIDO MONTADO Y TODAS LAS JUNTAS VERTICALES AJUSTADAS Y ENSAMBLADAS -- CON SUS HERRAJES COMPLETOS. EL AJUSTE Y EL SOLDEO DE LA JUNTA PUEDE INICIARSE ANTES QUE TODAS LAS VERTICALES SEAN SOLDADAS - PERO NO HACER NINGUNA OPERACIÓN BAJO UNA VERTICAL QUE NO HA - SIDO COMPLETAMENTE SOLDADA. PARAR A UN METRO APROXIMADAMENTE TE DE LA VERTICAL NO SOLDADA. EL PROCEDIMIENTO DESCRITO ES EL USADO POR CBI. SIN EMBARGO PARA EVITAR PROBLEMAS DE CONTRACCIONES MAYORES, ES ACONSEJABLE SOLDAR LA JUNTA FONDO/ENVOLVENTE HASTA COMPLETAR LA SOLDADURA DE 3ER. ANILLO DE LA ENVOLVENTE (VÉASE LA FIG. 3.0).

LA SOLDADURA DE UN LADO DE LA JUNTA FONDO-ENVOLVENTE DEBERÁ HACERSE ANTES QUE LAS PLACAS IRREGULARES SEAN SOLDADAS UNA A LA OTRA. SOLDAR UN LADO PRIMERO Y HACER LA PRUEBA CON LÍQUIDOS PENETRANTES. SOLDAR EL OTRO LADO CUALQUIER TIEMPO DESPUÉS.

POR LA DIFERENCIA DE ESPESORES ENTRE LAS PLACAS DE LA ENVOLVENTE Y LAS ANULARES O IRREGULARES, ES CONVENIENTE PRECALENTAR LA JUNTA ANTES DE SOLDAR.

SI SE USA EQUIPO AUTOMÁTICO DE SOLDAR, AMBOS LADOS DEBEN SOLDARSE SIMULTÁNEAMENTE Y RADIOGRAFIAR LA SOLDADURA.

EN FONDOS CON PLACAS IRREGULARES PERIMETRALES, DEJAR ALGUNAS PLACAS SIN SOLDAR EN LAS ZONAS BAYONETEADAS PARA FINES DE DRENAJE. NO FORZAR LOS PUNZONES O CUÑAS BAJO LA ENVOLVENTE PORQUE ÉSTO PUEDE FACILMENTE DESNIVELARLA Y CREAR PROBLEMAS DE PANDEO. NO HACER MEDIOS AGUJEROS U OTROS CORTES EN LA ENVOLVENTE PARA DRENAR O POR CUALQUIER OTRA RAZÓN. ESTOS AGUJEROS CREAN CONCENTRACIÓN DE ESFUERZOS QUE PUEDEN CAUSAR FALLAS.

3.6 MONTAJE DEL SEGUNDO Y DEMÁS ANILLOS DE LA ENVOLVENTE.

MONTAR LOS ANILLOS SUPERIORES CON EL EQUIPO DE LEVANTAMIENTO DISPONIBLE (VER FIG. 3.6A).

EN TANQUES CON DIÁMETRO DE 15.00 Ó MÁS METROS (50' Ó MÁS) LAS PLACAS DE LA ENVOLVENTE SON EMBARCADAS EN LOS TALLERES CON PUNTOS MARCADOS EN LOS TERCIOS DE SU LONGITUD EN EL LADO SUPERIOR Y POR EL INTERIOR. MONTAR LAS PLACAS DE MODO QUE AMBOS EXTREMOS COINCIDAN CON LOS PUNTOS MARCADOS EN EL PRIMER TERCIO

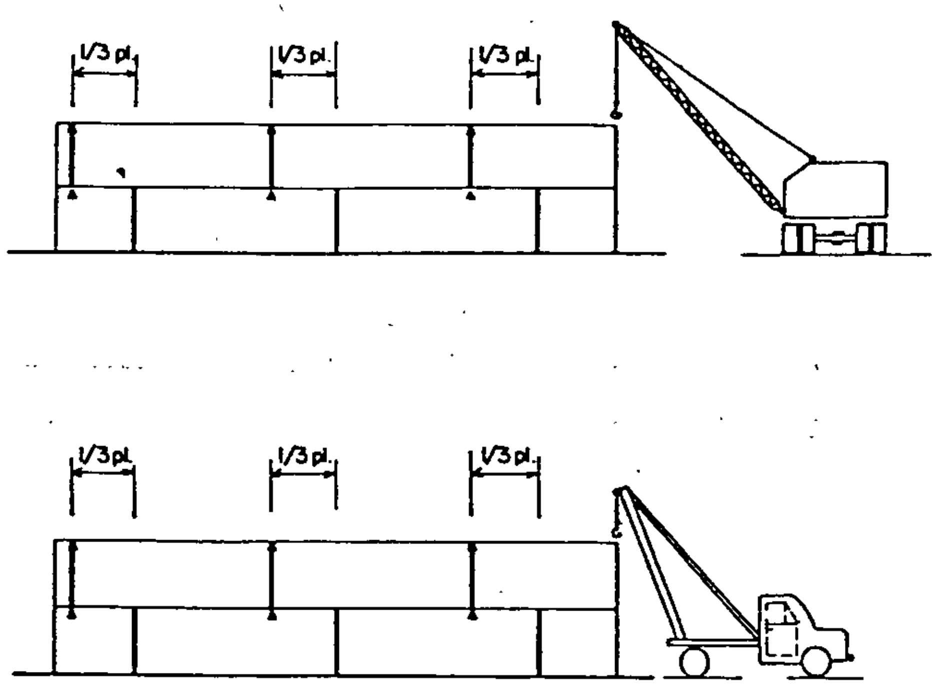


FIG. 3. 6a

DE LAS DOS PLACAS INFERIORES ADYACENTES. (VÉASE FIG. 3.6.A).

AMARRAR CADA PLACA AL ANILLO INFERIOR CON CANALES RIGIDIZANTES Y SEPARADORES COMO SE MUESTRA EN LA FIG. 3.6B.

LOS SEPARADORES SE USARÁN EN LA JUNTA HORIZONTAL AÚN CUANDO NO HAYA ABERTURA DE LA RAÍZ. ESTO ELIMINA LA NECESIDAD DE EMPLEAR LAS BARRAS EN U ANTIGUAMENTE USADAS PARA AJUSTAR LA COSTURA CIRCUNFERENCIAL. LOS SEPARADORES DEBERÁN ESPACIARSE ALREDEDOR DE 1.20 M. (4'). SIEMPRE ASEGURAR EL BORDE EXTREMO

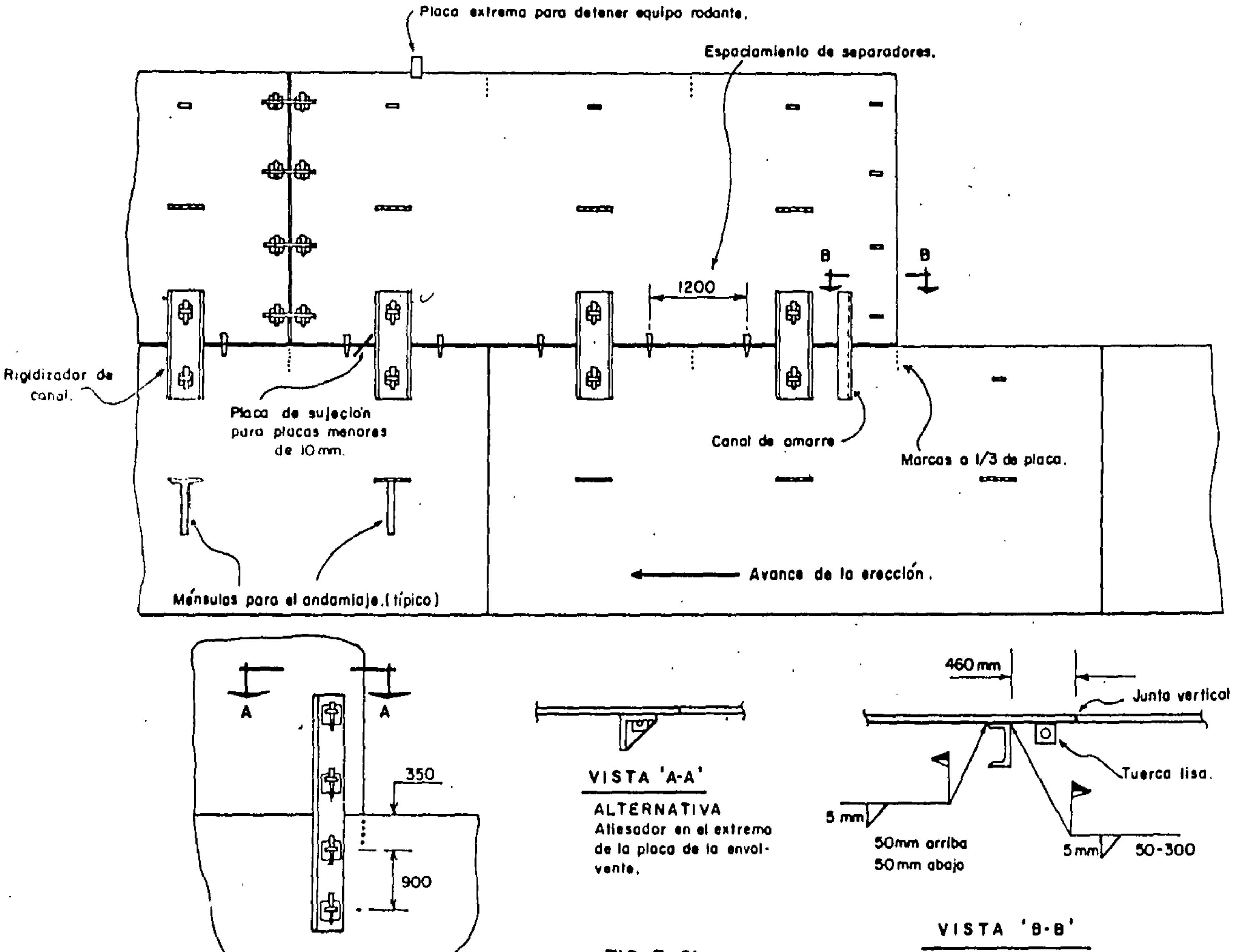


FIG. 3.6b

PEMEX

S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION

TANQUES CILINDRICOS VERTICALES
TECHO FLOTANTE

SECCION 3.0 ERECCION DE LA ENVOLVENTE

HECHO POR : Ing. I J L.
APROBADO POR : Ing. J. H B.

FECHA IV-86

HOJA 26 DE 39

MANUAL DE MONTAJE Nº 1

LA LONGITUD DE UNA PLACA (VER FIG. 3.6) NO SOLDAR LA JUNTA HORIZONTAL HASTA QUE TODAS LAS VERTICALES ARRIBA Y ABAJO DE LA MISMA HAN SIDO PREVIAMENTE SOLDADAS EN SU TOTALIDAD. SI NO SE SIGUE ESTA REGLA, AL CERRAR EL ANILLO PUEDE FALTAR O SOBRAR - PLACA DEBIDO A LA CONTRACCIÓN DEL MATERIAL POR EL SOLDEO VERTICAL.

EL MÉTODO DE ERECCIÓN DEL PUNTO A UN TERCIO ES ÚTIL SI SE USA CORRECTAMENTE . ESTO ES ESPECIALMENTE CIERTO CUANDO LA JUNTA HORIZONTAL ES SOLDADA AUTOMÁTICAMENTE. EN TANQUES SOLDADOS -- CON ÉSTE PROCEDIMIENTO, EL SOLDEO DE LA JUNTA, A MENUDO SE -- INICIA ANTES QUE TODAS LAS VERTICALES SEAN SOLDADAS Y ALGUNAS VECES ANTES QUE LA ÚLTIMA PLACA SEA MONTADA.

SIN EMBARGO, NO SUJETAR LA JUNTA HORIZONTAL SI SE PASA POR -- CUALQUIER VERTICAL QUE NO HAYA SIDO COMPLETAMENTE SOLDADA. LAS JUNTAS VERTICALES DEBEN TENER LIBERTAD PARA CONTRAERSE CUANDO SE ESTÁN SOLDANDO Y NO DEBEN ESTAR FRENADAS POR LAS JUNTAS HORIZONTALES PUNTEADAS O SOLDADAS.

CUANDO SE DISEÑA UNA ENVOLVENTE CUYOS ANILLOS ESTÁN FORMADOS POR UN NÚMERO DETERMINADO DE PLACAS EXACTAMENTE DE IGUAL LONGITUD, SI LAS PLACAS DE CADA ANILLO NO SON MONTADAS Y AJUSTADAS EN SU POSICIÓN CORRECTA, PODRÍA HABER DIFICULTADES EN MONTAR Y AJUSTAR LA ÚLTIMA PLACA, PUÉS LA LONGITUD DEL CLARO DONDE DEBERÍA ALOJARSE DICHA PLACA, PODRÍA NO CORRESPONDER A LA LONGITUD DE LA PLACA. CON LA JUNTA HORIZONTAL YA PARCIALMENTE SOLDADA, LLEGARÍA A SER MUY DIFICULTOSO DISTRIBUIR EL EXCESO DE PLACA EN LA ENVOLVENTE YA MONTADA. EN MÉXICO SE ACOSTUMBRA DISEÑAR CON CIERTO NÚMERO DE PLACAS IGUALES Y UNA ÚLTIMA PLACA, DE MUCHO MENOS LONGITUD, LLAMADA "PLACA DE AJUSTE", LA --

DE LA PRIMERA PLACA MONTADA, CON UNO DE LOS MÉTODOS ILUSTRADOS EN LA SECCIÓN A-A DE LA FIG. 3.6B. USAR CUANDO MENOS TRES (3) CANALES RIGIDIZANTES POR PLACA. SI EL MONTAJE DE UN ANILLO SE INTERRUMPE POR CUALQUIER RAZÓN (ACABARSE EL MATERIAL DE PLACAS, LA HORA DE LA COMIDA, ETC.) ASEGURAR EL BORDE DE ATAQUE DE LA ÚLTIMA PLACA MONTADA COMO SE INDICA EN LA SECCIÓN A-A DE LA FIG. 3.6B. EVITAR DEJAR UN ANILLO INCOMPLETO DURANTE LA NOCHE, PERO CUANDO ES NECESARIO, VÉASE EL PÁRRAFO 3.9 PARA INSTRUCCIONES SOBRE EL USO DE RETENIDAS ADICIONALES.

LAS PLACAS DE MENOS DE 6 MM. (1/4") DE ESPESOR PRESENTAN PROBLEMAS ESPECIALES. DEBIDO A QUE ÉSTAS GENERALMENTE NO SON ROLADAS, DEBERÁ USARSE UN TAMAÑO APROPIADO DE SEPARADORES PARA CURVAR LAS PLACAS Y SUJETARLAS PARA EVITAR SE LLEGUEN A CAER. SOLDAR UNA PLACA DE SUJECIÓN CRUZANDO LA JUNTA HORIZONTAL, MÁS O MENOS A 1.00 M. (3') DEL LADO DEL ATAQUE DE CADA PLACA DE MENOS DE 10 MM. (3/8") DE ESPESOR, CUANDO SE ESTÉN MONTANDO.

EVITAR AGRUPAR EQUIPO RODANTE (MÁQUINAS DE SOLDAR AUTOMÁTICAS) EN PLACAS DELGADAS Y EN EL EXTREMO DE PLACAS DE ANILLOS INCOMPLETOS. ESPECIALMENTE CUANDO SE USA ANDAMIAJE EXTERIOR, NO SE PERMITAN MANGUERAS PARA AIRE NI CABLES PARA LAS MÁQUINAS DE SOLDAR COLGADOS ENTRE EL EQUIPO RODANTE Y EL CENTRO DEL TANQUE. SUSPENDER ÉSTOS ADITAMENTOS VERTICALMENTE Y ADOSARLOS A LA PARED DE LA ENVOLVENTE.

AJUSTAR Y SOLDAR LAS JUNTAS VERTICALES COMO SE INDICA EN EL PÁRRAFO 3.4.

3.7 AJUSTE Y SOLDEO DE JUNTAS HORIZONTALES (CIRCUNFERENCIALES).

SI NO SE USA EL SISTEMA DE ERECCIÓN DEL PUNTO A UN TERCIO DE -

CUAL SE ENVÍA UN POCO MÁS LARGA Y SE CORTA Y ADAPTA EN EL CAMPO.

CON EL PUNTO MARCADO A UN TERCIO, CADA PLACA PUEDE SER COLOCADA EN SU UBICACIÓN CORRECTA. CADA PLACA ADICIONAL QUE SE MONTA DEBERÍA TENER SU EXTREMO CORRECTO, COINCIDIENDO SOBRE LA MARCA AL TERCIO. PODRÍAN MONTARSE JUNTAS PLACAS LARGAS Y CORTAS REGRESANDO A LOS PUNTOS AL TERCIO DE LA LONGITUD.

3.7.1 PROBLEMAS AL AJUSTAR JUNTAS HORIZONTALES.

AL AJUSTAR LAS JUNTAS HORIZONTALES SE PRESENTAN DOS PROBLEMAS: ALINEAMIENTO DE LAS PLACAS Y VARIACION DE LA ABERTURA DE LA RAIZ.

-.7.1.1 ANILLOS LARGOS O CORTOS.

CON LA TOLERANCIA ACEPTABLE PARA LA LONGITUD DE LAS PLACAS, PUEDE SUCEDER QUE EL DESARROLLO DEL ANILLO RESULTE LIGERAMENTE MUY LARGO O MÁS CORTO. CUANDO SE TRABAJA CON HERRAJES EN CUALQUIER TIPO DE JUNTA, EL AJUSTADOR DEBERÁ ESTAR CONSIENTE DE COMO SUS HERRAJES ESTÁN AFECTANDO OTRA PARTE DE LA ESTRUCTURA. MIENTRAS SE ESTÁ AJUSTANDO LA JUNTA HORIZONTAL, DEBERÍA OBSERVAR UNA Y MEDIA Ó DOS PLACAS MÁS ADELANTE Y TOMAR LAS MEDIDAS PERTINENTES SEGÚN EL CASO.

1. SI HAY UNA PLACA CORTA ADELANTE, EL AJUSTADOR PUEDE AFLOJAR ALGO LAS PLACAS DE ADELANTE Y HACER EL AJUSTE.

2. O, SI EL AJUSTADOR ESTÁ TRABAJANDO EN UNA PLACA CORTA, ÉL PUEDE CONSEGUIR EL AFLOJAMIENTO DE UNA PLACA LARGA DE ADELANTE.

3. SI UN ANILLO RESULTA MÁS PEQUEÑO, EL AJUSTADOR DEBE MOVER EL EJE DE LAS PLACAS DEL MISMO HACIA ADENTRO.

4. SI UN ANILLO RESULTA MÁS GRANDE, SE DEBE MOVER EL EJE DE LAS PLACAS HACIA AFUERA.

LOS CÓDIGOS CUBREN LA TOLERANCIA ADMISIBLE EN EL DESALINEAMIENTO DE LA JUNTA HORIZONTAL. LA TOLERANCIA POR DESALINEAMIENTO SE REFIERE A LA CANTIDAD QUE LA PLACA SUPERIOR SOBRESALE HORIZONTALMENTE DE LA INFERIOR YA SEA HACIA ADENTRO O HACIA AFUERA. VÉASE API 650 STD SECCIÓN 5.2.3.

3.7.1.2 VARIACIONES EN LA ABERTURA DE LA RAIZ DE LA SOLDADURA.

LA VARIACIÓN DE LA ABERTURA DE LA RAÍZ EN LA JUNTA HORIZONTAL PUEDE SER ORIGINADA POR UNA ENVOLVENTE FUERA DE NIVEL Y/O MALA FABRICACIÓN.

NO USAR CANDADOS PARA JALAR LA ABERTURA DE LA JUNTA HORIZONTAL. ESTO PODRÍA ORIGINAR DOBLECES EN EL ANILLO SUPERIOR MÁS DELGADO. SI LA ENVOLVENTE ABAJO DE LA JUNTA ESTÉ FUERA DE PLOMO, DEBERÁ RE-NIVELARSE.

SEPARACIONES NO UNIFORMES EN LA JUNTA HORIZONTAL, PUEDE SER EL RESULTADO DE UNA MALA FABRICACIÓN. EN ESTOS CASOS LA ORILLA DE LA PLACA DEBERÁ RELLENARSE CON SOLDADURA PARA PRODUCIR UNA ABERTURA UNIFORME. REPORTAR A LA SUPTCIA. LOCAL DE CONSTRUCCIÓN SIEMPRE QUE OCURRA ÉSTA SITUACIÓN PARA QUE LA FABRICACIÓN PUEDE SER CORREGIDA. PUESTO QUE LA CONTRACCIÓN ES IGUAL EN CADA MITAD DE LA ABERTURA, HABRÁ PROBLEMAS SI NO ES RELLENADA APROPIADAMENTE. LA CONTRACCIÓN EN UNA ABERTURA IRREGULAR JALA EL ANILLO PONIENDOLO FUERA DE NIVEL CON EL RESULTADO DE ZONAS PLANAS Y/O ONDULACIONES O DOBLECES.

3.7.2 SOLDEO DE LAS COSTURAS HORIZONTALES. SOLDAR LA JUNTA DE ACUERDO CON EL PROCEDIMIENTO INDICADO EN LAS HOJAS RESPECTIVAS Y LAS TÉCNICAS DE SOLDEO MANUAL O AUTOMÁTICA (VÉASE LA SECCIÓN A.)

3.8 MONTAJE DE MIEMBROS ESTRUCTURALES EN LA ENVOLVENTE.

TERMINADA LA ERECCIÓN Y LA SOLDADURA DEL ÚLTIMO ANILLO SE PROCEDERÁ A MONTAR LOS MIEMBROS ESTRUCTURALES COMO ÁNGULOS DE CORONAMIENTO, TRABES DE REFUERZO CONTRA EL VIENTO Y ÁNGULOS ATIESADORES ADICIONALES. LA ERECCIÓN DE ÉSTOS MIEMBROS, ES UNA OPERACIÓN COMÚN Y SOLAMENTE SE DAN COMENTARIOS GENERALES.

3.8.1 ANGULOS DE CORONAMIENTO Y ATIESADORES. ANTES DE PROCEDER AL MONTAJE DE ÉSTOS ELEMENTOS DEBERÁN REVISARSE DE ACUERDO CON LOS PLANOS DE FABRICACIÓN, COMO SIGUE:

1. REVISAR LOS EXTREMOS DE CADA PIEZA PARA ASEGURARSE QUE LOS ÚLTIMOS 600 Ó 900 MM. ESTÉN ROLADOS APROPIADAMENTE.

2. SI LOS EXTREMOS NO VIENEN ROLADOS CORTAR LA PARTE RECTA; ASEGURARSE QUE HAY SUFICIENTE ÁNGULO PARA COMPLETAR EL ANILLO.

3. ANTES DE SOLDAR EL ÁNGULO DE CORONAMIENTO A LA ENVOLVENTE, PLOMEAR EL ALA VERTICAL DEL ÁNGULO.

4. ANTES DE SOLDAR CUALQUIER ATIESADOR, ASEGURARSE QUE EL ANILLO DE LA ENVOLVENTE AL CUAL SE CONECTA, ESTÁ CON LA REDONDEZ DENTRO DE LAS TOLERANCIAS MARCADAS EN LA SECCIÓN 1.0.

5. Soldar el ángulo de coronamiento siguiendo las indica-- ciones de los planos de montaje.

3.8.2 Trabes perimetrales de refuerzo contra el viento.

Antes de montar éstos miembros, revisar la redondez de la - parte superior del tanque. Usese el doble de la tolerancia por redondez del primer anillo dada en la sección 1.0. También revisar la verticalidad de la envolvente en cada junta vertical del último anillo. Véase la sección 1.0 para tolerancias permisibles. Si el tanque no está redondo o la en-- volvente no está a plomo, revisar la horizontalidad del anillo de cimentación y hacer las correcciones requeridas an-- tes de montar la trabe de refuerzo.

1. Trazar la localización de la sección correspondiente de la escalera exterior en la envolvente.

2. Trazar la localización de las ménsulas de soporte en la envolvente.

3. Montar las ménsulas.

4. Levantar las secciones de la trabe y apoyarlas en las -- ménsulas. Sujetar las juntas a tope con candados entre - una y otra sección.

5. Colocar la protección con cables en la trabe alrededor - del tanque.

6. AJUSTAR Y FIJAR TODAS LAS JUNTAS A TOPE DE LA TRABE EXCEPTO UNA. ASEGURARSE QUE ESTAS JUNTAS TENGAN LA ABERTURA -- APROPIADA A TODO SU LARGO. ESTO AYUDARÁ A MANTENER LA TRABE REDONDEADA AL RADIO DE DISEÑO. CON EL USO DE UNA CERCHA REVISAR QUE LA CURVATURA SEA LA CORRECTA.

7. SOLDAR TODAS LAS JUNTAS A TOPE DE LA TRABE EXCEPTO UNA.

8. EMPEZANDO EN LA PARTE OPUESTA A LA JUNTA NO SOLDADA, TRABAJAR EN AMBOS SENTIDOS ALREDEDOR DEL TANQUE FIJANDO LA - TRABE A LA ENVOLVENTE.

9. AJUSTAR Y SOLDAR LA JUNTA A TOPE QUE QUEDÓ PENDIENTE.

10. SOLDAR LA TRABE A LA ENVOLVENTE.

11. AJUSTAR Y SOLDAR LA TRABE A LAS MÉNSULAS.

12. REVISAR EL DIÁMETRO DEL TANQUE NUEVAMENTE.

3.8.3 <u>REVISIÓN DE LA REDONDEZ. ENVOLVENTE EN TANQUES ABIERTOS</u>.

A CONTINUACIÓN SE SUGIERE UN MÉTODO PARA REVISAR LA REDONDEZ - DE LA ENVOLVENTE DE TANQUES ABIERTOS EN SU PARTE SUPERIOR DONDE SE INSTALAN TECHOS FLOTANTES.

PARA REVISAR SI LA ENVOLVENTE DEL TANQUE EN SU PARTE MÁS ALTA- ESTÁ FUERA DE REDONDEZ, SE PROCEDE COMO SIGUE:

1. SELECCIONAR UNA BARRA O UN ÁNGULO COMO ESCANTILLÓN DE CUANDO MENOS 100 MM. MÁS LARGO QUE LA PARTE MÁS ANCHA DE LA -- TRABE DE REFUERZO CONTRA EL VIENTO (TRABE DE RIGIDEZ).

2. FIJAR UN ALAMBRE RESISTENTE O UNA CUERDA DE PIANO CON -- UNA PLOMADA U OTRO CONTRAPESO DE MÁS O MENOS 5 KG. EN EL EXTREMO DEL ESCANTILLÓN.

3. USANDO EL DISPOSITIVO ANTERIOR, BIEN FIJO EL ESCANTILLÓN A LA ENVOLVENTE, APOYADO EN LA TRABE DE RIGIDEZ COMO SE ILUSTRA EN LA FIGURA 3.8.3, MEDIR Y REGISTRAR LA DIMENSIÓN "A" EN PULGADAS.

4. EN LA MISMA FORMA, HACER MEDICIONES Y REGISTRARLAS EN TODAS LAS JUNTAS VERTICALES Y A LA MITAD DE TODAS LAS PLACAS DEL ÚLTIMO ANILLO.

5. CUANDO CADA MEDICIÓN ANTERIOR ES HECHA, REGISTRAR TODAS LAS DIMENSIONES "B" CORRESPONDIENTES, TAMBIÉN EN PULGADAS.

6. USESE LA SIGUIENTE EXPRESIÓN PARA DETERMINAR TOLERANCIAS: 0.01 (D+H)= TOLERANCIA DEL DIÁMETRO EN PULGADAS, DONDE D ES EL DIÁMETRO DEL TANQUE Y H SU ALTURA, AMBAS DIMENSIONES EN PIES. SI LA DIFERENCIA ENTRE LA MÁS GRANDE Y LA -- MÁS PEQUEÑA DIMENSIÓN "B" EN TODO EL PERÍMETRO DEL TANQUE ES IGUAL O MENOR QUE LA TOLERANCIA DEL DIÁMETRO CALCULADA, LA REDONDEZ DE LA ENVOLVENTE SE CONSIDERA CORRECTA PARA UN FUNCIONAMIENTO SATISFACTORIO DEL TECHO FLOTANTE.

   LA DIFERENCIA B-A DE CADA MEDICIÓN DA, EN CADA PUNTO MEDIDO LA CANTIDAD FUERA DE PLOMO DE LA ENVOLVENTE.

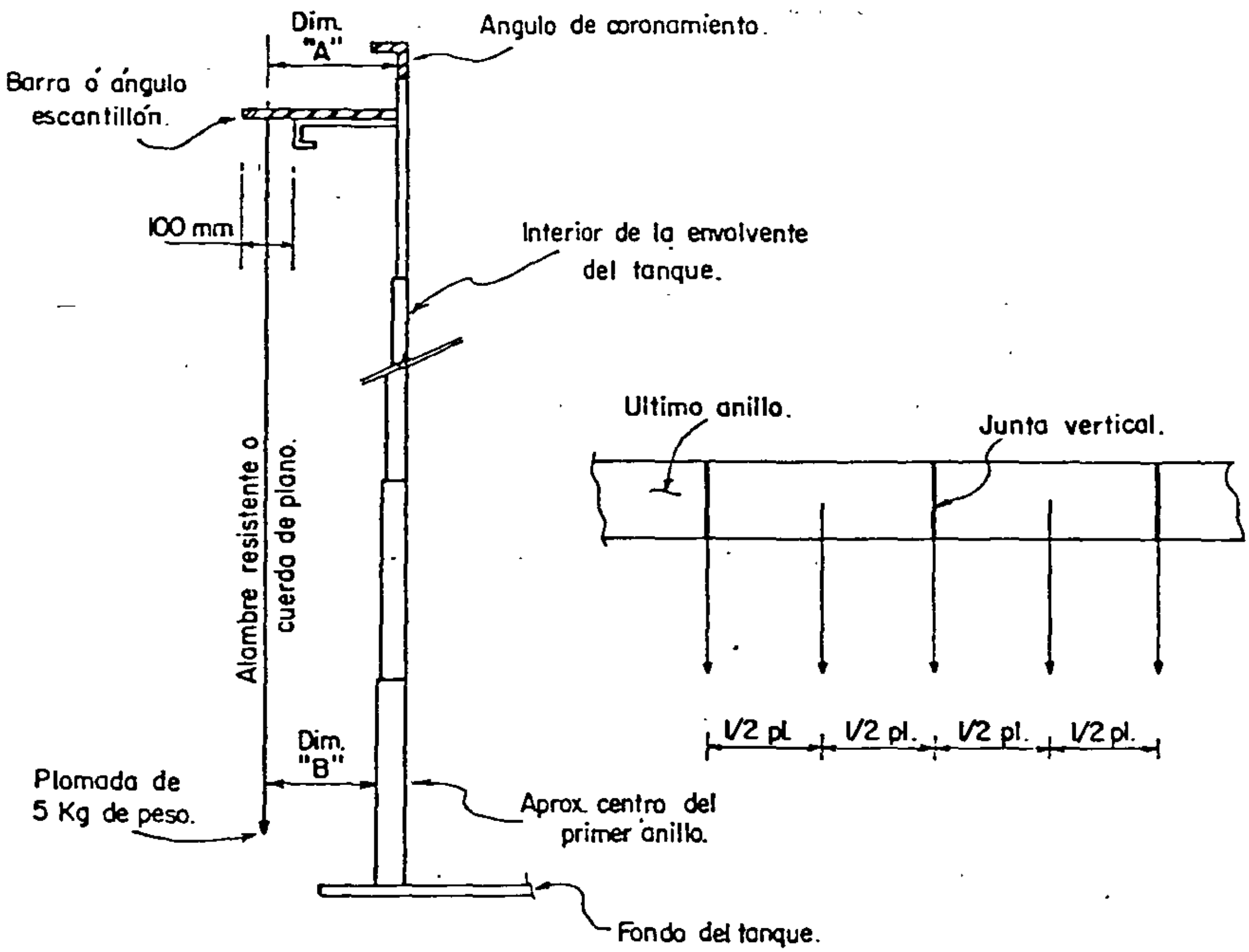


FIG. 3. 8 .3

3.9 PROTECCIÓN CONTRA EL VIENTO.

LOS PROCEDIMIENTOS PARA SUMINISTRAR PROTECCIÓN CONTRA DAÑOS ORIGINADOS POR EL VIENTO, VARÍAN CON LA LOCALIZACIÓN DEL TANQUE Y LA ÉPOCA DEL AÑO EN QUE SE HACE LA ERECCIÓN. DEBERÁ USARSE LA EXPERIENCIA LOCAL PARA DETERMINAR LA PROTECCIÓN ADICIONAL QUE DEBERÁ SER PROPORCIONADA, ADEMÁS DE LOS REQUERIMIENTOS MÍNIMOS. EL PÁRRAFO 3.9.2 ENLISTA LAS MEDIDAS ADICIONALES QUE DEBERÁN TOMARSE EN EL CASO QUE HAYA FUERTES VIENTOS. EL CRITERIO ES LA ÚNICA GUÍA DISPONIBLE PARA DETERMINAR CUANTA PROTECCIÓN ES NECESARIA. EL RESIDENTE DE LA CONTRATISTA Y EL

SUPERVISOR DE PEMEX DEBERÁN ESTAR SEGUROS QUE LA COMPAÑÍA -- TIENE Y USARÁ EL EQUIPO ADECUADO DURANTE LA ÉPOCA EN QUE SE ESPERAN FUERTES VIENTOS. LA CONTRATISTA DEBE ESTAR SUFICIENTEMENTE ENTERADA DE LA PREDICCIÓN DEL TIEMPO, DE MODO QUE -- PUEDA HACER COMPLETO USO DE TODO SU EQUIPO DISPONIBLE CUANDO SE ESPERAN VENDAVALES LOCALES.

3.9.1 REQUERIMIENTOS MÍNIMOS.

LOS SIGUIENTES SON LOS REQUERIMIENTOS MÍNIMOS QUE DEBERÁN -- PONERSE EN OPERACIÓN AL FINALIZAR CADA DÍA DE TRABAJO.

1. SE PUEDE USAR EL ANDAMIO HECHO A BASE DE MÉNSULAS Y TA-- BLONES COMO TRABE DE REFUERZO CONTRA EL VIENTO. SE PUEDE INSTALAR EN EL TERCER ANILLO, CUANDO SE HA INICIADO EL - MONTAJE DEL CUARTO. APRETAR LOS GANCHOS EN "J", PUNZONES Y TABLONES PORQUE UN ANDAMIO ES UNICAMENTE TAN RESISTENTE COMO SU PUNTO MÁS DÉBIL.

2. CUANDO SE ESTÁ MONTANDO UN ANILLO SIEMPRE ES CONVENIENTE COMPLETARLO ANTES QUE EL PERSONAL SE RETIRE DE LA OBRA - AL LLEGAR LA NOCHE, PERO ÉSTO NO SIEMPRE ES POSIBLE. - - CUANDO NO PUEDE COMPLETARSE EL MONTAJE, LOS EXTREMOS - - ABIERTOS DEL ANILLO PARCIALMENTE MONTADO, DEBERÁN CONTRAVENTEARSE CON RETENIDAS HACIA ADENTRO Y HACIA AFUERA CON CABLE DE 10 MM. (3/8") MÍNIMO DE DIÁMETRO.

3. CUANDO EN UN TANQUE SE DISEÑAN ATIESADORES PERMANENTES, ÉSTOS DEBERÁN MONTARSE Y SOLDARSE INMEDIATAMENTE DESPUÉS QUE LA JUNTA HORIZONTAL ARRIBA DE ELLOS HA SIDO SOLDADAS SI EL ATIESADOR NO INTERFIERE CON LA SOLDADURA DE LOS CORDONES HORIZONTALES, DEBERÁ MONTARSE TAN PRONTO COMO LA SOL

DADURA DE LAS JUNTAS VERTICALES HA SIDO COMPLETADA. VÉASE EL PÁRRAFO 3.8.

4. EL EQUIPO AUTOMÁTICO DE SOLDAR U OTROS EQUIPOS PESADOS -- SUSPENDIDOS DE LA ENVOLVENTE DEBERÁN SUJETARSE CON CABLES DE RETENIDA DE 10 MM. (3/8") HACIA ADENTRO Y HACIA AFUERA, DURANTE LAS NOCHES. AMARRAR CUALQUIER OTRO EQUIPO RODANTE PARA IMPEDIR QUE SE MUEVA.

5. EN TANQUES DE TECHO FLOTANTE DE 38.00 M. (125') O MÁS DE DIÁMETRO ES RECOMENDABLE USAR LA TRABE PERMANENTE DE RE-- FUERZO CONTRA EL VIENTO COMO ANDAMIAJE.

.9.2 RECOMENDACIONES ADICIONALES.

LA EXPERIENCIA LOCAL PUEDE DETERMINAR QUE PROTECCIÓN ADICIO-- NAL DEBERÁ SUMINISTRARSE A LA OBRA POR VIENTOS FUERTES OCURRIEN DO NORMALMENTE. DEPENDIENDO DE LA SEVERIDAD DE ÉSTOS VIENTOS ESPERADOS Y DATOS ESTADÍSTICOS DE LA LOCALIDAD , PODRÍAN RE-- QUERIRSE PRECAUCIONES ADICIONALES COMO LAS ENLISTADAS A CONTI NUACIÓN:

1. LA ADICIÓN DE UN SEGUNDO ANDAMIO EJERCIENDO LA FUNCIÓN DE UNA TRABE DE RIGIDEZ O LA ADICIÓN DE DOS CONTRAVIENTOS -- (MÍNIMO DE 10 MM. DE DIÁMETRO DE CABLE) POR PLACA, O AMBAS COSAS A LA VEZ.

2. DESMONTAR LA MÁQUINA AUTOMÁTICA DE SOLDAR O CUALQUIER - - OTRO EQUIPO PESADO DE LA ENVOLVENTE. CUALQUIER EQUIPO RO DANTE QUE NO SE DESMONTE, DEBERÁ SITUARSE SOBRE UNA MÉNSU LA DE ANDAMIO O SOBRE UN JUEGO DE CONTRAVIENTOS. ADEMÁS DE-

BERÁ AMARRARSE PARA IMPEDIR QUE SE DESPLACE. SI HAY VARIOS EQUIPOS, DISTRIBUIRLOS EN DISTINTAS MÉNSULAS Y NO DEJARLOS JUNTOS.

3. SOLDAR UNA TUERCA LISA EN LA ENVOLVENTE COMO TOPE ARRIBA DE CADA MÉNSULA PARA IMPEDIR QUE ÉSTA SE SALGA DE LA ABRAZADERA.

4. APRETAR ANCLAJES SI HAY FORMANDO PARTE DE LA ESTRUCTURA.

5.- DEJAR LAS PUERTAS DE LIMPIEZA, REGISTROS Y BOQUILLAS ABIERTAS, PARA IMPEDIR SE HAGA UN VACÍO EN EL INTERIOR DEL TANQUE.

3.10 LIMPIEZA DE LA ENVOLVENTE DEL TANQUE.

LA SUPERFICIE EXTERIOR E INTERIOR DE TODOS LOS TANQUES Y TODAS LAS QUE SE VAN A PINTAR SE LIMPIARÁN COMO SIGUE:

1. REMOVER LA ESCORIA Y LAS SALPICADURAS DE LAS SOLDADURAS.

2. CON CINCEL LIMPIAR LAS REBABAS DE LOS CORDONES.

3. CINCELAR, ALISAR Y PULIR ESMERILANDO DONDE SE REQUIERE - REMOVER SALIENTES PUNTIAGUDOS Y ÁSPEROS.

4. REMOVER ACUMULACIONES DE LODO, POLVO Y OTRAS SUBSTANCIAS EXTRAÑAS ANTES DE LEVANTAR Y MONTAR LAS PLACAS EN SU LUGAR.

   ADEMÁS, NO ARRASTRAR MATERIAL CON LA PRIMERA MANO DE PINTURA

APLICADA EN EL TALLER. EN EL INTERIOR DE LOS TANQUES DE TECHO FLOTANTE, TODOS LOS SALIENTES PUNTIAGUDOS DE MÁS DE 1.5 MM. -(1/16") DE ALTURA, DEBERÁN SER CINCELADOS DE MODO QUE QUEDEN-SUPERFICIES LISAS. CUALQUIER SALIENTE QUE PROYECTE FILOS O --PUNTAS DEBE SER ESMERILADO.

3.11 ANDAMIAJE.

LA CÍA. CONTRATISTA DEBERÁ PROPORCIONAR A LA SUPTCIA. LOCAL DE CONSTRUCCIÓN, A TRAVÉS DE SU RESIDENTE Y EL SUPERVISOR DE PE--MEX, EL SISTEMA DE ANDAMIAJE EXTERIOR O INTERIOR, ESCALAS Y --DEMÁS PROTECCIONES QUE SE VA A USAR PARA QUE EL PERSONAL DE --MONTAJE, SOLDADORES, INSPECTORES, ETC. TRABAJEN CON TODA LA SEGURIDAD POSIBLE.

HAY MUCHOS TIPOS DE ANDAMIOS, DESDE LOS MÁS SENCILLOS EMPLEANDO MÉNSULAS ENCAJADAS EN SOLERAS EN "U" Y TABLONES EN EL PISO-HASTA LOS MÁS SOFISTICADOS QUE SE DESLIZAN POR LA ORILLA SUPERIOR DE LOS ANILLOS DE LA ENVOLVENTE. LO IMPORTANTE ES QUE LOS QUE SE VAN A USAR SEAN SEGUROS PARA QUE EL PERSONAL LABORE CON TODA CONFIANZA.

EN ALGUNOS CASOS, LA TRABE DE REFUERZO CONTRA EL VIENTO QUE SE INSTALA DEFINITIVAMENTE EN EL EXTREMO SUPERIOR DEL ÚLTIMO ANILLO, SE ARMA Y SE MONTA PROVISIONALMENTE EN EL PRIMER ANILLO Y SE VA ELEVANDO A MEDIDA QUE AVANZA EL MONTAJE DE LOS SIGUIENTES ANILLOS, SIRVIENDO DE ANDAMIAJE EXTERIOR.

4.0 MONTAJE DEL TECHO FLOTANTE.

4.1 GENERALIDADES.

LOS TANQUES DE TECHO FLOTANTE FABRICADOS Y MONTADOS HASTA -- AHORA DE 500,000 BARRILES DE CAPACIDAD, HAN SIDO DISEÑADOS - CON LOS TECHOS A BASE DE DIAFRAGMAS SENCILLOS, PONTÓN PERIMETRAL, TUBO-SELLO Y UNA SERIE DE BOYAS REPARTIDAS SIMÉTRICA-- MENTE EN TODA LA SUPERFICIE EXTERIOR DEL DIAFRAGMA. EL FUN-- CIONAMIENTO EN CONJUNTO DEL PONTÓN Y LAS BOYAS ES, POR LO -- TANTO, LA MÁS SIMPLE, LÓGICA Y ECONÓMICA RESPUESTA AL PROBLEMA DE SUMINISTRAR FLOTACIÓN EN TANQUES DE GRAN DIÁMETRO. EL TAMAÑO, FORMA Y CANTIDAD DE BOYAS VARÍA CON LAS CONDICIONES ESPECIALES DE CADA PROYECTO Y CAPACIDAD DE LOS TANQUES. EL - VOLÚMEN DE FLOTABILIDAD PROPORCIONADO POR EL PONTÓN Y LAS BOYAS ES MÁS QUE EL ADECUADO PARA SOSTENER EL DIAFRAGMA FLOTANDO SI OCURRIESE UNA ROTURA. POR LO QUE SE REFIERE A LOS TANQUES DE 200,000 A 55,000 BARRILES, EL DISEÑO DEL TECHO ES -- MÁS SENCILLO. SOLAMENTE CONSTAN DEL TUBO-SELLO, DIAFRAGMA -- SENCILLO Y PONTÓN PERIMETRAL, CUYAS DIMENSIONES SON LAS CO-- RRECTAS PARA MANTENER FLOTANDO AL DIAFRAGMA.

4.2 SECUENCIA DE MONTAJE DEL TECHO.

EL MONTAJE DEL TECHO FLOTANTE SE PUEDE INICIAR UNA VEZ QUE - SE HAYA TERMINADO DE SOLDAR EL FONDO Y LOS TRES PRIMEROS ANILLOS DE LA ENVOLVENTE DEL TANQUE. EN TÉRMINOS GENERALES, LAS MANIOBRAS DEL MONTAJE SE LLEVAN A CABO SIGUIENDO EL ORDEN INDICADO A CONTINUACIÓN. LAS OPERACIONES DETALLADAS DE CADA PASO, SE DESCRIBEN EN LOS PÁRRAFOS QUE SIGUEN MÁS ADELANTE.

1. ENSAMBLE DEL PONTÓN Y SU MONTAJE.

2. ARMADO DE UNA OBRA PROVISIONAL DE APUNTALAMIENTO PARA APOYAR EL DIAFRAGMA.

3. ARREGLO Y TENDIDO DE LAS PLACAS DEL DIAFRAGMA DEL TECHO.

4. SECUENCIA DE SOLDEO DEL DIAFRAGMA.

5. INSTALACIÓN DE BOYAS Y DE LOS POSTES DE SOPORTE DEFINITIVOS DEL TECHO Y PONTÓN.

6. INSTALACIÓN DE ACCESORIOS COMO EL SISTEMA DE DRENAJE DEL TECHO, ESCALERAS INTERIOR Y EXTERIOR, GUÍA ANTIROTACIÓN, VÁLVULAS, ETC.

4.2.1 SUB-ENSAMBLE Y MONTAJE DEL PONTÓN.

LAS PARTES PRINCIPALES DEL PONTÓN SON: LA ENVOLVENTE EXTERIOR (1) COMPUESTA DE DOS PLACAS, LA SUPERIOR (1A) Y LA INFERIOR (1B) - EN LOS TANQUES DE 500,000 BLS. (FIG. 4.2.1A) Y EN LOS DE MENOR CAPACIDAD SOLAMENTE LA PLACA (1) DE UNA SOLA PIEZA (FIGURA 4.2.1B), LA ENVOLVENTE INTERIOR (2), EL SECTOR SUPERIOR (3) Y EL

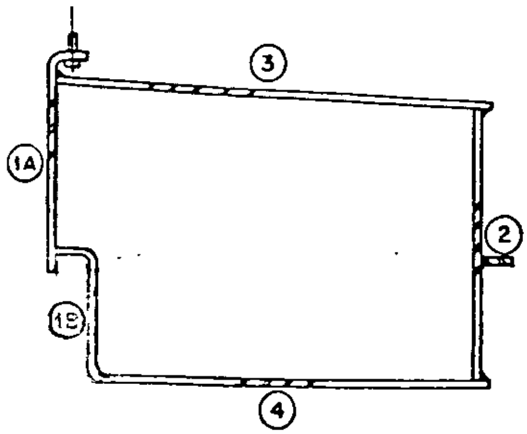


FIG. 4.2.1a

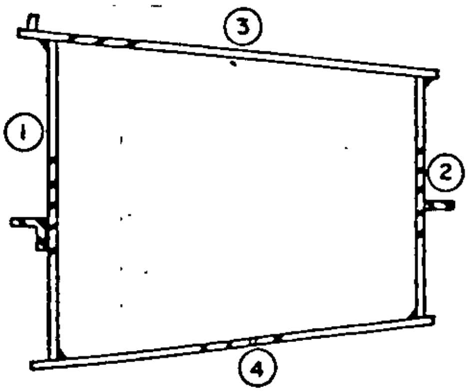


FIG. 4.2.1b

INFERIOR 4. LLEVA ADEMÁS OTROS ELEMENTOS COMO REGISTROS DE HOMBRE, PLACAS SEPARADORAS DE COMPARTIMIENTOS, SOPORTE DEL PONTÓN EN EL FONDO, ETC.

NORMALMENTE, EL PONTÓN SE FABRICA EN SECCIONES DE LARGOS MANEJABLES. ESTAS SECCIONES SE TRANSPORTAN AL CAMPO CON TODOS SUS ELEMENTOS SUELTOS PARA ENSAMBLARLOS EN LA OBRA MISMA. ESTO SE PUEDE HACER FUERA DEL TANQUE SOBRE UNA CAMA BIEN NIVELADA O, EN EL INTERIOR DEL MISMO DIRECTAMENTE SOBRE EL FONDO PERO TOMANDO EN CUENTA SU PENDIENTE, NIVELANDO EL SECTOR INFERIOR 4 CON CALZAS. SÍGASE EL ORDEN DE ARMADO SIGUIENTE PERO CONSULTANDO SIEMPRE LOS PLANOS DE MONTAJE RESPECTIVOS.

A. INICIAR EL ENSAMBLE POR SECCIONES, TENDIENDO LAS PLACAS DEL SECTOR INFERIOR DE CADA SECCIÓN SOBRE LA CAMA NIVELADA O DENTRO DEL TANQUE, DONDE SE PREFIERA. CALZARLAS PARA PONERLAS A NIVEL. UNIRLAS ENTRE SÍ PUNTEANDO LAS JUNTAS RADIALES.

B. LA ENVOLVENTE EXTERIOR DEL PONTÓN, COMO YA SE INDICÓ, CONSTA DE DOS PARTES EN LOS TANQUES DE 500,000 BARRILES; COLOCAR LA INFERIOR SOBRE LA CUBIERTA O SECTOR INFERIOR PUNTEÁNDOLA Y EN SEGUIDA LA SUPERIOR. EN LOS OTROS TANQUES, LA ENVOLVENTE VIENE DE UNA PIEZA, PUNTEARLA AL SECTOR INFERIOR.

C. COLOCAR PLACAS DIVISORIAS DE LOS COMPARTIMIENTOS DEL PONTÓN, PUNTEÁNDOLAS A LA ENVOLVENTE Y AL SECTOR INFERIOR.

D. COLOCAR Y PUNTEAR EN LA MISMA FORMA QUE LA ENVOLVENTE EXTERIOR, LA INTERIOR, ASÍ COMO LAS PLACAS DE EXPANSION

E. MONTAR, AJUSTAR Y PUNTEAR EL SECTOR SUPERIOR A LAS ENVOLVENTES EXTERIOR E INTERIOR, PUNTEAR TAMBIÉN LAS PLACAS DIVISORIAS DE LOS COMPARTIMIENTOS CONFORME SE VAYA CERRANDO EL PONTÓN. CUIDAR QUE NO COINCIDAN LAS JUNTAS VERTICALES DE LAS ENVOLVENTES CON LAS UNIONES RADIALES DE AMBOS SECTORES.

F. SEGUIR LA MISMA SECUENCIA DE ENSAMBLADO INDICADA PARA LA PRIMERA SECCIÓN DEL PONTÓN EN LAS RESTANTES, MONTANDOLAS A UNA ALTURA ADECUADA Y EN FORMA PROVISIONAL, TAL COMO SE INDICA EN LA FIG. 4.2.2B. PUEDEN EMPLEARSE SEPARADORES EN LAS JUNTAS VERTICALES DE LAS ENVOLVENTES, AJUSTANDO Y LIGANDO TODAS LAS SECCIONES HASTA CERRAR EL CÍRCULO DEL PONTÓN. SE PERMITE USAR PLACAS DE CIERRE EN EL AJUSTE FINAL.

G. SOLDEO DEL PONTÓN. UNA VEZ ENSAMBLADA Y MONTADAS CON APOYOS PROVISIONALES, CADA UNA DE LAS SECCIONES, INICIAR EL SOLDEO DE LAS MISMAS, PRIMERO LAS ENVOLVENTES EXTERIOR E INTERIOR AL SECTOR INFERIOR, SOLDAR LUEGO LAS JUNTAS RADIALES Y LAS VERTICALES ENTRE LAS SECCIONES, AL MISMO TIEMPO SOLDAR LAS PLACAS DE LOS COMPARTIMIENTOS A LAS ENVOLVENTES Y AL FONDO DEL PONTÓN Y FINALMENTE EL SECTOR SUPERIOR O TAPA A LAS MISMAS ENVOLVENTES. ES NECESARIO DISPONER EN TODO MOMENTO DE UN JUEGO DE PLANOS DE MONTAJE DEL PONTÓN Y CONSULTARLO CONSTANTEMENTE PARA LOS EFECTOS DEL SOLDEO Y TRAZOS DE TODOS LOS ELEMENTOS ADICIONALES QUE LLEVA, COMO REGISTROS, GUÍA ANTIROTACIÓN, CAMISAS PARA LOS SOPORTES Y SUS REFUERZOS, ÁNGULOS DE SOSTÉN DEL SELLO Y SOLERA CIRCUNFERENCIAL DE APOYO DEL DIAFRAGMA.

4.2.2 OBRA FALSA PARA APOYO Y ARMADO DEL TECHO.

UNA VEZ SOLDADO EN SU TOTALIDAD EL PORTÓN PERO INSTALADO PRO-

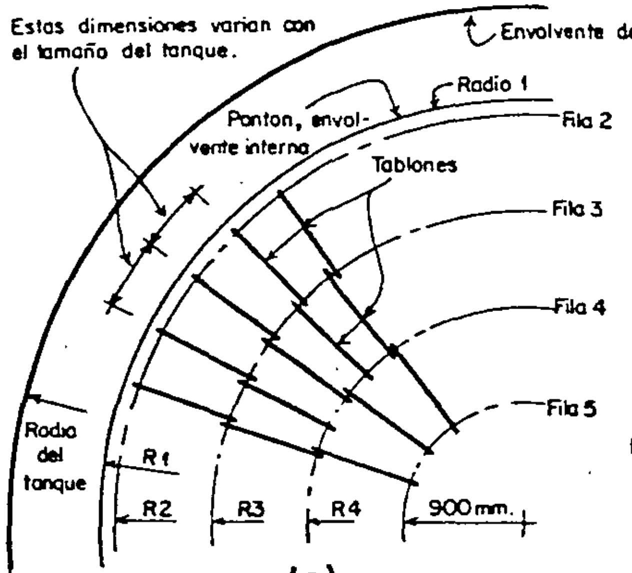
VISIONALMENTE, A FÍN DE TENER UNA SUPERFICIE NIVELADA PARA EL MONTAJE DEL DIAFRAGMA, ES NECESARIO PROYECTAR UN SISTEMA DE - OBRA FALSA PARA TENDER LAS PLACAS DEL TECHO EN UN PLANO HORIZONTAL CON RESPECTO AL FONDO CÓNICO DEL TANQUE.

UN PROYECTO SENCILLO DE OBRA FALSA, ES UTILIZAR UN SISTEMA DE APOYOS AJUSTABLES DEL DIAFRAGMA, CON TABLONES COLOCADOS RADIALMENTE, DE LOS UTILIZADOS EN LOS ANDAMIAJES, DE 2" X 8" X 8' Y 10' (LO QUE HAYA DISPONIBLE) A FIN DE OBTENER LA HORIZONTALIDAD QUE SE NECESITA.

LA FIGURA 4.2.2A INDICA UN ARREGLO TÍPICO DE LOS COMPONENTES DEL SISTEMA DE APUNTALAMIENTO. SE PUEDEN INTRODUCIR VARIANTES A ESTE ARREGLO, SIEMPRE QUE SE MANTENGA LA CONDICIÓN DE - LOGRAR UNA SUPERFICIE A NIVEL.

LOS APOYOS AJUSTABLES SON TUBOS CUYO EXTREMO INFERIOR, CON VARIAS PERFORACIONES LONGITUDINALES SE INTRODUCE EN UNA CAMISA TUBULAR CON LAS MISMAS PERFORACIONES Y UNA PLACA DE BASE SOLDADA A LA CAMISA, PERO INCLINADA A LA MISMA PENDIENTE DEL FONDO DEL TANQUE (FIG. 4.2.2.C). POR SU EXTREMO SUPERIOR, LLEVA SOLDADA UNA HORQUILLA HECHA CON PLACAS DE 13 MM DE ESPESOR -- DONDE SE APOYAN LOS TABLONES RADIALES. EN LOS SOPORTES DONDE HAY EMPALME DE TABLONES, LA HORQUILLA ES DE ANCHO DOBLE PARA ALOJAR DOS TABLONES. EN AMBOS CASOS LOS TABLONES SE FIJAN A - LAS HORQUILLAS CON UN PAR DE PERNOS DE 16 Ó 19 MM DE DIÁMETRO. MEDIANTE LA SERIE DE AGUJEROS EN LA BASE DE LOS POSTES, SE LOGRA UNIFORMIZAR LAS ALTURAS PARA TENER LOS TABLONES EN POSICIÓN HORIZONTAL. SE SUGIERE QUE CADA POSTE SEA FIJADO AL FONDO DEL TANQUE CON PUNTOS DE SOLDADURA, DEBIENDO ARRIOSTRARSE CONVENIENTEMENTE CON PERFILES ANGULARES EN ESTRELLA, CONECTADOS HORIZONTALMENTE MÁS O MENOS A LAS MEDIA ALTURA DE LOS POSTES DE SOSTEN, PARA IMPEDIR QUE ÉSTOS SE MUEVAN MIENTRAS SE -

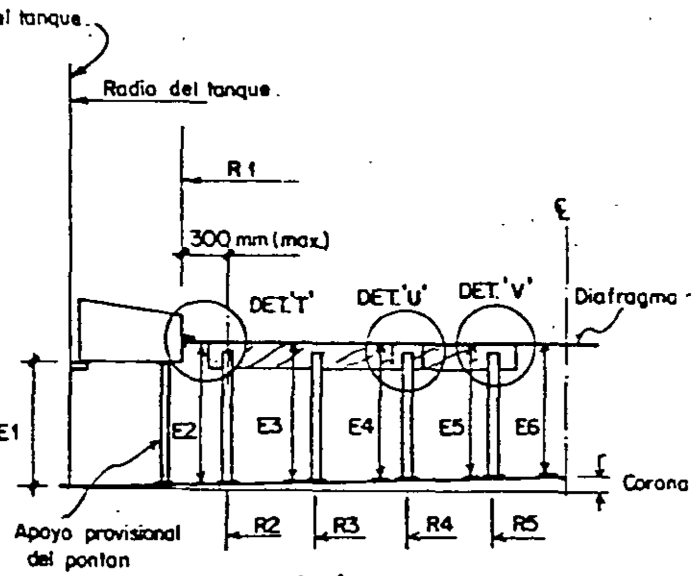
DISEÑO DE LA OBRA FALSA Y ESQUEMA DE SU INSTALACION



(a)

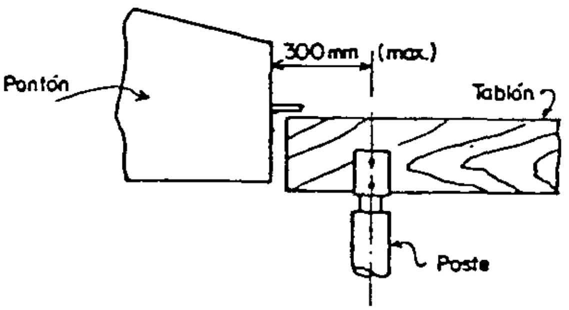
PLANTA PARCIAL

(Véase NOTA 2 para los radios)

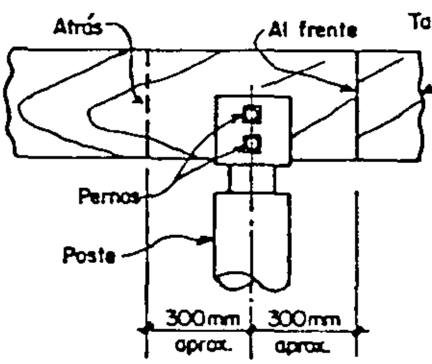


(b)

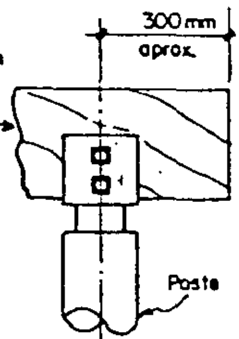
ELEVACION PARCIAL DEL ARREGLO DE POSTES Ver NOTAS 1 y 2.



DETALLE "T"



DETALLE "U"



DETALLE "V"

NOTA 1

Las elevaciones de los postes de soporte son ajustadas de modo que el diafragma se tiende plano sin pendientes.

La elevación de los postes de soporte es elegida, de modo que el diafragma tenga que elevarse con gatos 15 mm. cuando el techo está en posicion alta.

NOTA 2

Radios y número de radios son determinados tomando en cuenta la longitud de los tableros disponibles.

FIG. 4. 2. 2a

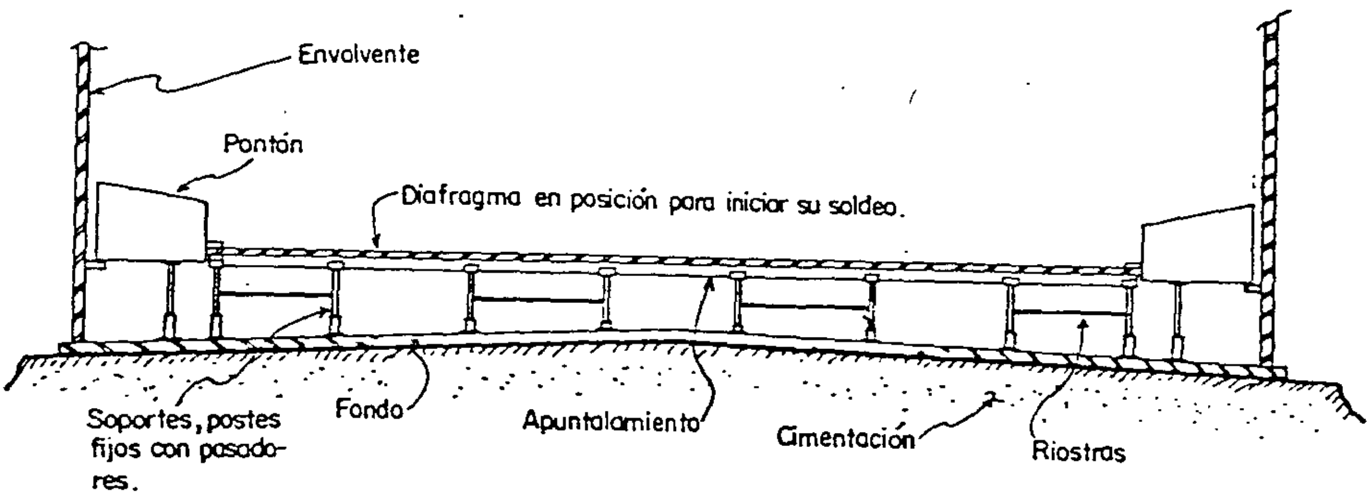


FIG. 4.2.2d

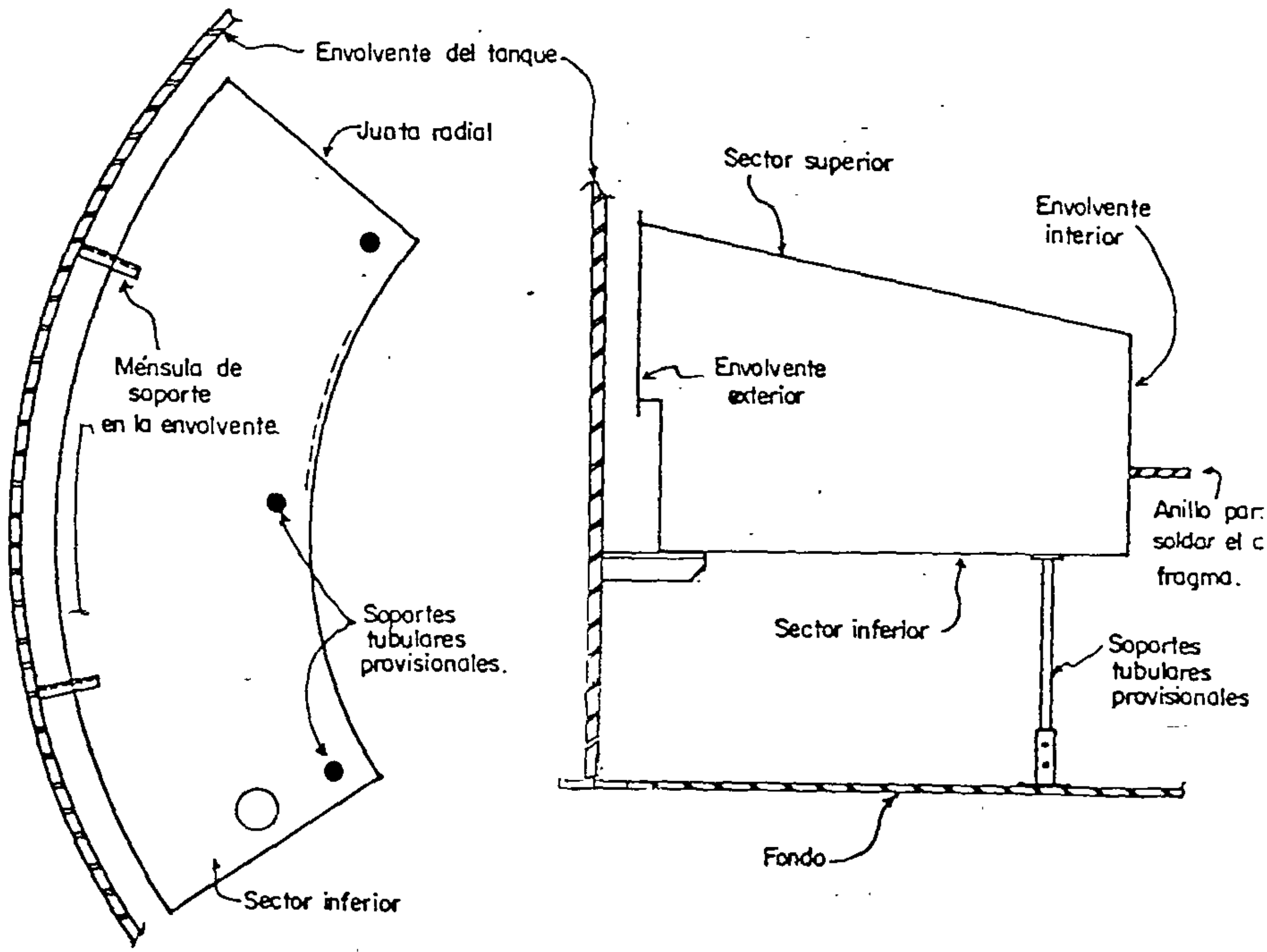
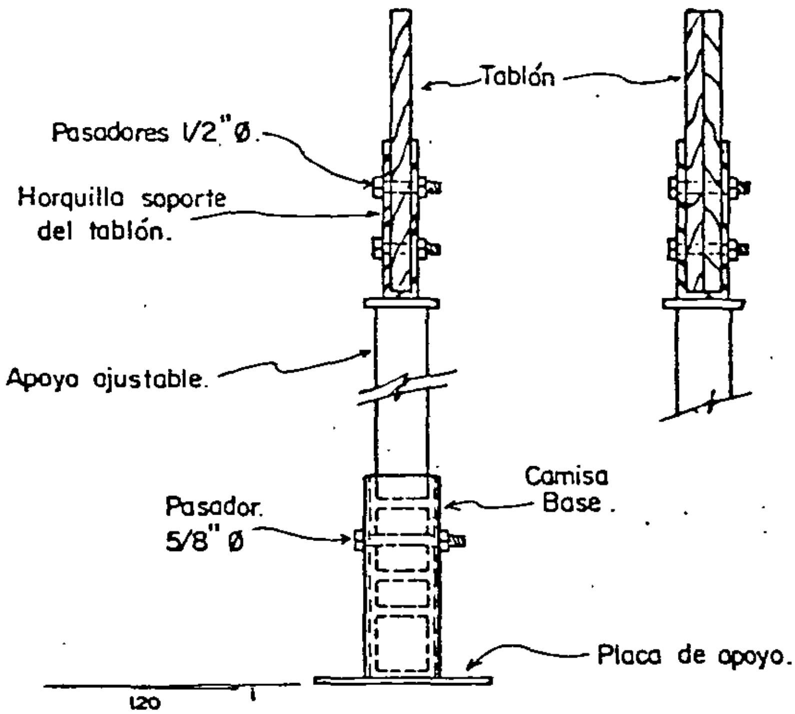


FIG. 4.2.2b



Detalle de un poste

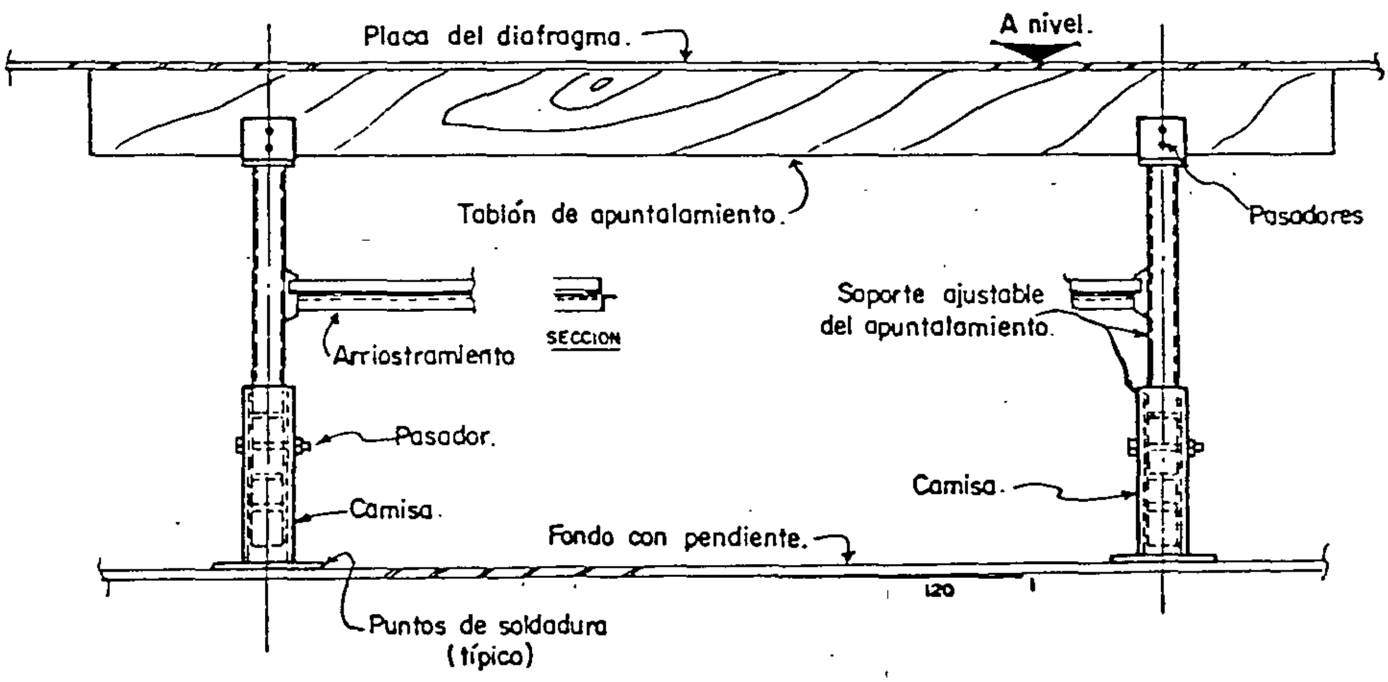


FIG. 4. 2. 2c

ESTÁ TENDIENDO EL TECHO. EN LA FIGURA 4.2.2D SE MUESTRA EN FORMA ESQUEMÁTICA EL ARREGLO GENERAL DE LA OBRA FALSA CON SUS SOPORTES ARRIOSTRADOS Y LISTA PARA RECIBIR LAS PLACAS DEL DIAFRAGMA.

4.2.3 ARREGLO Y TENDIDO DE LAS PLACAS DEL DIAFRAGMA DEL TECHO.

CON UN MÉTODO SEMEJANTE AL USADO PARA EL TENDIDO DE LAS PLACAS DEL FONDO (SECCIÓN 2.0) SE TIENDEN Y SE AJUSTAN LAS PLACAS DEL TECHO A BASE DE DIAFRAGMA SENCILLO, SOBRE LA OBRA PROVISIONAL DE APUNTALAMIENTO AVANZANDO DE LA PERIFÉRIA HACIA EL CENTRO DEL TANQUE. LOS DISTINTOS CONCEPTOS TALES COMO LÍNEAS DE DRENAJE, BOYAS, REGISTROS, ETC. DEBERÁN INTRODUCIRSE AL TANQUE ANTES DE COMPLETAR EL TENDIDO DEL DIAFRAGMA. LAS FIGURAS 4.2.3A Y 4.2.3.B EXHIBEN LOS DOS ARREGLOS USUALES DE TENDIDO DE PLACAS DE DIAFRAGMA.

4.2.4 SECUENCIA DE SOLDEO DEL DIAFRAGMA.

LAS FIGURAS MENCIONADAS EN EL PÁRRAFO ANTERIOR, INDICAN TAMBIÉN LA SECUENCIA DE LA SOLDADURA EN LAS PLACAS DEL DIAFRAGMA. COMO EN EL FONDO, SOLDAR SIEMPRE DEL CENTRO HACIA LA PERIFÉRIA. EL USO DE CANDADOS EN LAS JUNTAS ENTRE LAS PLACAS HORIZONTALES Y LAS IRREGULARES ESTÁ PERMITIDO PUÉS EVITARÁN DEFORMACIONES MAYORES DEL DIAFRAGMA EN ESTAS PARTES. TODAS LAS RECOMENDACIONES DADAS PARA EL SOLDEO DEL FONDO DEL TANQUE, SON APLICADAS A LA SOLDADURA DEL DIAFRAGMA.

4.2.5 INSTALACIÓN DE BOYAS Y DE SOPORTES DEFINITIVOS DEL TECHO.

EN LA FIG. 4.2.5A SE SUGIERE UN ARREGLO DE BOYAS Y DE LOS POSTES DE SOPORTE. SE NOTARÁ QUE ALGUNOS DE LOS POSTES TUBU-

PEMEX	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR : Ing. I. J. L.	FECHA	HOJA
	APROBADO POR : Ing. J. H. B.	IV-86	10 DE 16
SECCION 4.0 MONTAJE DEL TECHO FLOTANTE	MANUAL DE MONTAJE Nº 1		

SECUENCIA DE OPERACIONES

1. Fijar y soldar los cordones de cada cuadrante en el orden numérico indicado.
2. Las secuencias 3 5 y 7 son solo operaciones de sujeción con candados de placa
3. Emplear la técnica de retroceso.

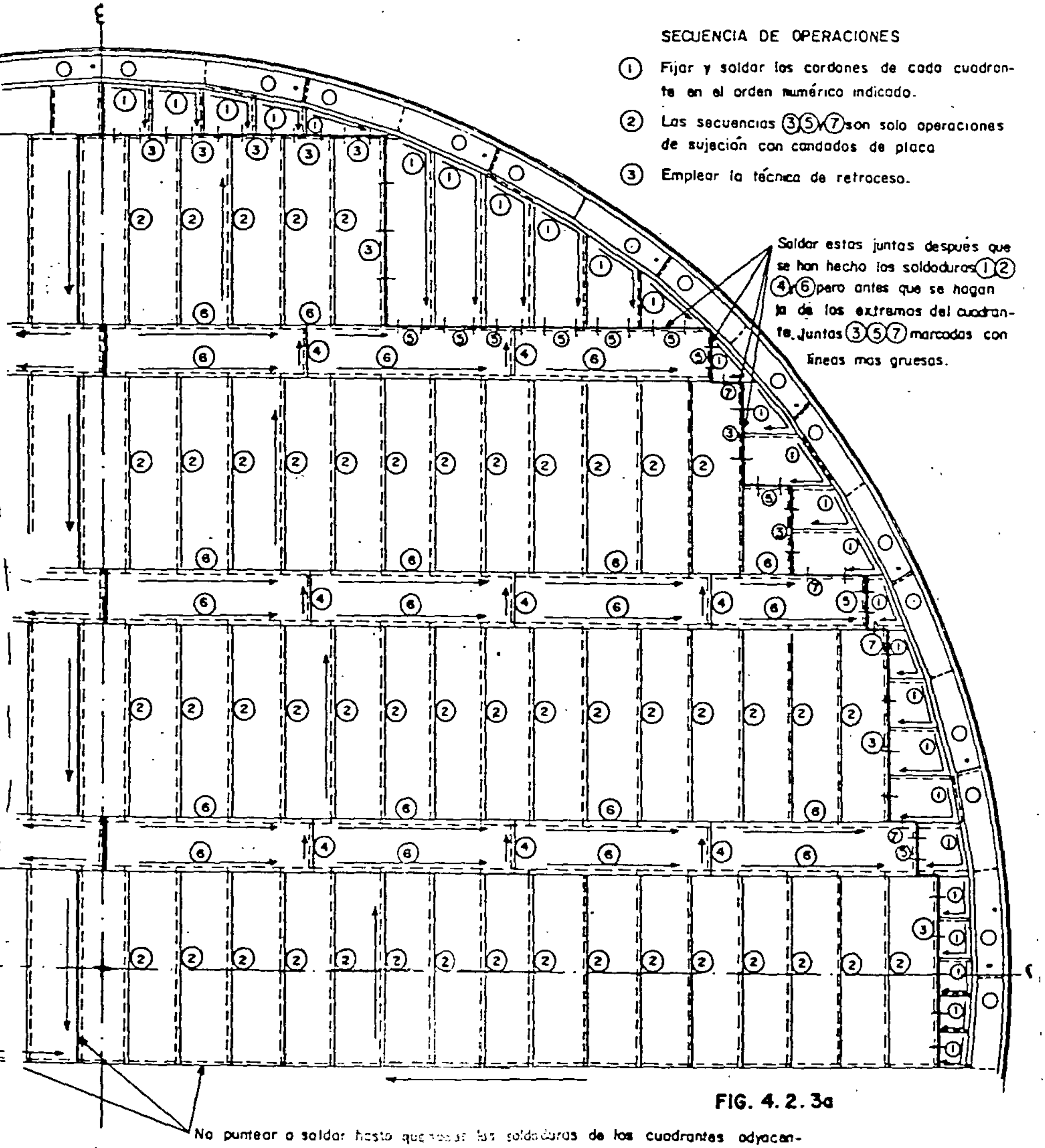


FIG. 4.2.3a

ARREGLO DEL TENDIDO DE PLACAS DEL DIAFRAGMA EN TANQUES DE TECHO FLOT.

ARREGLO DEL TENDIDO DE PLACAS DEL DIAFRAGMA EN TANQUES DE TECHO FLOT. Y SECUENCIA DE SOLDEO

NOTAS :

1.- Fijar y soldar los cordones de cada cuadrante en el orden numérico indicado.
2.- Las secuencias (3) (5) y (19) son solo operaciones de sujeción con candados de placa.
3.- Emplear la técnica de retroceso en todos los cordones.

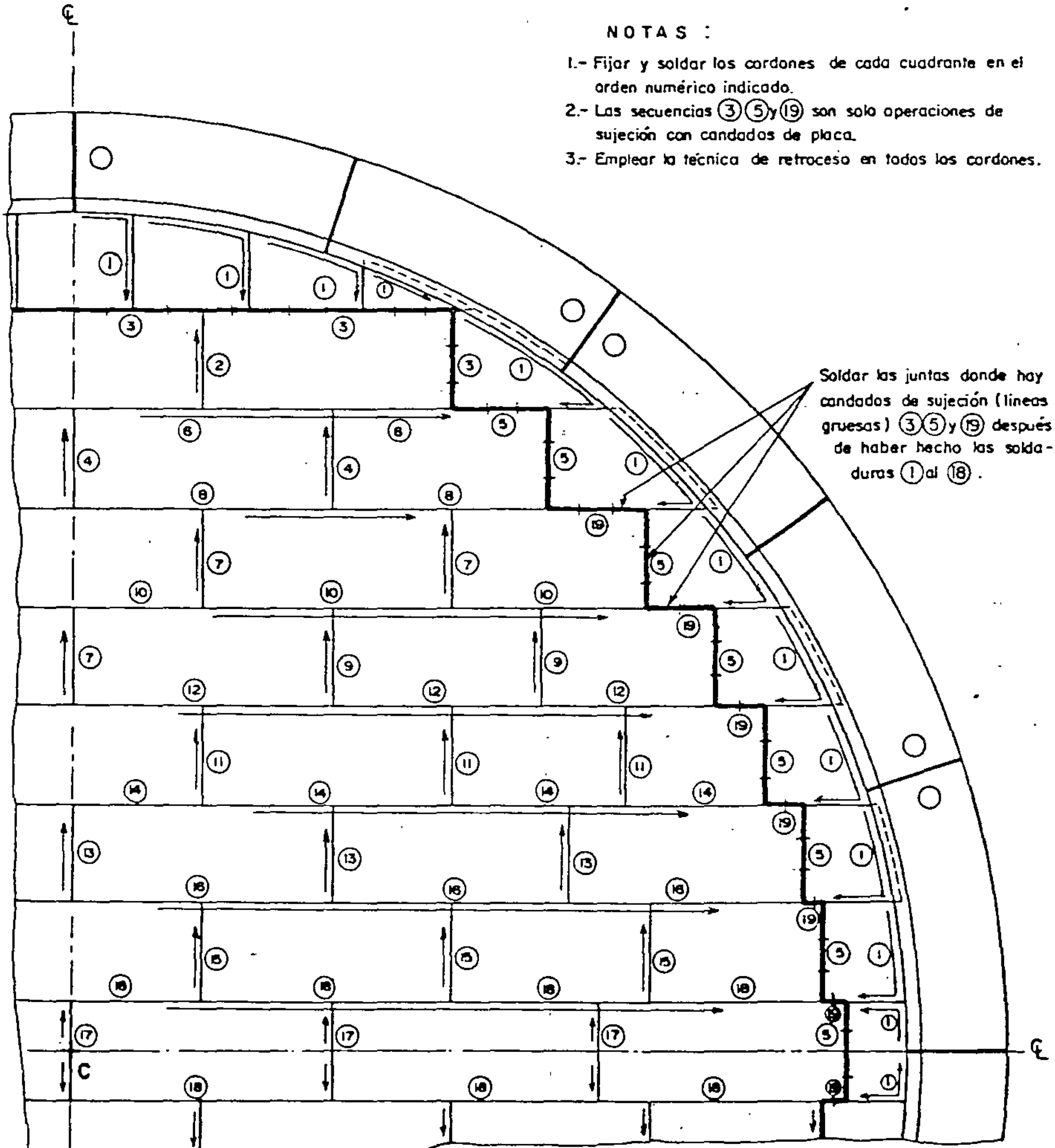
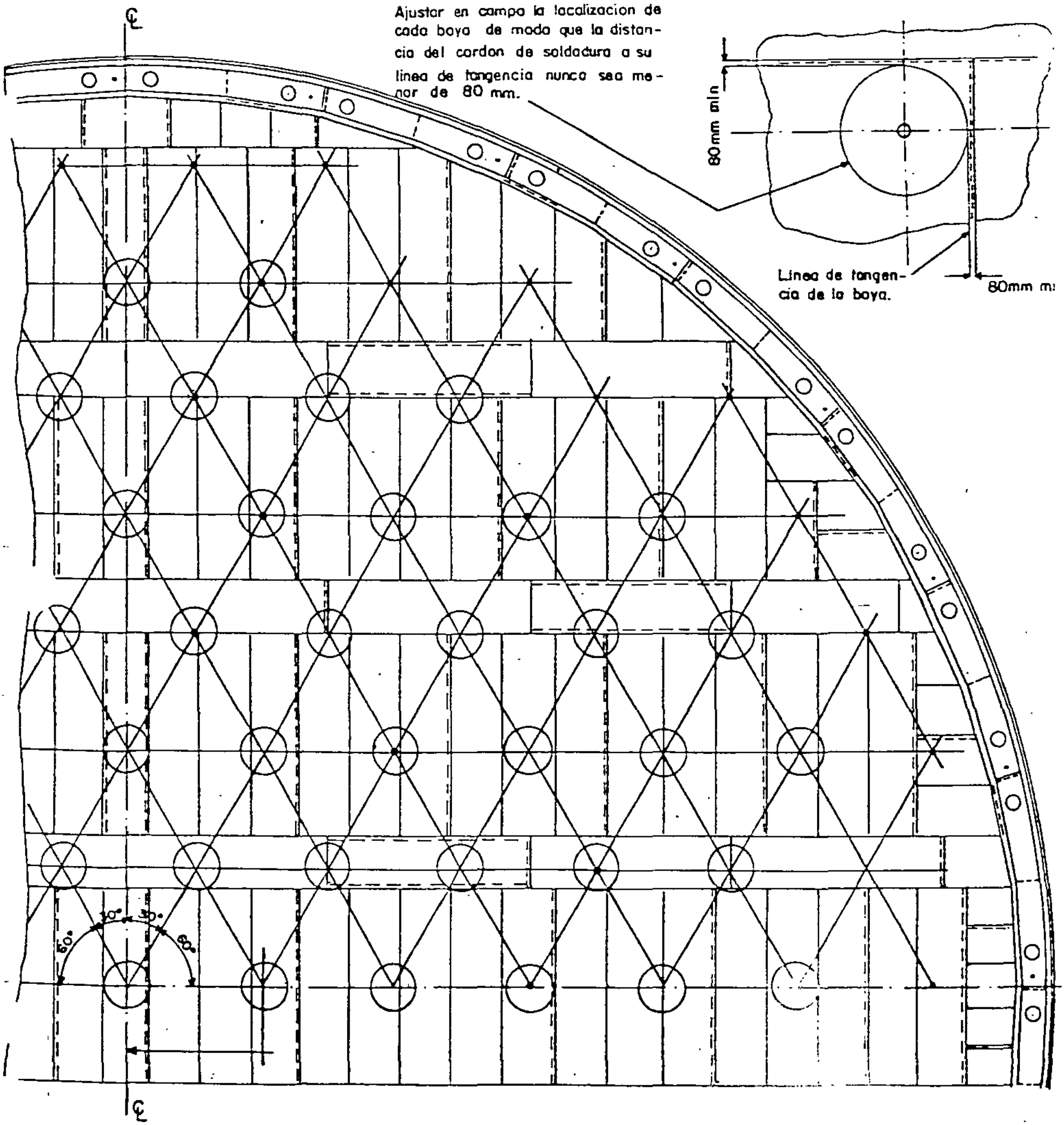


FIG. 4. 2.3b



Distribucion de boyas en tanques de 500,000 bls. de capacidad

FIG. 4. 2. 5a

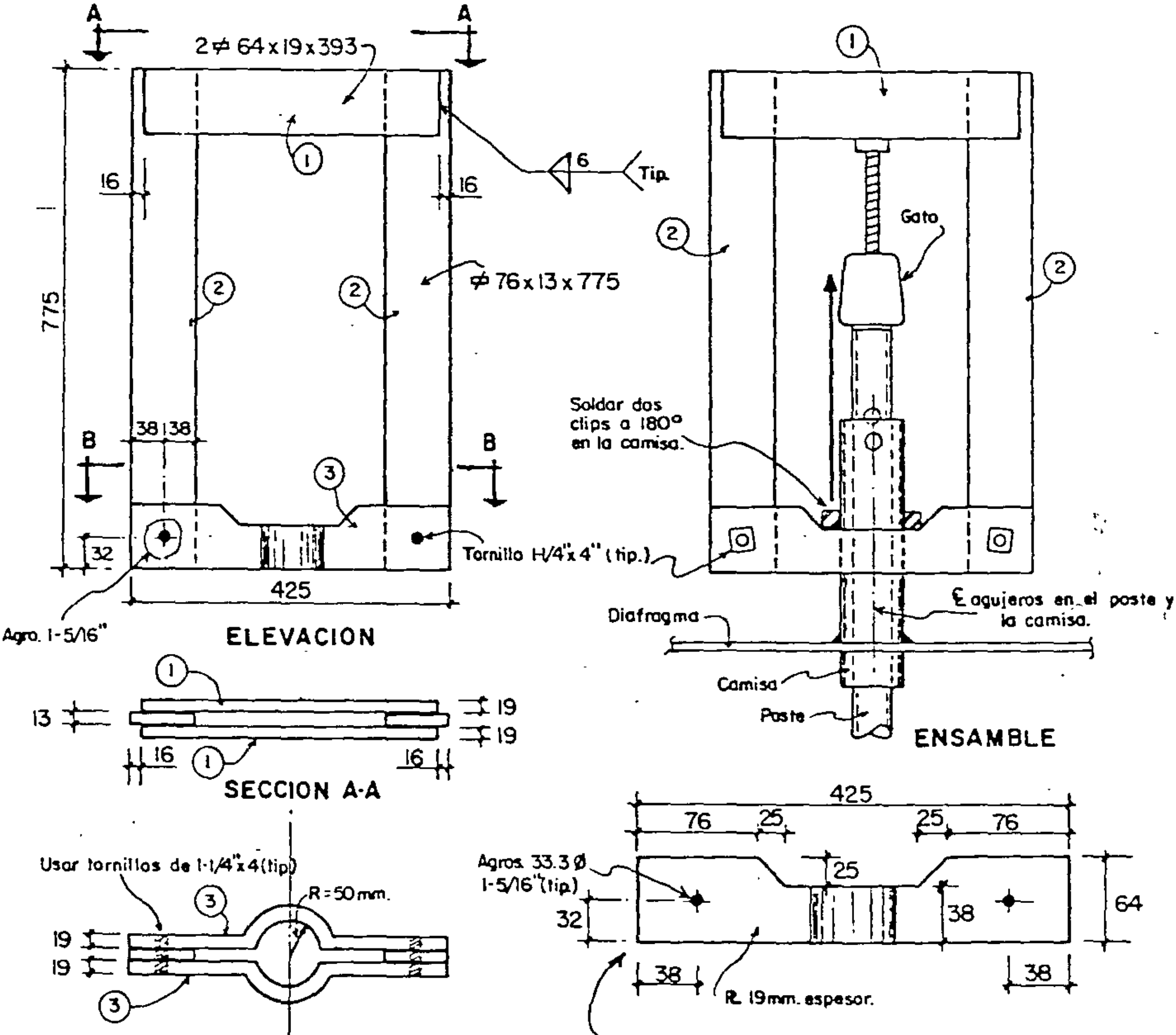
LARES CAEN DIRECTAMENTE EN COSTURAS DE SOLDADURA. SE RECOMIENDA, EN ÉSTOS CASOS, QUE ÉSTOS APOYOS SEAN RELOCALIZADOS EN EL CAMPO DE MODO QUE NO COINCIDAN CON LAS JUNTAS SOLDADAS DEL DIAFRAGMA. AGREGANDO A LO ANTERIOR, SERÁ NECESARIO DESVIAR BOYAS (DONDE SEA NECESARIO) UNA PEQUEÑA DISTANCIA A FIN DE SITUARLAS A UNOS 75 MM MÍNIMOS, O BIEN MOVERLAS A QUEDAR SOBRE LA COSTURA. SOLDARLAS AL DIAFRAGMA Y ABRIR AGUJEROS PARA EL PASO DE LAS CAMISAS DE SUS POSTES DE APOYO. INSERTARLAS EN EL AGUJERO DE LA PLACA DE REFUERZO Y SOLDARLAS.

DESPUÉS DE LA INSTALACIÓN DE LA TOTALIDAD DE LAS BOYAS Y ALOJADOS LOS POSTES DE APOYO EN SUS CAMISAS EN EL DIAFRAGMA Y EL PONTÓN, ES NECESARIO ASEGURARLOS MEDIANTE PASADORES. VÉANSE LOS PLANOS DE MONTAJE CORRESPONDIENTES A CADA CAPACIDAD DE TANQUES, CONSULTANDOLOS CUANTAS VECES SEA NECESARIO. SE HARÁN LAS PRUEBAS CORRESPONDIENTES DE TODA LA INSTALACIÓN Y DE LAS SOLDADURAS FINALES.

PARA ASEGURAR LOS POSTES A SU CAMISA CORRESPONDIENTE COMO SE INDICA EN EL PLANO DE MONTAJE, ES NECESARIO ELEVAR EL DIAFRAGMA HASTA 15 MM PARA INSERTAR EL PERNO DE SUJECIÓN. SE SUGIERE UNA TÉCNICA A BASE DE GATOS APOYADOS EN BASTIDORES HECHOS DE FIERRO PLANO. LA FIGURA 4.2.5B INDICA EL MÉTODO MENCIONADO Y LA TÉCNICA DESARROLLADA PARA EFECTUAR LA OPERACIÓN, ASÍ COMO EL DETALLE DE LOS ELEMENTOS DEL BASTIDOR PARA SU FABRICACIÓN.

DESPUÉS QUE SE HA TERMINADO LA INSTALACIÓN DEFINITIVA DE LA TOTALIDAD DE LOS POSTES, LA OBRA FALSA DE SOPORTE PUEDE SER DESMANTELADA Y SIGUIENDO A ÉSTO YA SE PUEDEN INSTALAR LOS DRENAJES DEL TECHO Y LOS REGISTROS.

DETALLES Y USO DE BASTIDORES Y GATOS PARA ENSAMBLAR
LOS POSTES DE SOPORTE DEL DIAFRAGMA DEL TECHO FLOTANTE A SUS CAMISAS.



Se necesitan 2 pzas. S/croquis.
PESO APROX. DEL BASTIDOR = 28 Kg.

NOTAS :

1.- Soldar una abrazadera (3) a las soleras verticales (2) y dejar suelta la otra, atornillándola después de abrazar la camisa.

2.- Usar simultaneamente tres bastidores colocados en tres postes de soporte. Con gatos poner los postes en posición e insertar pasadores a través de los agujeros de la camisa y del poste. Continuar esta secuencia hasta que todos los pasadores han sido instalados.

FIG. 4.2.5b

4.2.6 INSTALACIÓN DE ACCESORIOS.

TERMINADO EL MONTAJE DEL DIAFRAGMA, PROCEDER A LA INSTALACIÓN DE LOS ACCESORIOS COMPLEMENTARIOS REQUERIDOS PARA EL FUNCIONAMIENTO DEL TECHO FLOTANTE, DE ACUERDO CON LAS RECOMENDACIONES SIGUIENTES:

1. LOCALIZAR Y ALINEAR LOS CARRILES DE LA ESCALERA RODANTE Y LAS OREJAS QUE VAN SOLDADAS A LA PLACA DE EXTENSIÓN DE LA ENVOLVENTE. TENER ESPECIAL CUIDADO EN LA INSTALACIÓN DE LOS CARRILES PARA EL DESPLAZAMIENTO DE LA ESCALERA.

2. ARMAR LAS SECCIONES DE LA ESCALERA SOBRE LOS CARRILES EN SU POSICIÓN EXTREMA HORIZONTAL. SOLDAR LAS SECCIONES ENTRE SÍ, Y LEVANTAR EL EXTREMO SUPERIOR HASTA ENSARTAR EL PERNO DE ARTICULACIÓN. HACER LA INSTALACIÓN COMPLETA ANTES DE LA PRUEBA DE LLENADO CON AGUA.

3. ESCALERA EXTERIOR EN ESPIRAL. EL MONTAJE DE ESTA ESCALERA SE LLEVA A CABO DESPUÉS DE TERMINADA LA ERECCIÓN Y SOLDEO DE LA ENVOLVENTE DEL TANQUE. SEGUIR EL ORDEN DEL MONTAJE DEL EXTREMO INFERIOR AL SUPERIOR.

4. LOCALIZAR EL INDICADOR DE NIVEL SOBRE LA EXTENSIÓN DE LA ENVOLVENTE Y CON PLOMADA LOCALIZAR SOBRE EL PONTÓN EL POZO DEL FLOTADOR. COLOCAR ÉSTE, SOLDARLO Y COMPROBAR SU HERMETICIDAD.

5. LOCALIZADA LA POSICIÓN DE LA GUÍA ANTIROTACIÓN, COLOCAR EL SOPORTE SUPERIOR Y CON PLOMADA TRANSPORTAR LA ABERTURA DE LA CAMISA GUÍA EN EL PONTÓN Y LA POSICIÓN DEL SOPORTE INFERIOR. ARMAR Y SOLDAR EL CONJUNTO Y VERIFICAR LA HERME

TICIDAD DE LA CAMISA-GUÍA. LA INSTALACIÓN SE HARÁ ANTES DEL LLENADO DEL TANQUE CON AGUA.

6. LOCALIZAR LAS VÁLVULAS AUTOMÁTICAS DE VENTEO, ABRIR SUS AGUJEROS, MONTAR CAMISAS SOBRE EL DIAFRAGMA Y SOLDAR.

7. LOCALIZAR E INSTALAR POZOS Y REGISTROS DE MUESTREO, VENTILAS MANUALES, BARRAS CENTRADORAS, GUARDA MANGUERAS, PARRILLAS DE DRENAJE, ETC. TODA PERFORACIÓN HECHA AL DIAFRAGMA DEBERÁ VERIFICARSE CON LÍQUIDO PENETRANTE DESPUÉS DE SOLDAR EL ACCESORIO.

NOTA: TODAS LAS LOCALIZACIONES INDICADAS, VIENEN BIEN DEFINIDAS EN LOS PLANOS DE MONTAJE DEL TECHO. CONSULTARLOS PARA DISIPAR DUDAS. TAMBIÉN EN DICHOS PLANOS SE INDICAN LA SOLDADURA DE CAMISAS, REFUERZOS, ETC.

5.0 INSTALACION DEL TUBO-SELLO

5.1 GENERALIDADES.

EL SISTEMA DE SELLADO PARA LOS TANQUES DE TECHO FLOTANTE EN USO EN PEMEX ES EL DENOMINADO TUBO-SELLO. SE TRATA DE UN DISPOSITIVO IDEADO PARA CERRAR HERMÉTICAMENTE EL ESPACIO ANULAR ENTRE EL PONTÓN PERIMETRAL DEL TECHO FLOTANTE Y LA ENVOLVENTE CILÍNDRICA O PARED DEL TANQUE. ES UN TIPO DE SELLO MUY EFECTIVO PARA REDUCIR A UN MÍNIMO LAS PÉRDIDAS POR EVAPORACIÓN DEL PRODUCTO ALMACENADO, MINIMIZANDO EL ESCAPE DE LOS VAPORES AL MEDIO AMBIENTE. LA ADOPCIÓN POR PARTE DE PEMEX DEL TIPO DE SELLO DESCRITO, SE BASÓ EN SU ALTA EFICACIA CON RELACIÓN A OTROS TIPOS DE DISEÑO MECAÑICO Y CON MATERIALES METÁLICOS.

EL TUBO-SELLO CONSTA ESCENCIALMENTE DE UN TUBO FLEXIBLE, BANDA DE DESGASTE Y PROTECCIÓN DE LA INSTALACIÓN PRINCIPALMENTE CONTRA LA LLÚVIA. EL TIPO EXPANSIBLE TIENE UN SOPORTE ADICIONAL DE APUNTALAMIENTO. EL TUBO SE LLENA GENERALMENTE CON PETRÓLEO DIAFANO PERO PUEDEN USARSE OTROS LÍQUIDOS SI SON COMPATIBLES CON EL MATERIAL DEL TUBO Y CON EL RANGO DE TEMPERATURA AMBIENTE ENTRE EL VERANO Y EL INVIERNO, EN ZONAS DE CLIMA EXTREMOSO. COMO EL TUBO QUE ES PROPIAMENTE EL SELLO, SE LLENA CON UN LÍQUIDO, SE ACOMODA ASIMISMO A LAS MENORES IRREGULARIDADES DE LA ENVOLVENTE TALES COMO LAS COSTURAS DE LAS SOLDADURAS. LAS FIGURAS 5.1A Y 5.1B REPRESENTAN INSTALACIÓN COMPLETA DEL TUBO-SELLO, ASÍ COMO SUS PARTES COMPONENTES.

EL TUBO Y LA BANDA DE DESGASTE, SE FABRICAN CON HULE SINTÉTICO RESISTENTE A LA ABRASIÓN Y A LOS ELEMENTOS QUÍMICOS DEL CRUDO Y DE LOS PRODUCTOS LIGEROS ALMACENADOS. EL RANGO DE TEM

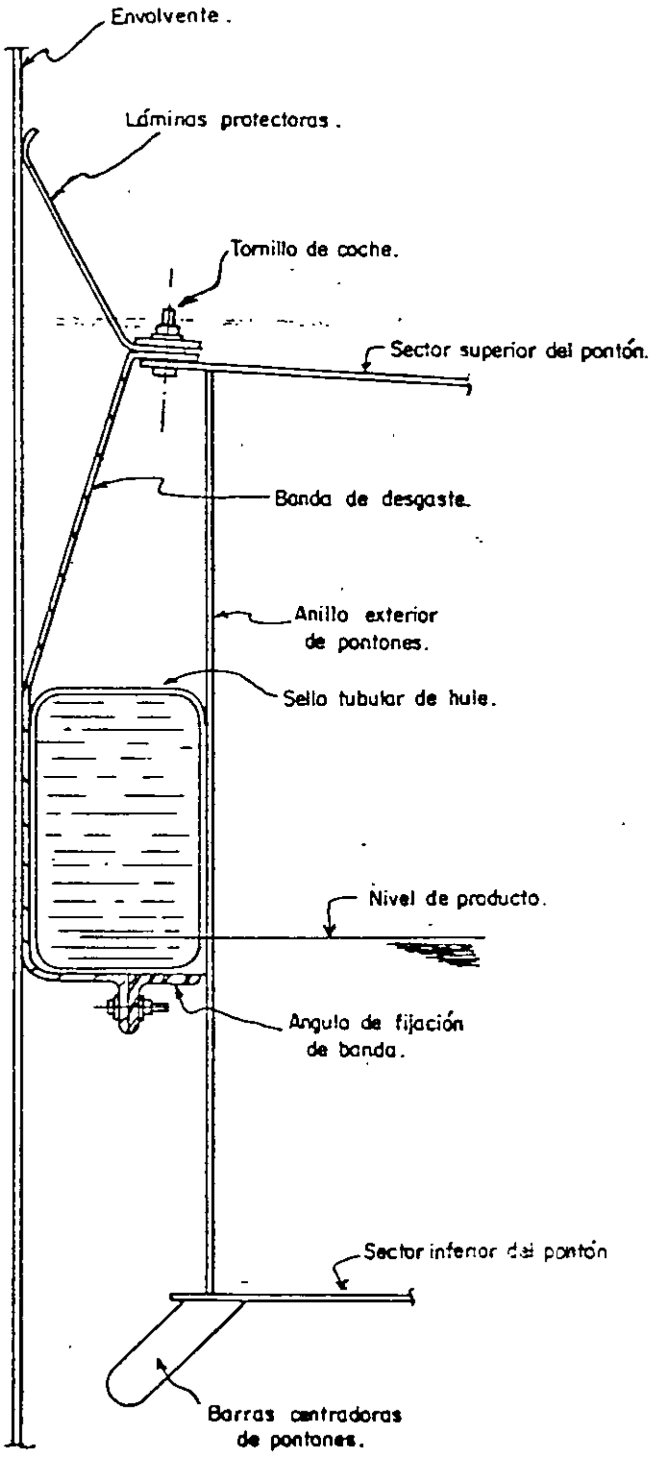


Fig. 5.1.a Instalacion del tubo-sello en tanque de 200,100 y 55 M.B.

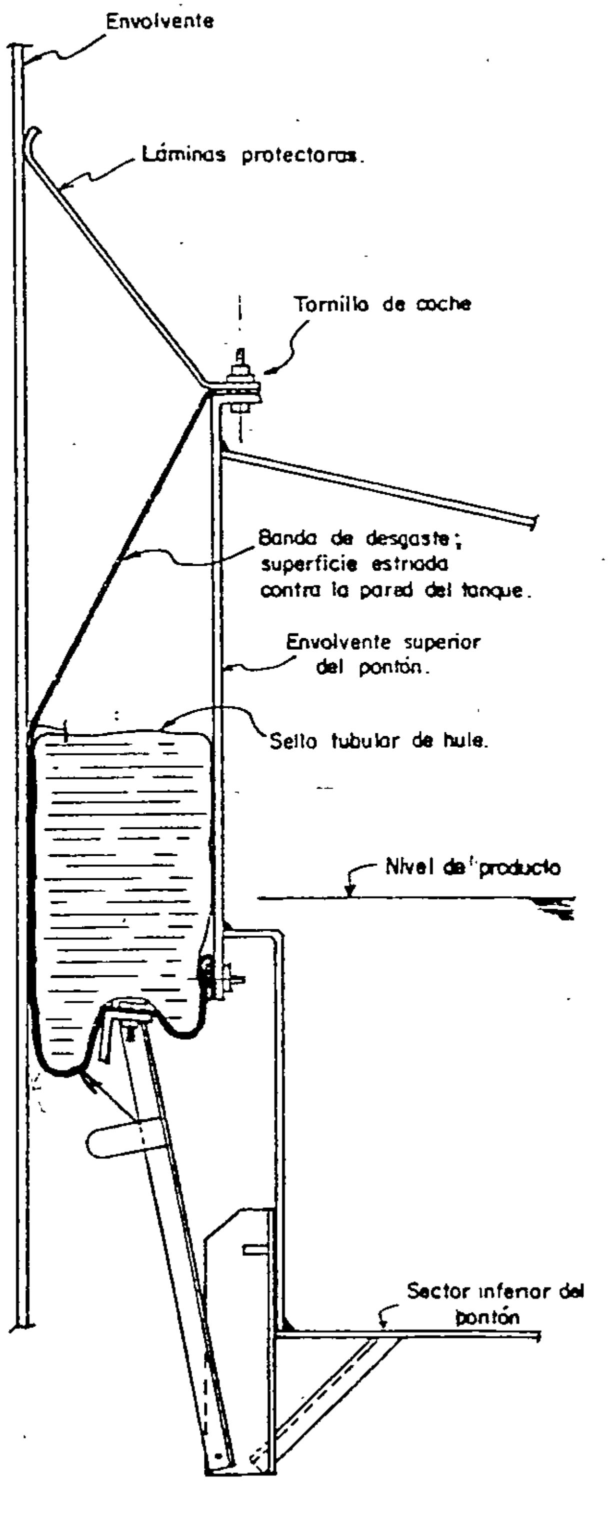


Fig. 5.1.b Instalacion del tubo-sello en tanques de 500 M.B.

PERATURA PARA EL MATERIAL ESTANDAR ES DE - 29°C A + 93°C - - (-20°F A + 200°F). SE DISPONE, SIN EMBARGO DE FÓRMULAS ESPECIALES PARA LA FABRICACIÓN DE SELLOS PARA CONDICIONES MÁS SEVERAS DEL MEDIO AMBIENTE.

5.2 INSTRUCCIONES PARA LA INSTALACIÓN DEL SELLO.

A CONTINUACIÓN SE EXPONE LA SECUENCIA QUE SE SIGUE PARA LA -- INSTALACIÓN CORRECTA DEL TUBO-SELLO, APLICANDO LAS INSTRUCCIONES CONTENIDAS EN EL PLANO RESPECTIVO.

1. ANTES DE INICIAR LA INSTALACIÓN DEL TUBO, DEBERÁ ESTAR -- COMPLETAMENTE MONTADO Y SOLDADO EL FONDO, LA ENVOLVENTE Y EL TECHO FLOTANTE DEL TANQUE. EL TECHO, APOYADO EN EL FONDO CON SUS SOPORTES DEFINITIVOS Y CONCÉNTRICO CON LA ENVOLVENTE DEL TANQUE. REVISAR QUE LA SEPARACIÓN ENTRE LA - ENVOLVENTE EXTERIOR DEL PONTÓN Y LA PARED DEL TANQUE, ESTÉ DE ACUERDO CON LAS DIMENSIONES DEL PLANO DE MONTAJE. - LA ENVOLVENTE DEL PONTÓN DEBERÁ ESTAR COMPLETAMENTE VERTICAL SIN NINGUNA CURVATURA O COMBA EN SU PARTE SUPERIOR.

2. CADA DOS TORNILLOS UNO SI Y EL OTRO NO, LOCALIZADOS EN EL SECTOR SUPERIOR DEL PONTÓN, ESTÁN EN LÍNEA DIRECTAMENTE - CON CADA AGUJERO DEL ÁNGULO INFERIOR DE SUJECIÓN. ES SUFICIENTE VERIFICAR MÁS O MENOS CADA DIEZ PERNOS CON SUS CORRESPONDIENTES AGUJEROS DEL ÁNGULO, QUE ESTÉN EN LÍNEA Y COMPROBAR IGUALMENTE QUE SUS SEPARACIONES SEAN LAS MISMAS.

3. LOS TORNILLOS DEBERÁN SOLDARSE AL PONTÓN Y TODAS LAS DIVERSAS SOLDADURAS Y ESMERILADOS DEL SECTOR SUPERIOR DEL - PONTÓN DEBERÁN COMPLETARSE ANTES DE INSTALAR LA BANDA DE DESGASTE.

4. LAS JUNTAS VERTICALES DEL ÁNGULO INFERIOR DE SUJECIÓN, DEBERÁN ESTAR ALINEADAS Y AL RAS; SOLDARLAS Y ESMERILARLAS A DEJARLAS ALISADAS. CUALQUIER SALIENTE EN LAS ALAS DEL ÁNGULO, TAMBIÉN SERÁ REBAJADO.

5.- SALPICADURAS DE SOLDADURAS, REBABAS Y CUALQUIER OTRO SALIENTE CORTANTE QUE HAYA EN EL ESPACIO DONDE SE ALOJARÁ EL SELLO, DEBERÁN SER REMOVIDOS.

6. ANTES DE DESEMPACAR LA BANDA DE DESGASTE Y EL TUBO-SELLO-LIMPIAR, BARRIENDO EL DIAGRAMA Y EL PONTÓN. LA BANDA Y EL TUBO VIENEN EN CAJAS SEPARADAS Y ÉSTAS SE ABRIRÁN HASTA QUE SE REQUIERA.

7. LA BANDA DE DESGASTE TIENE UNA CARA LISA Y LA OTRA ESTRIADA. SE INSTALA COMO UN ANILLO CONTINUO, PERO VIENE EN VARIOS TRAMOS. DESEMPACAR ÉSTOS CUIDADOSAMENTE, DESENROLLARLOS SOBRE EL PONTÓN Y EMPALMARLOS SIGUIENDO LAS INSTRUCCIONES DEL PLANO DE MONTAJE. TENDER LA BANDA CON SU CARA-LISA HACIA ARRIBA Y CON LAS PERFORACIONES PARA INSERTAR LOS TORNILLOS DEL PONTÓN CADA 152 MM (6") HACIA EL INTERIOR DEL TANQUE,. FIJAR LA BANDA EN LOS TORNILLOS COMO SE MUESTRA EN LA FIGURA 5.2A Y HACIENDO UN GIRO DE LA BANDA-PARA QUE SU CARA ESTRIADA MIRE HACIA LA ENVOLVENTE DEL TANQUE, DESCOLGARLA EN EL ESPACIO ENTRE PONTÓN Y ENVOLVENTE Y APOYARLA EN EL ÁNGULO INFERIOR DE SUJECIÓN. COLOCAR-EN FORMA PROVISIONAL SOLERAS DE FIJACIÓN (LAS QUE TRAEN AGUJEROS OVALADOS) APROXIMADAMENTE A CADA METRO Y APRETAR LAS TUERCAS EN LOS TORNILLOS, HASTA DEJAR FIRME LA BANDA-SIN DETERIORARLA POR EXCESO DE APRIETE. DESPUÉS SE ATORNILLARÁN EN FORMA DEFINITIVA, AL INSTALAR EL TUBO - - - - -

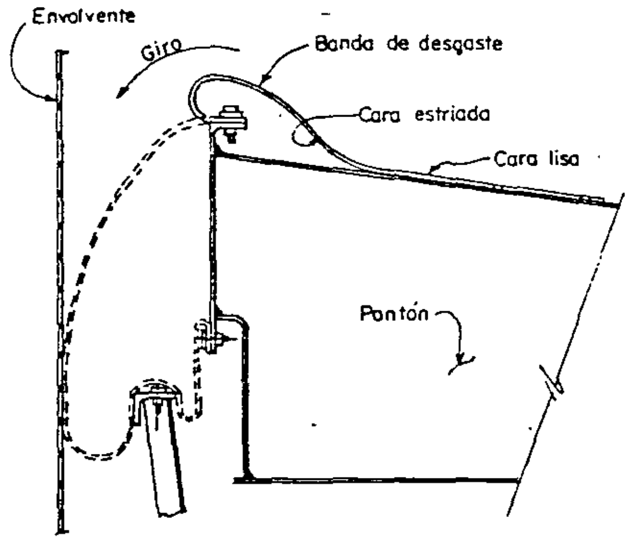
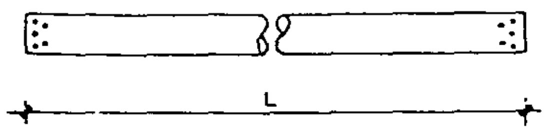
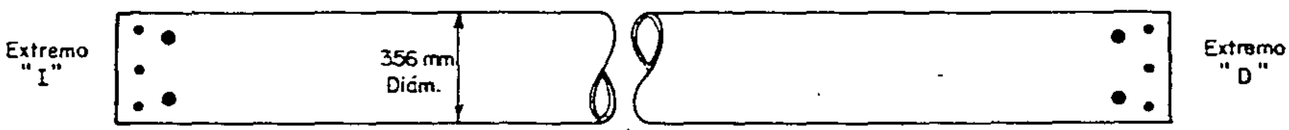


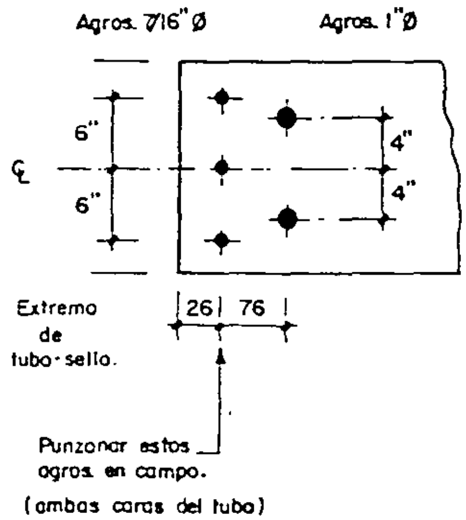
FIG. 5.2a

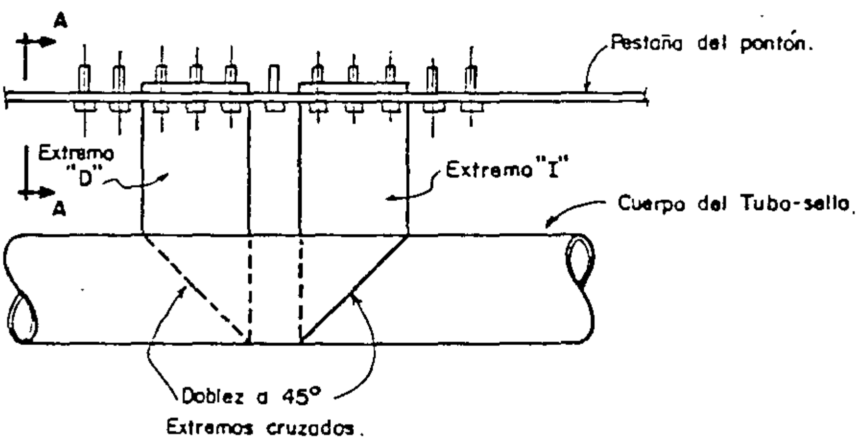
TUBO-SELLO



L = 270,053 mm. en tanques de 500 MB.
L = 172,974 mm. en tanques de 200 MB.
L = 128,778 mm. en tanques de 100 MB.
L = 96,317 mm. en tanques de 55 MB.







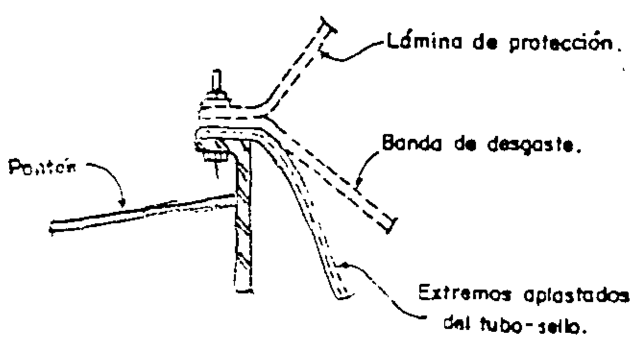


FIG. 5.2b

SECCION A-A

SELLO Y LAS LÁMINAS DE PROTECCIÓN. ATORNILLAR LA BANDA AL ÁNGULO DE FIJACIÓN INFERIOR, USANDO TORNILLOS DE COCHE -- CON LA CABEZA ALISADA HACIA LA ENVOLVENTE. NO APRETAR LOS TORNILLOS. EN LOS TANQUES DE 500,000 BARRILES, LA BANDA - SE FIJA ABAJO EN DOS APOYOS: EN EL ÁNGULO INFERIOR SOBRE EL BRAZO ANGULAR Y DIRECTAMENTE EN LA PLACA INFERIOR 1B - DE LA ENVOLVENTE EXTERIOR DEL PONTÓN (FIG. 5.1B). EN LOS TANQUES DE 200,000 A 55,000 BLS. DE CAPACIDAD, SOLAMENTE SE FIJA LA BANDA EN UN ÁNGULO INFERIOR Y CON SU SOLERA - CORRESPONDIENTE DE SUJECIÓN (FIG. 5.1A). AL APRETAR LOS TORNILLOS DESPUÉS DE INSTALADO EL TUBO-SELLO, SE OBTEN-- DRÁ UN SELLO LÍQUIDO HERMÉTICO.

8. DESPUÉS DE COMPLETAR EL ATORNILLADO PROVISIONAL EN EL ÁN GULO INFERIOR, DESEMPACAR EL TUBO-SELLO INSPECCIONANDO - EL INTERIOR DE LA CAJA, POR SI HAY CLAVOS QUE HAYAN PICA DO EL TUBO. SACARLO CON MUCHO CUIDADO PARA EVITAR UNA PI CADURA. DESENROLLARLO Y TENDERLO CERCA DEL PERÍMETRO EX- TERIOR DE LA TAPA DEL PONTÓN. TODAS LAS TORCEDURAS Y - - ARRUGAS EN EL TUBO DEBERÁN SUPRIMIRSE, ALISANDOLO CUANDO SE ESTÁ EXTENDIENDO. PREPARAR SUS EXTREMOS (FIG. 5.2B) - APLASTANDO LAS PUNTAS Y PUNZONAR LOS TRES AGUJEROS DE 11 MM (7/16") A LAS DISTANCIAS INDICADAS. LOS DOS AGUJEROS DE 25.4 MM. (1") EN CADA EXTREMO YA VIENEN HECHOS DE FÁ- BRICA Y SIRVEN DE RESPIRADERO Y PURGA.

9. VACIARLE AL TUBO POR UN EXTREMO APROXIMADAMENTE 40 LI-- TROS DE PETRÓLEO DIAFANO. SOSTENER EL EXTREMO LEVANTA- DO Y A UNOS 2.00 METROS APROXIMADAMENTE ELEVAR EL TUBO A LA MISMA ALTURA PARA ACUMULAR EL LÍQUIDO EN EL COLUM- PIO RESULTANTE E INSPECCIONAR EL TRAMO CUIDADOSAMENTE - PARA DESCUBRIR POSIBLES FUGAS. ENTRE DOS TRABAJADORES, REPETIR ÉSTA OPERACIÓN RECORRIENDO LA LONGITUD TOTAL --

DEL TUBO Y CON EL LÍQUIDO RETENIDO EN SU PARTE BAJA, BUSCAR SEÑALES DE GOTEO O HUMEDAD. REPARAR EN CASO NECESARIO CON EL EQUIPO DE REPARACIÓN QUE SE SUMINISTRA. REVISADO EL ESTADO DEL TUBO-SELLO, PROCEDER A COLOCARLO EN SU LUGAR DESCOLGANDO DE LOS TORNILLOS DEL PONTÓN LA BANDA DE DESGASTE, EN UNA LONGITUD DE 2.50 A 3.00 METROS, DEJANDO CAER EL TUBO SOBRE LA BANDA. DÓBLENSE LOS EXTREMOS DEL TUBO CON EL DOBLEZ HACIA EL PONTÓN Y FÍJENSE EN LOS TORNILLOS SOLDADOS DEL DOBLEZ (FIG. 5.2B). SUBIR Y ENGANCHAR LA BANDA. DESCOLGAR OTRO TRAMO DE IGUAL LONGITUD DEJANDO CAER EL TUBO, INMEDIATAMENTE SUBIR LA BANDA Y ENGANCHARLA EN LOS TORNILLOS. REPETIR ESTA OPERACIÓN EN TODA LA LONGITUD DEL TUBO. AL COLOCARLO, ASEGURARSE QUE SU COSTURA LONGITUDINAL QUEDE DEL LADO DEL PONTÓN Y DEBERÁN TOMARSE TODAS LAS PRECAUCIONES PARA NO LASTIMARLO CON LAS CUERDAS DE LOS TORNILLOS O LAS ARISTAS METÁLICAS.

10. DESPUÉS DE COLOCADO TODO EL TUBO Y LA BANDA DE DESGASTE ENGANCHADA EN SU LUGAR (NO ATORNILLADA) PROCEDER A LLENAR EL TUBO-SELLO INSERTANDO LA PUNTA DE LA MANGUERA DE LLENADO EN EL EXTREMO DEL TUBO-SELLO MÁS ALLÁ DEL DOBLEZ A 45° CON EL OTRO EXTREMO ENSARTADO Y SIN APRETAR LAS TUERCAS. VACIAR LÍQUIDO HASTA COMPLETAR LA PRIMERA CUARTA PARTE DE LA CANTIDAD TOTAL ESPECIFICADA. PARAR EL LLENADO, DESPRENDER LA BANDA DE DESGASTE DE TRES EN TRES TORNILLOS Y EFECTUAR UNA MINUCIOSA INSPECCIÓN DEL SELLO CERCIORANDOSE QUE EL TUBO ESTÉ LISO SIN ARRUGAS NI TORCEDURAS. NO DESPRENDER LA BANDA DE TRES TORNILLOS SI NO SE HAN ENGANCHADO LOS TRES ANTERIORES, SEGUIR LLENANDO HASTA COMPLETAR LA SEGUNDA CUARTA PARTE PARAR Y EFECTUAR UNA NUEVA REVISIÓN. CONTINUAR EN LA MISMA FORMA HASTA -

QUE EL TUBO ESTÉ COMPLETAMENTE LLENO CON LA CANTIDAD NORMAL QUE ES CASI SIEMPRE ALREDEDOR DEL 80% DE LA CANTIDAD TOTAL ESTIPULADA EN EL PLANO DE MONTAJE RESPECTIVO. ESTA CANTIDAD ES SUFICIENTE SI DESPUÉS DE APRETAR TODOS LOS TORNILLOS EXISTE UN CONTACTO HERMÉTICO CON LA ENVOLVENTE DEL TANQUE EN TODA LA PERIFÉRIA. ES PREFERIBLE USAR PETRÓLEO DIAFANO PARA LLENAR EL TUBO; PODRÍA USARSE AGUA EN CLIMAS NO FRÍOS Y AÚN EN REGIONES FRÍAS, PERO EN ESTE CASO DEBERÁ AGREGARSELE UNA SOLUCIÓN ANTI-CONGELANTE. NUNCA DEBERÁ EMPLEARSE AGUA SALADA.

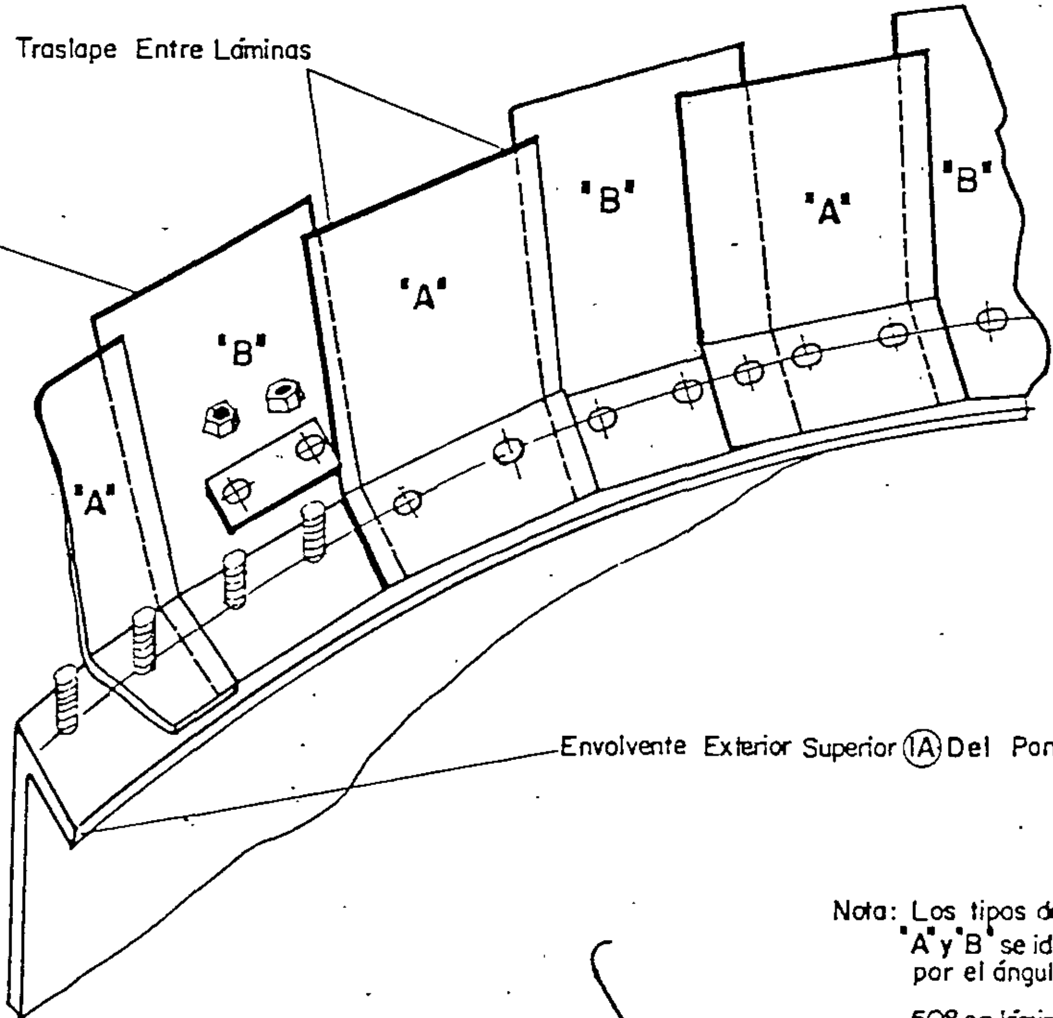
11. LA PARTE INFERIOR DE LA BANDA DE DESGASTE SERÁ INSPECCIONADA DESPUÉS DE 24 HORAS DE EFECTUADA LA OPERACIÓN DE LLENADO, PARA BUSCAR FUGAS O DISMINUCIÓN DE LA PRESIÓN DEL TUBO-SELLO. SI ÉSTO OCURRE, ES INDICACIÓN DE UNA FUGA EN EL TUBO; REINSPECCIONARLO PARA DESCUBRIR PUNTOS DE HUMEDAD QUE PUDIERAN EXISTIR Y REPARAR EN SU CASO.

12. INSTALAR LAS LÁMINAS DE PROTECCIÓN CONTRA LA LLUVIA EN EL ORDEN INDICADO EN EL PLANO DE MONTAJE. FIJARLAS CON LAS SOLERAS DE RETENCIÓN QUE PRESIONARÁN TAMBIÉN LA BANDA DE DESGASTE Y EL TUBO-SELLO. VÉASE LA FIGURA 5.2.C.

13. PARA CUALQUIER PONCHADURA O PIQUETE, QUE PUDIERE DESARROLLARSE EN EL TUBO, SE PUEDEN HACER LAS REPARACIONES DE ACUERDO CON LAS INSTRUCCIONES QUE SE ADJUNTAN EN CADA EQUIPO DE REPARACIÓN.

14. VERIFICAR NUEVAMENTE QUE TODA REBABA, BORDE, CORDÓN DE SOLDADURA, SALPICADURAS, ETC. EN LA PARTE INTERIOR DE LA ENVOLVENTE HAYA SIDO ALISADA TOTALMENTE TANTO ARRIBA DEL

DETALLE COLOCACION DE LAS LAMINAS DE PROTECCION



Nota: Los tipos de láminas "A" y "B" se identifican por el ángulo de doblez
50° en láminas tipo "A"
60° en láminas tipo "B"

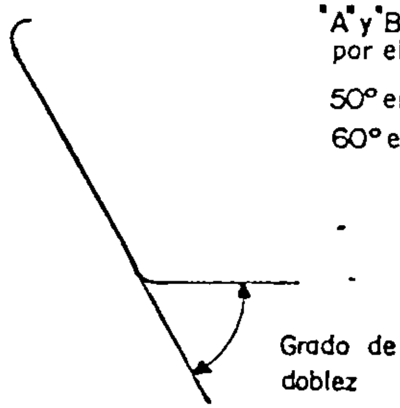


Fig. 5.2.C DETALLE DE COLOCACION DE LAMINAS DE PROTECCION TIPO "A" y "B".

SELLO COMO ABAJO DEL MISMO, YA QUE EN OPERACIÓN EL TECHO PUEDE ESTAR A SU ALTURA MÁXIMA COMO EN LA PARTE MÁS BAJA A SU NIVEL DE APOYO EN EL FONDO.

15. INSPECCIONAR CUIDADOSAMENTE QUE NO QUEDEN EN EL INTERIOR DEL TANQUE, HERRAMIENTA, ANDAMIOS, ETC. Y BARRER EL FONDO PARA DEJARLO LIMPIO.

16. CONECTAR Y PROBAR LA HERMETICIDAD DEL SISTEMA DE DRENAJE DEL DIAFRAGMA CON LA CUAL QUEDA TERMINADO EL MONTAJE DEL TECHO FLOTANTE.

17. SE PREPARA ENSEGUIDA LA PRUEBA DE FLOTACIÓN LLENANDO EL TANQUE CON AGUA HASTA DESBORDARSE. DURANTE EL PRIMER CICLO DE RECORRIDO DEL TECHO HASTA EL NIVEL SUPERIOR Y REGRESO AL FONDO, OBSERVAR CUIDADOSAMENTE LA CARA SUPERIOR DEL DIAFRAGMA Y EL INTERIOR DEL PONTÓN Y BOYAS PARA DETERMINAR SI HAY FUGAS. DURANTE LA TRAVESÍA HACIA ARRIBA PUEDE SUCEDER QUE SE LIBERE ALGO DEL LÍQUIDO DEL TUBO-SELLO PARA LO CUAL SE HAN DEJADO LOS EXTREMOS SIN PRENSAR. CONCLUIDO EL RECORRIDO, SE COLOCARÁN LAS PLACAS DE PROTECCIÓN FALTANTES Y LAS BARRAS DE FIJACIÓN DEL TUBO-SELLO.

6.0 ACCESORIOS.

6.1 INSTALACIÓN DE ACCESORIOS.

REGISTROS DE HOMBRE, BOQUILLAS Y OTROS ACCESORIOS DEBERÁN INSTALARSE Y SOLDARSE APROPIADAMENTE PARA IMPEDIR LA FORMACIÓN DE GRIETAS. AÚN PEQUEÑAS GRIETAS, CUANDO ESTÁN SUJETAS A ESFUERZOS ALTOS, PUEDEN EXTENDERSE EN LA ENVOLVENTE DEL TANQUE CAUSANDO FALLAS DESASTROSAS.

LOS CORTES EN LA ENVOLVENTE PARA LA ENTRADA DE LAS BOQUILLAS DEBERÁN HACERSE CON EXACTITUD. LA PERIFÉRIA DE LA ABERTURA DEBE ESTAR LISA Y LIBRE DE CORTADURAS, DE BORDES O CANTOS ÁSPEROS Y ESQUINAS CON FILOS. TODA LA ESCORIA, REBABAS O RECORTES DEBERÁN REMOVERSE ANTES DE SOLDAR Y LAS ESQUINAS REDONDEADAS CON ESMERIL. SIEMPRE QUE SEA POSIBLE, NO DEBERÁN DISEÑARSE --

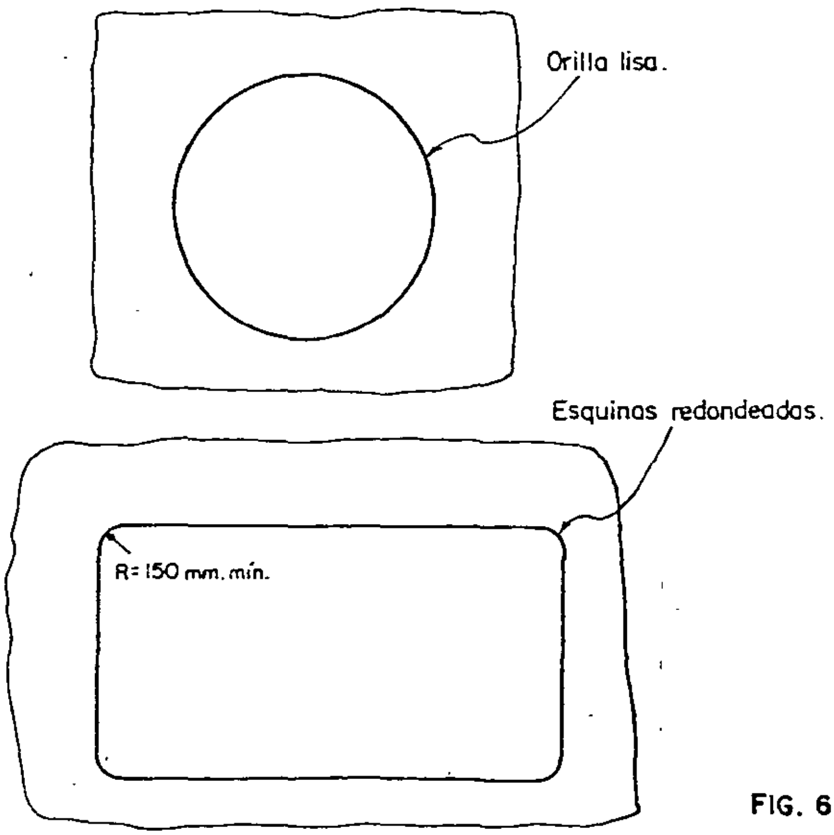


FIG. 6.1

ENTRADAS RECTANGULARES O CUADRADAS. CUANDO SEA NECESARIO HACERLO, LAS ESQUINAS DEBERÁN REDONDEARSE CON UN RADIO NO MENOR DE 150 MM. VÉASE LA FIG. 6.1. LAS BOQUILLAS TAMBIÉN DEBEN ESTAR BIEN ACABADAS CON ESQUINAS ESMERILADAS, BIEN ALISADAS, LIBRES DE GRIETAS Y RECORTES. TODAS ÉSTAS PREPARACIONES DEBERÁN HACERSE ANTES DE INICIAR LA SOLDADURA DE LAS BOQUILLAS.

6.1.1 LOCALIZACIÓN DE ACCESORIOS. USAR EL ESQUEMA DE LOCALIZACIÓN DE BOQUILLAS Y REGISTROS NORMALMENTE REFERIDO AL NORTE CONSTRUCTIVO. EN CASOS ESPECIALES PUEDE REQUERIRSE LOCALIZAR UNA BOQUILLA EN EL CAMPO A PETICIÓN DEL USUARIO Y POR UNA CONDICIÓN ESPECIAL. PARA CUMPLIR CON LOS REQUERIMIENTOS MÍNIMOS DE DISTANCIAS DE LOS CORDONES DE SOLDADURA ENTRE REFUERZOS Y DEL FONDO AL CORDÓN DE LA BOQUILLA MÁS BAJA CONSULTAR EL API 650 SECCIÓN 3.7.3 O VÉANSE LAS FIGURAS 6.1.1A, 6.1.1B, 6.1.1C Y 6.1.1D.

SIEMPRE HABRÁ QUE NOTIFICAR A INGENIERÍA DE DISEÑO A TRAVÉS DE LA SUPTCIA. LOCAL DE CONSTRUCCIÓN, CUANDO POR NECESIDAD DE SERVICIO, SE HAGAN MODIFICACIONES EN EL CAMPO AL DISEÑO ORIGINAL DEL TANQUE. INGENIERÍA DEBERÁ REVISAR LOS PLANOS ORIGINALES Y PONERLOS AL DÍA PARA INCLUIR LAS MODIFICACIONES O CAMBIOS HECHOS POR CONSTRUCCIÓN.

6.2 SOLDEO DE ACCESORIOS.

EL PROCESO DE SOLDEO EN LA PERIFÉRIA DE LAS ABERTURAS DE ENTRADA DE LAS BOQUILLAS CREA ESFUERZOS DE CONTRACCIÓN, LOS CUALES PUEDEN SER MEJOR CONTROLADOS USANDO EL PROCEDIMIENTO EN "CASCADA" EN UNA SECUENCIA APROPIADA.

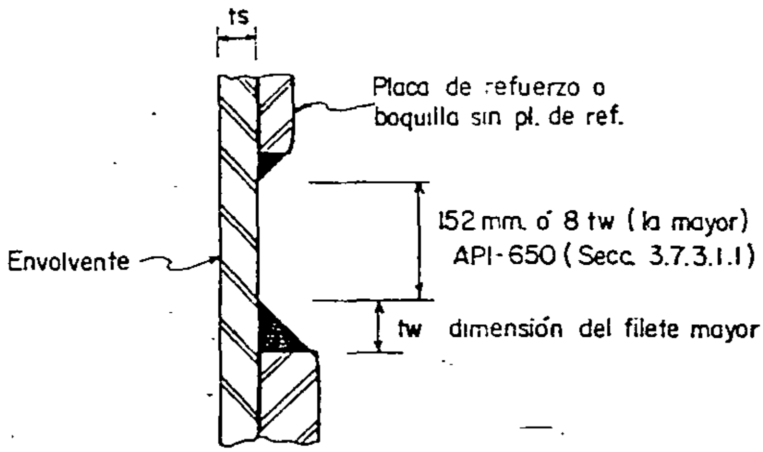


FIG. 6.1.1a.- Espaciamiento mínimo entre accesorios en la envolvente.

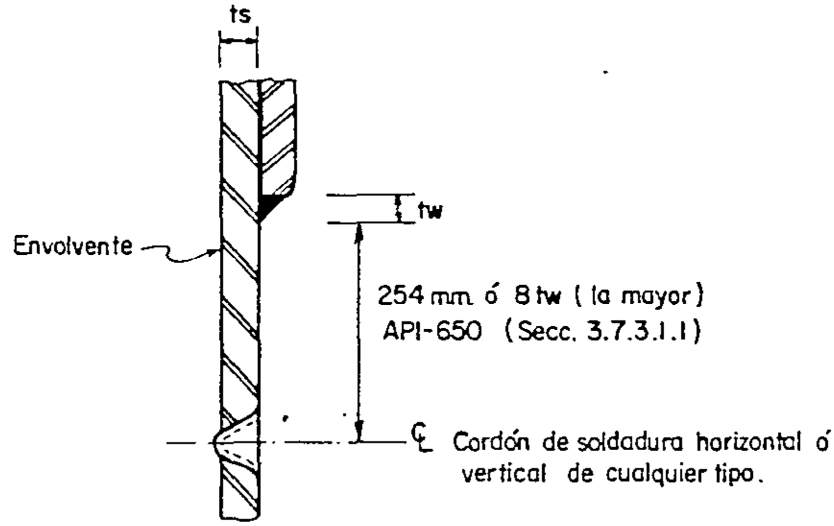


FIG. 6.1.1b.- Espaciamiento mínimo entre accesorios de la envolvente y cordones de soldaduras a tope de cualquier tipo.

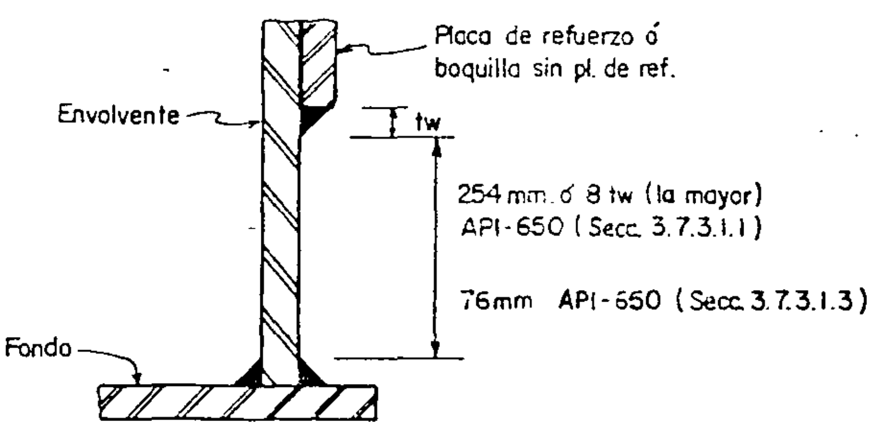


FIG. 6.1.1c.- Espaciamiento mínimo entre accesorios de la envolvente y el fondo.

TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR : Ing. I. J. L. / APROBADO POR : Ing. J. H. B.	FECHA IV-86	HOJA 4 DE 7
SECCION 6.0 INSTALACION DE ACCESORIOS	MANUAL DE MONTAJE Nº 1		

"A" es soldado primero

"B" es soldado despues

"C" es soldado al ultimo

use el metodo de cascada (6.2 b)

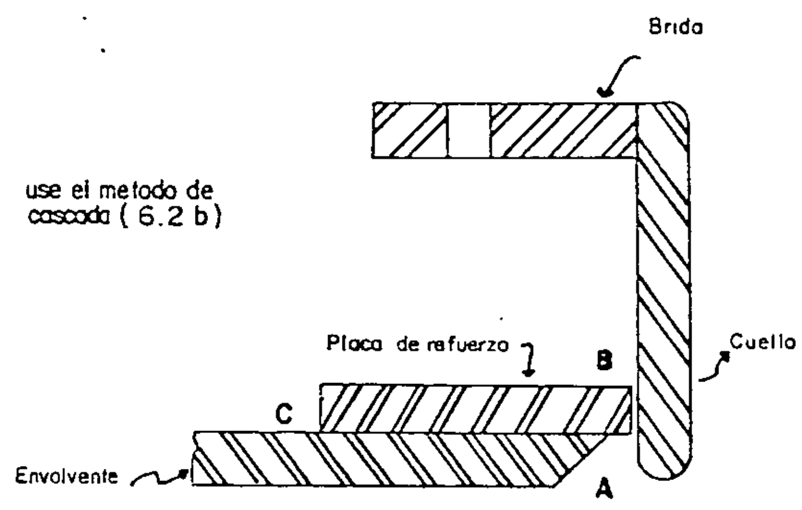


Fig. 6.2a Secuencia de soldeo en una boquilla

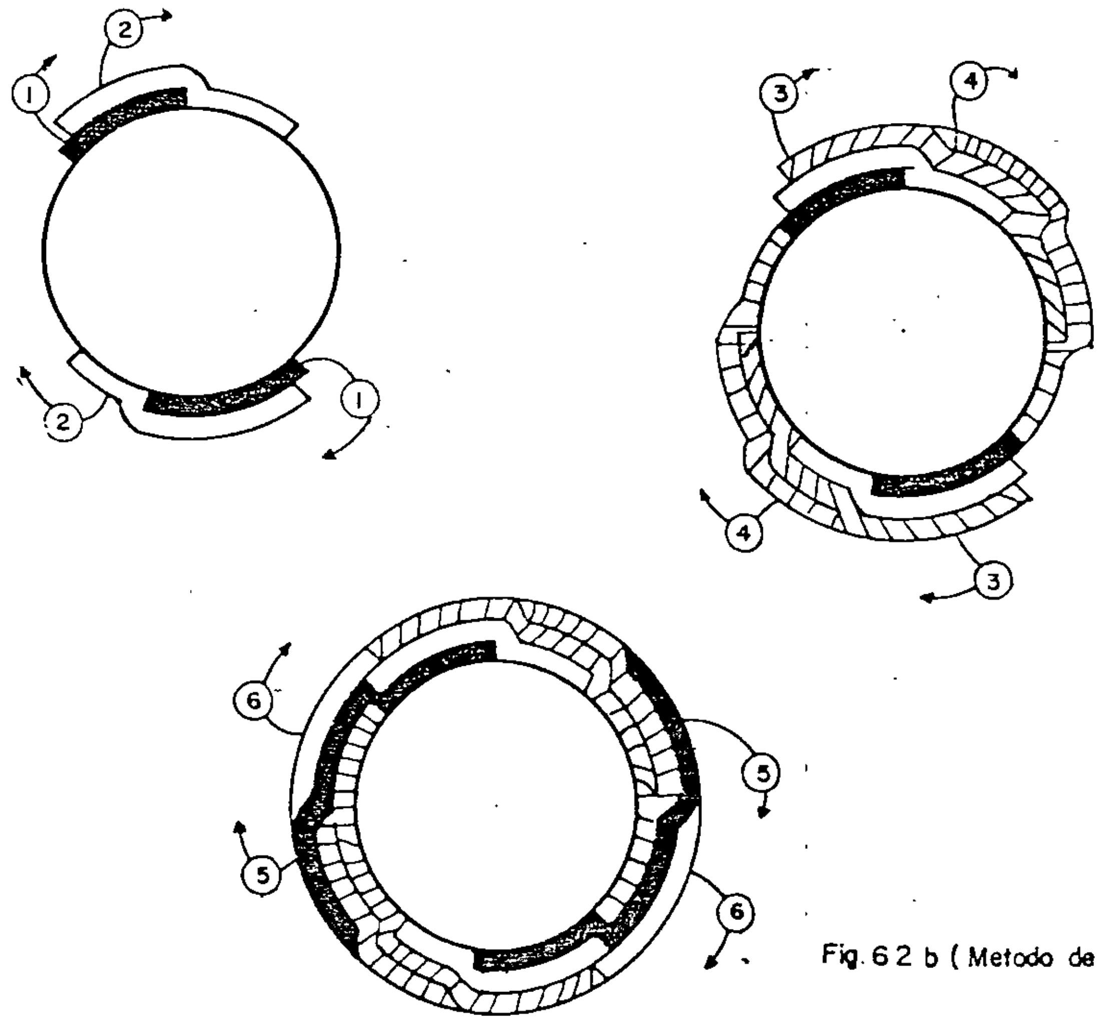


Fig. 6 2 b (Metodo de Cascada)

La fig. 6.2a muestra un detalle seccional de una boquilla con las soldaduras marcadas A, B y C. Siempre hágase primero la - soldadura A, luego la B y finalmente la C.

La fig. 6.2.b ilustra el método de soldeo en "cascada". Se - - piensa que al aplicarlo se sostiene el accesorio caliente durante el soldeo. No martillar la primera y la última capa pero si se permite en las capas intermedias para evitar grietas y distorsiones. Después de iniciar el soldeo alrededor de - - cualquier entrada, debe continuarse sin interrupción hasta -- que toda la soldadura es completada y mientras el área está - aún caliente.

Siempre que las condiciones de viento y la temperatura del medio ambiente son tan severas que el método en "cascada" no -- mantiene la boquilla caliente, la placa de la envolvente, la de refuerzo y el cuello de la boquilla deberán pre-calentarse a 40°C y sostener ésta temperatura hasta que la boquilla se - ha soldado totalmente. No depositar cantidades excesivas de - electrodo fundido. Las soldaduras de filete deberán ser de -- las dimensiones especificadas en los planos de diseño.

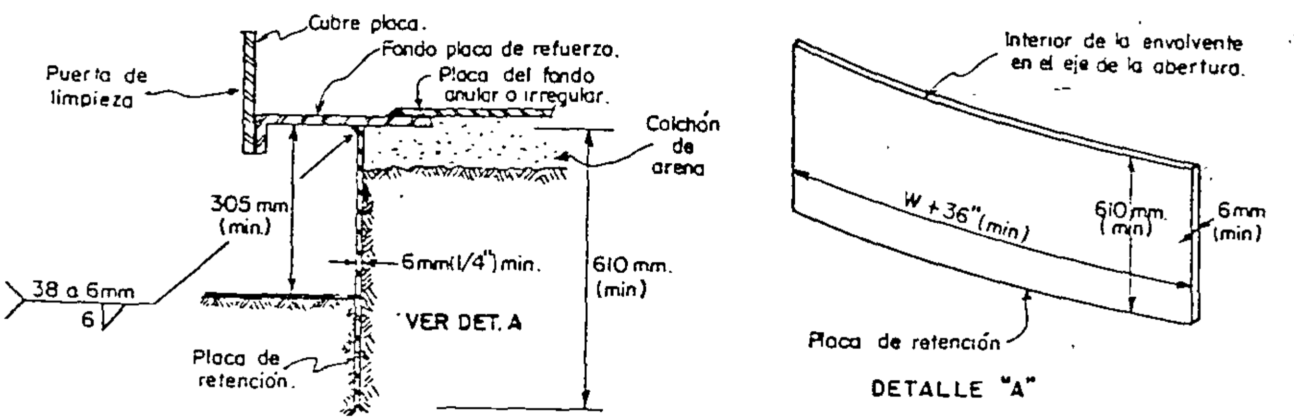
6.3 Pruebas en las placas de refuerzo.

Hasta completar la fabricación y antes de llenar el tanque -- con el agua de la prueba hidrostática, las placas de refuerzo - en cada boquilla serán probadas aplicando hasta 15 lb/pulg2 (1.1 kg/cm2) de presión manométrica con aire entre la envolvente del tanque y la placa de refuerzo usando un agujero de prueba de 6 mm. (1/4") de diámetro hecho en la placa de refuerzo. Al mismo tiempo que a cada boquilla se le aplica tal presión, una película de jabonadura, aceite de linaza u otro

MATERIAL APROPIADO PARA DESCUBRIR FUGAS, SE APLICARÁ A TODA LA SOLDADURA ALREDEDOR DEL REFUERZO TANTO ADENTRO COMO AFUERA DEL TANQUE. SI SE DESCUBRE CUALQUIER FUGA, RELEVAR LA PRESIÓN DEL AIRE, REMOVER LA SOLDADURA DEFECTUOSA CON CINCEL O ARCO-AIRE, REPARARLA Y VOLVER A PROBAR. AL TERMINAR LA PRUEBA, RETIRAR EL EQUIPO Y DEJAR EL AGUJERO ABIERTO A LA ATMÓSFERA.

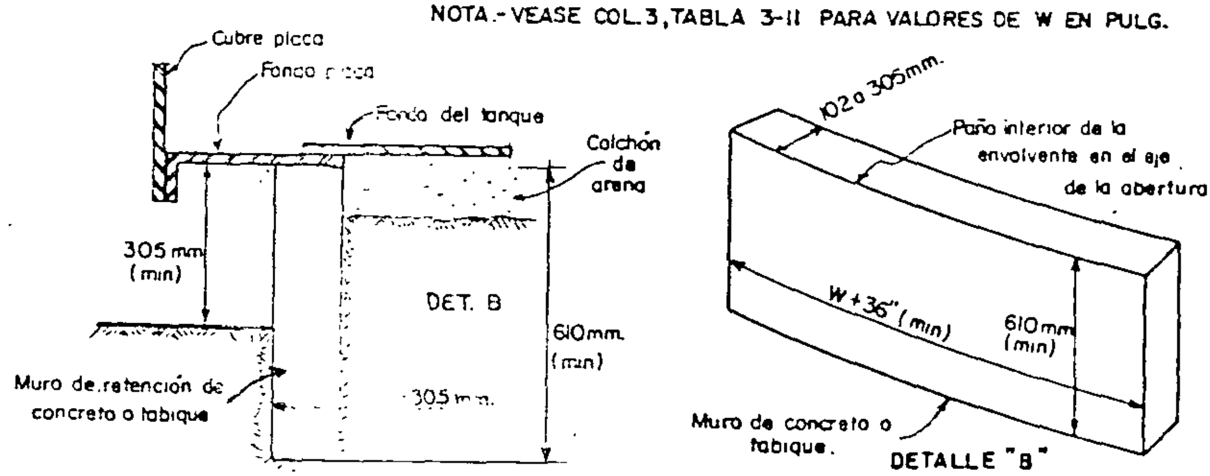
6.4 INSTALACIÓN DE LAS PUERTAS DE LIMPIEZA.

DE ACUERDO AL TIPO DE CIMENTACIÓN DEL TANQUE, SE HACEN LOS ARREGLOS PARA LA INSTALACIÓN CORRECTA DE LAS PUERTAS DE LIMPIEZA. EN MÉXICO, NORMALMENTE SE PROYECTA LA CIMENTACIÓN A -



METODO A.-Placa de retención para tanques apoyados en cimentación de piedra o grava:

NOTA.-VEASE COL.3, TABLA 3-II PARA VALORES DE W EN PULG.



METODO B.-Muro de concreto o tabique para tanques apoyado en cimentacion de piedra o grava.

FIG. 6.4 Arreglo en cimentaciónes de piedra o grava para la instalacion correcta de puertas

BASE DE ANILLOS DE CONCRETO (SECCIÓN 1.0, PÁRRAFO 1.3 DEL MANUAL) AUNQUE TAMBIÉN DEBIERA USARSE EN TERRENOS RESISTENTES - LA CONSTRUIDA CON GRAVA O PIEDRA TRITURADA. EN EL PRIMER CASO, INGENIERÍA DE DISEÑO HA ELABORADO UN PLANO CON TODAS LAS INDICACIONES PARA LA INSTALACIÓN CORRECTA DE LAS PUERTAS Y - EN EL CASO DE CIMENTACIÓN A BASE DE PIEDRA O GRAVA TRITURADAS, EL API-650 SECCIÓN 3.7.7 FIGURA 3.9 DA LAS INDICACIONES NECESARIAS PARA LA INSTALACIÓN DE LAS PUERTAS. SIN EMBARGO PUEDE SEGUIRSE LA SIGUIENTE SECUENCIA DE MONTAJE, MÉTODOS A Ó B -- (FIGURA 6.4)

1. LOCALIZAR LA POSICIÓN DE LAS PUERTAS ANTES DE TENDER EL FONDO.

2. HACER LA EXCAVACIÓN PARA INSTALAR UNA PLACA DE RETENCIÓN (MÉTODO A) O CONSTRUIR UN MURO DE RETENCIÓN DE CONCRETO O TABIQUE (MÉTODO B).

3. COLOCAR LA PLACA DE RETENCIÓN O FABRICAR EL MURO A LA ALTURA, AL RADIO EXACTOS Y SIMÉTRICA CON RESPECTO A LA ENTRADA DE LA PUERTA (VÉASE LA FIG. 6.4).

4. REEMPLAZAR LA TIERRA POR LA PARTE INTERIOR DE LA PLACA O DEL MURO, RELLENANDO EL HUECO CON UN COLCHÓN DE ARENA AL RAZ DE LA PLACA DE BASE DE LA PUERTA. COMPACTAR CONCIENZUDAMENTE EL RELLENO DE TIERRA Y ARENA ANTES DE CUBRIRLO CON LAS PLACAS DEL FONDO DEL TANQUE.

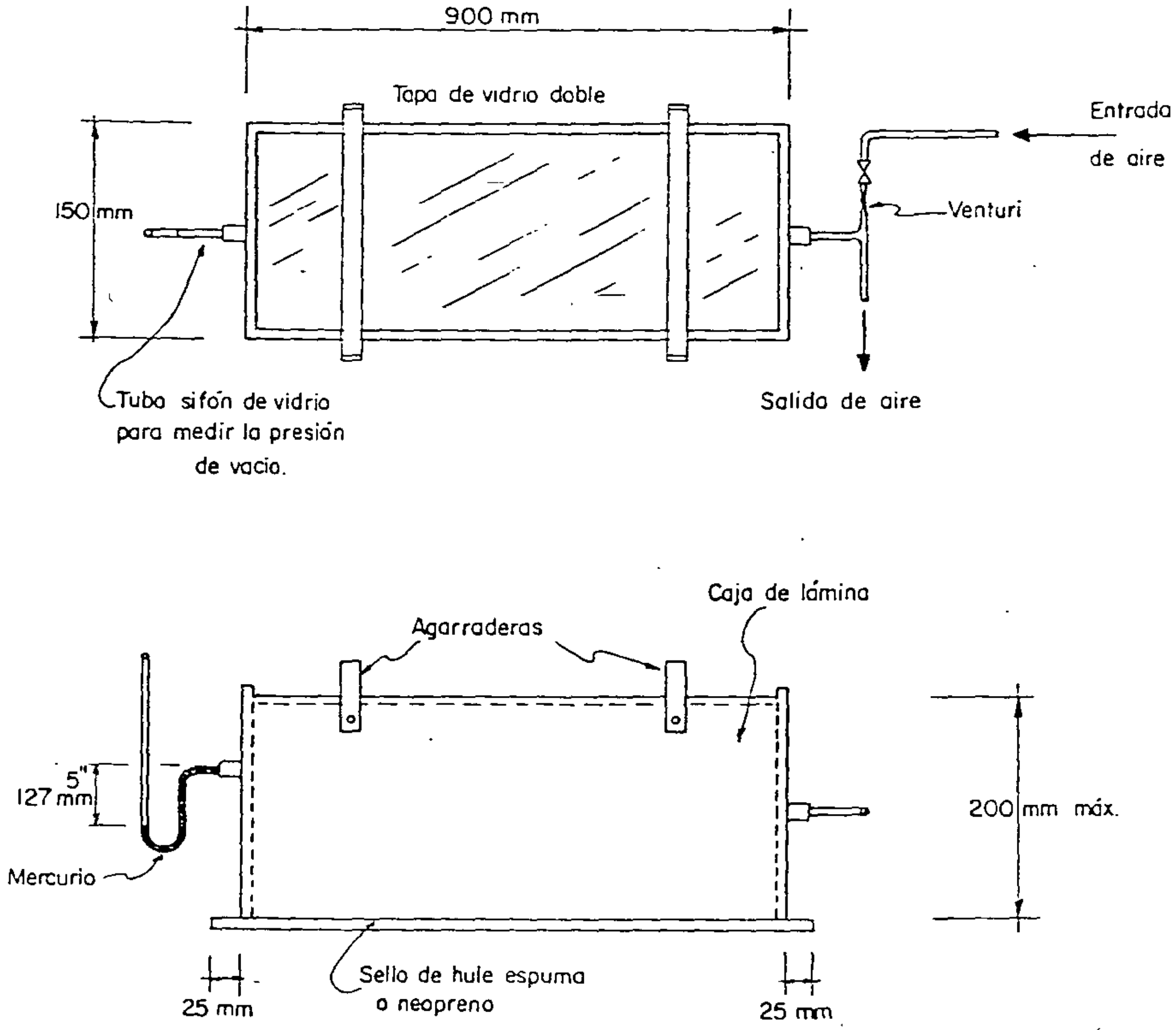
5. TENDER LAS PLACAS DEL FONDO Y HACER EL CORTE PARA LA ENTRADA DE LAS PLACAS DE LAS PUERTAS. MONTAR EL PRIMER ANILLO DE LA ENVOLVENTE DEL TANQUE, EMPEZANDO CON LAS PLACAS DE LAS PUERTAS DE LIMPIEZA.

7.0 PRUEBAS, INSPECCIONES, LIMPIEZA, PINTURA Y CONCLUSIONES.

LAS PRUEBAS INDICADAS EN ÉSTA SECCIÓN SE HARÁN CONFORME SE VAYA TERMINANDO LA ERECCIÓN DE LAS DIFERENTES PARTES. PARA GARANTIZAR QUE SE ESTÁ DE ACUERDO CON LOS REQUERIMIENTOS DEL CÓDIGO API-650 SECCIÓN 5.3, LA CÍA. CONTRATISTA DESARROLLARÁ LOS PROCEDIMIENTOS NECESARIOS PARA LLEVAR A CABO DICHAS PRUEBAS A MEDIDA QUE SE VAYA REQUIRIENDO Y HACERLAS DE ACUERDO CON LOS ESTANDARES DEL API.

7.1 INSPECCIÓN DE SOLDADURAS DEL FONDO Y TECHO DEL TANQUE.

UN PROCEDIMIENTO EFICAZ PARA INSPECCIONAR LOS CORDONES DE SOLDADURA DE FONDOS Y APROBADO POR API, ES MEDIANTE LA PRUEBA DE VACÍO HECHA POR MEDIO DE UNA CAJA DE METAL DE 150 MM. DE ANCHO Y 900 MM. DE LARGO (FIG. 7.1) CON UNA TAPA DE DOBLE CRISTAL Y EL FONDO ABIERTO EL CUAL ES SELLADO CONTRA LA SUPERFICIE DEL FONDO DEL TANQUE CON UN EMPAQUE DE NEOPRENO O DE HULE ESPUMA. LA CAJA TIENE ADEMÁS UNA CONEXIÓN DE TUBO APROPIADO, VÁLVULA Y UN TUBO SIFÓN PARA MEDIR EL VACÍO (FIG.7.1). APROXIMADAMENTE 900 MM. DE LA SOLDADURA POR PROBARSE ES MOJADA CON UNA SOLUCIÓN DE JABONADURA O ACEITE DE LINAZA. (EN TEMPERATURA AMBIENTAL MUY FRÍA ES NECESARIO AGREGAR UNA SOLUCIÓN ANTICONGELANTE). SE COLOCA LA CAJA SOBRE EL CORDÓN ENJABONADO Y SE ORIGINA UN VACÍO. LA PRESENCIA DE POROSIDAD O FUGAS EN LA COSTURA ES INDICADA POR BURBUJAS O ESPUMA PRODUCIDAS POR AIRE SUCCIONADO A TRAVÉS DEL CORDÓN DE SOLDADURA. EL VACÍO EN LA CAJA SE OBTIENE CONECTANDO UN COMPRESOR DE 7 KG. COMO MÁXIMO CON UNA MANGUERA DE LARGO SUFICIENTE PARA CUBRIR TODO EL FONDO, O CONECTANDO LA CAJA AL MÚLTIPLE DE ADMISIÓN DE UN MOTOR DE GASOLINA O DIESEL A UN EYECTOR DE AIRE O UNA BOMBA ESPECIAL DE VACÍO.



5" de mercurio = 2.46 lb/pulg.^{2} = 0.173 Kg/cm.^{2}

FIG. 7.1 Caja metálica para pruebas de vacio.

La columna de mercurio del tubo curvado debe registrar 5 pulgadas para obtener un vacío parcial de 2.5 lbs/pulg2 (173 gr/cm2). suficiente para proteger el sello de la base. Al cerrar la válvula de aire y quedar la caja sin vacío, sola vuelve a su posición normal y así poder seguir con la prueba hasta completar el fondo. Se ha venido generalizando el uso de la caja de vacío en virtud de que el resultado de la prueba es muy seguro y con respecto a otros medios empleados, resulta sumamente económico.

En la misma forma que se prueban las costuras traslapadas del fondo, con la caja de vacío, se probará la soldadura del diafragma. Si se descubren porosidades o fugas, reparar de inmediato. Asimísmo la soldadura en el fondo/envolvente en el primer anillo, será probada con líquido penetrante después de soldar el cordón exterior. Rociar petróleo diáfano (Kerosene) por la junta interior antes de soldarla. Despúes que todas las fugas de la soldadura externa han sido reparadas, podrá soldarse el cordón interior.

7.2 Pruebas en el fondo y boyas.

Después de terminadas las soldaduras en todo el diafragma, y pontones y la de las boyas al difragma incluyendo sus placas de refuerzo, se les hará una inspección visual y luego serán probadas con líquidos penetrantes rociando abundante petróleo diáfano por el interior del techo. Después de un periódo de 24 horas, las superficies superiores de estas áreas soldadas serán revisadas para descubrir fugas. Todas las conexiones así como registros y boquillas en el techo también serán probadas con líquido desde la parte interna del diagrama.

7.3 ENVOLVENTE DEL TANQUE Y PRUEBA DE FLOTACIÓN.

LA ENVOLVENTE SE PRUEBA LLENANDO EL TANQUE A SU CAPACIDAD NORMAL, CON AGUA. CUANDO SE EMPIEZA LA PRUEBA HIDROSTÁTICA, TAN PRONTO COMO EL TECHO EMPIEZA A FLOTAR, SE INTERRUMPE EL LLENADO Y SE HACE UNA REVISIÓN EXHAUSTIVA DEL DIAFRAGMA Y PONTÓN. CONTINUAR CON EL LLENADO DEL TANQUE Y MIENTRAS EL TECHO ESTÁ SUBIENDO, REVISAR EL TUBO-SELLO, LA LÁMINA DE PROTECCIÓN Y LA ESCALERA RODANTE. CUANDO EL TANQUE SE VACÍA Y EL TECHO BAJA, SE SEGUIRÁ LA INSPECCIÓN EN LA MISMA FORMA QUE SE HIZO CUANDO EL TECHO IBA HACIA ARRIBA. TAMBIÉN SE HARÁN INSPECCIONES PERIÓDICAS DEL PONTÓN MIENTRAS EL DIAFRAGMA SUBE Y BAJA. ESTA INSPECCIÓN ES MUY IMPORTANTE PORQUE MUCHAS VECES PUEDEN OCURRIR FUGAS DURANTE EL MOVIMIENTO DEL DIAFRAGMA.

UNA ATENCIÓN ESPECIAL DEBERÁ DARSE TAMBIÉN A LA ESCALERA RODANTE PORQUE CUALQUIER EXCESO DE LIGADURAS PUEDE CAUSAR MÁS TARDE UN GRAN DAÑO O AVERÍA.

CUANDO SE TIENE EL TANQUE LLENO DE AGUA SE DEBE HACER UNA REVISIÓN OCULAR DE LAS SOLDADURAS POR SI SE DESCUBRE ALGUNA FUGA. ESTA PERMITIDO GOLPEAR CON UN MARTILLO DE BOLA LAS SOLDADURAS ESPECIALMENTE LOS CRUCES PARA EL MISMO OBJETO.

TAMBIÉN ES MUY IMPORTANTE LA REVISIÓN QUE SE HACE A LA CIMENTACIÓN, MIENTRAS SE LLEVA A CABO LA PRUEBA HIDROSTÁTICA. SI HAY UN ASENTAMIENTO EXCESIVO EN CUALQUIER PUNTO, DEBERÁ TOMARSE UNA ACCIÓN CORRECTIVA ADECUADA.

7.4 INSPECCIÓN FINAL.

ANTES DE ENTREGAR EL TANQUE TOTALMENTE TERMINADO AL USUARIO,

EL MONTADOR JUNTAMENTE CON EL RESIDENTE DE LA CONTRATISTA Y EL SUPERVISOR DE PEMEX, HARÁN UNA AMPLIA REVISIÓN FINAL AL TRABAJO HECHO PARA CONFIRMAR QUE ESTÁ COMPLETO Y POR ENCIMA DE LA CALIDAD REQUERIDA. LA SIGUIENTE ES LA MÍNIMA INSPECCIÓN REQUERIDA:

1. FONDO:

   A. REVISAR, BUSCANDO JUNTAS SIN SOLDAR, SOLDADURAS DE MENOR DIMENSIÓN, SOLDADURAS DEFECTUOSAS Y SOCAVACIONES.

   B. BARRER TODO EL FONDO PARA DEJARLO LIMPIO Y REVISAR SI SE DESCUBREN SALIENTES, REBABAS Y MELLAS O MUESCAS DONDE CANALES O MÉNSULAS PUDIERAN HABERSE DESPRENDIDO.

   C. REMOVER TODOS LOS SALIENTES Y REBABAS.

   D. REMOVER LA ESCORIA DE TODAS LAS SOLDADURAS.

   E. REPARAR LOS SOCAVADOS Y MUESCAS.

2. FONDO AL PRIMER ANILLO:

   A. REMOVER LA ESCORIA DE TODA LA SOLDADURA DE FILETE INTERIOR Y EXTERIOR.

   B. REVISAR PARA LOCALIZAR SOLDADURAS DE MENOR DIMENSIÓN, SOCAVADOS Y JUNTAS NO SOLDADAS.

   C. TODAS LAS REBABAS SERÁN REMOVIDAS DE LA INTERSECCIÓN DE PLACAS.

3. ENVOLVENTES:

A. TODOS LOS SALIENTES O CONEXIONES A ANDAMIAJES SERÁN - - REMOVIDOS Y RESANADOS.

B. LAS REBABAS SE QUITARÁN CON CINCEL, LOS SOCAVADOS, RE-- LLENADOS Y LUEGO ESMERILADOS. DEBERÁ DARSE ESPECIAL - - ATENCIÓN A LAS ÁREAS ALREDEDOR DE LAS ESCALERAS.

C. TODAS LAS SOLDADURAS VERTICALES Y HORIZONTALES SERÁN -- INSPECCIONADAS PARA DESCUBRIR SOCAVADOS Y QUE LOS REFUERZOS Y POROSIDADES ESTÉN DENTRO DE LAS TOLERANCIAS ESPECIFICADAS.

4. TRABES DE REFUERZO Y ÁNGULOS DE CORONAMIENTO:

A. EN LA MISMA FORMA QUE EN EL FONDO Y ENVOLVENTE, REVISAR TODAS LAS SOLDADURAS LOCALIZANDO SOCAVADOS, POROSIDADES, CORDONES DE MENOR DIMENSIÓN Y ÁREAS SIN SOLDAR.

B. LAS SOLDADURAS A TOPE EN LA TRABE DE REFUERZO Y ÁNGULO DE CORONAMIENTO SE REVISARÁN PARA QUE LA JUNTA TENGA PENETRACIÓN COMPLETA Y SEA DE LA MISMA CALIDAD QUE LAS -- VERTICALES DE LA ENVOLVENTE.

C. LAS SOLDADURAS HORIZONTALES SE REVISARÁN PARA CERCIORARSE QUE SON DE LA MISMA CALIDAD QUE LAS HORIZONTALES DE - LA ENVOLVENTE.

5. ACCESORIOS:

A. TODAS LAS SOLDADURAS SERÁN DEL TAMAÑO INDICADO EN PLANOS Y SIN SOCAVADOS.

B. LOS AGUJEROS DE ENTRADA Y LAS CARAS DE TODAS LAS BRIDAS SE REVISARÁN PARA QUE ESTÉN DE ACUERDO A LOS PLANOS RESPECTIVOS.

C. ASEGURARSE QUE TODOS LOS REFUERZOS HAN SIDO PROBADOS.

D. LAS BRIDAS CIEGAS, TAPAS DE REGISTROS DE HOMBRE, PERNOS Y EMPAQUES DEBERÁN INSTALARLAS APROPIADAMENTE.

E. LAS REBABAS ALREDEDOR DE LAS BOQUILLAS SERÁN REMOVIDAS.

6. ESCALERAS Y ESCALAS:

A. LAS ESCALERAS SERÁN REVISADAS PARA UN CONTORNO APROPIADO, LAS HUELLAS A NIVEL, LOS BARANDALES A PLOMO Y TODAS LAS SOLDADURAS COMPLETAS SEGÚN DIBUJO.

B. LAS SOLDADURAS EN LOS PASAMANOS SERÁN ALISADAS CON ESMERIL. REVISAR ÉSTO.

C. LAS ESCALAS SE INSTALARÁN DERECHAS Y A PLOMO.

D. LAS PROTECCIONES EN LAS ESCALAS SE INSTALARÁN DERECHAS.

7.5 PINTURA INTERIOR Y EXTERIOR.

1. LA PINTURA EN LOS TANQUES SE APLICARÁ DE ACUERDO CON LAS ESPECIFICACIONES DE PEMEX. NORMAS 2.132.01, 3.134.01, 4.32.01 Y 5.132.01.

2. EL TRABAJO DE PINTURA INTERIOR SE INICIA CUANDO SE HA TERMINADO:

   A) ARMADO Y SOLDADO DEL FONDO.

   B) PRUEBA DEL FONDO.

   C). ARMADO Y SOLDADO DEL DIAFRAGMA DEL TECHO.

   D). ARMADO Y SOLDADO DE LA ENVOLVENTE.

   E). LIMPIEZA Y RESANE GENERAL DEL INTERIOR DEL TANQUE.

3. PINTURA INFERIOR DEL DIAFRAGMA, PONTÓN Y FONDO. ENVOLVENTE POR EL LADO INTERIOR (TODAS LAS SUPERFICIES METÁLICAS EN CONTACTO CON EL CRUDO). BOYAS.

   A). LA PINTURA SE APLICARÁ ANTES DE HACER LA PRUEBA DE FLOTABILIDAD E HIDROSTÁTICA DEL CUERPO DEL TANQUE.

   B). NO ARMAR EL SELLO Y LA BANDA DE DESGASTE ANTES DE HABER TERMINADO LA PINTURA INFERIOR Y SUPERIOR DEL DIAFRAGMA.

   C). LAS PLACAS DEL DIAFRAGMA, PREVIO AL TENDIDO LLEVARÁN APLICADA LA PINTURA QUE FIJA LA NORMA. LOS RESANES --

ORIGINADOS POR LA APLICACIÓN DE LA SOLDADURA SE HARÁN COMO COMPLEMENTO DE TERMINACIÓN.

4. PINTURA EXTERIOR DE LA ENVOLVENTE:

A). INICIAR ÉSTA ETAPA CUANDO SE HAYA TERMINADO DE APLICAR EN SU TOTALIDAD, LA PINTURA INTERIOR, LOS RESANES Y LA LIMPIEZA DE REBABAS DE SOLDADURA.

B). EL MONTAJE DEL TUBO-SELLO SE HARÁ SIMULTÁNEAMENTE CON LA ETAPA DE PINTURA EXTERIOR SIEMPRE QUE NO HAYA PROBLEMAS CON LA ARENA QUE ARRASTRA EL VIENTO, YA QUE ÉSTO DEFICULTA E IMPIDE LA APLICACIÓN DE PINTURA POR EL EXTERIOR DEL TANQUE.

7.6 CONCLUSIÓN:

DESPUÉS DE TERMINAR LAS REVISIONES E INSPECCIONES INDICADAS EN EL PÁRRAFO ANTERIOR, SE PROCEDE A ENTREGAR EL TANQUE TERMINADO Y PINTADO AL USUARIO, ES DECIR, AL PERSONAL AUTORIZADO DE LA OPERATIVA, SIGUIENDO LOS PROCEDIMIENTOS APROBADOS EN ÉSTOS CASOS.

EN LA EXPOSICIÓN DE LOS MÉTODOS DE MONTAJE DE LAS DIFERENTES PARTES QUE INTEGRAN UN TANQUE DE TECHO FLOTANTE, NO SE HA SEGUIDO UNA SECUENCIA DETERMINADA, ES DECIR, EL ORDEN EN QUE SE PRESENTA CADA SECCIÓN DEL MANUAL NO CONCUERDA CON EL QUE SE SIGUE REALMENTE EN EL MONTAJE DE UN TANQUE. ESTO QUE PUDIERA

ORIGINAR ALGUNA CONFUSIÓN ENTRE LAS DISTINTAS ETAPAS DEL MONTAJE CONSIGNADAS EN EL MANUAL Y EL TRABAJO REAL DE LA ERECCION DE UN TANQUE, MÁXIME QUE ALGUNAS OPERACIONES SON EJECUTADAS SIMULTÁNEAMENTE, QUEDA SUBSANADO MEDIANTE EL USO DEL DIAGRAMA QUE SE PRESENTE AL FINAL DE ESTE PÁRRAFO Y DONDE SE EXPONE UNA SECUENCIA DEL MONTAJE DESDE SU INICIO HASTA LA TERMINACIÓN Y ENTREGA DEL TANQUE, MOSTRANDO ADEMÁS AQUELLAS OPERACIONES QUE SE TRABAJAN PARALELAMENTE. ES CONVENIENTE TENER SIEMPRE A LA MANO, EN LA OBRA ESTE DIAGRAMA Y CONSULTARLO CUANTAS VECES SEA NECESARIO PARA SEGUIR SUS INDICACIONES Y ESTAR DE ACUERDO CON LOS PROGRAMAS DE ERECCIÓN DE LOS TANQUES.

AL TERMINAR EN ESTE PÁRRAFO EL MANUAL NO. 1 DE ERECCIÓN DE TANQUES DE TECHO FLOTANTE, PUEDE APARECER COMO INCOMPLETO POR FALTAR LAS SECCIONES NO MENOS IMPORTANTES DE PROCEDIMIENTO DE SOLDADURA Y PROCEDIMIENTOS DE INSPECCIÓN Y TOLERANCIAS ADMISIBLES. NO SE PUEDE USAR EL MANUAL SIN LA LECTURA DE ESTAS SECCIONES Y PARA TAL FIN, DADO QUE LOS CONCEPTOS VERTIDOS EN ELLAS, SON APLICABLES TANTO A TANQUES DE TECHO FLOTANTE COMO DE TECHO FIJO. (MANUAL DE ERECCIÓN NO. 2), SE TENDRÁN DOS SECCIONES COMPLEMENTARIAS: LA "A" CORRESPONDIENTE A LOS PROCEDIMIENTOS ESTANDAR DE SOLDADURA Y LA "B", PROCEDIMIENTOS DE INSPECCIÓN Y TOLERANCIAS ADMISIBLES. COMO SE ESPECIFICÓ YA, AMBAS SECCIONES PODRÁN USARSE EN TANQUES CON TECHO FLOTANTE O CON TECHO FIJO.

FINALMENTE, QUEDA ESTABLECIDO QUE EL USO DE ESTE MANUAL ES OBLIGATORIO EN LA ERECCIÓN DE TANQUES DE TECHO FLOTANTE Y PARA EVITAR CONFUSIONES QUE PUDIERAN PRESENTARSE, SE CANCELAN

TODOS LOS INSTRUCTIVOS DE MONTAJE Y RECOMENDACIONES AFINES QUE SE ENCUENTRAN EN USO A LA FECHA. SIN EMBARGO, SE TOMARÁN EN -- CUENTA TODAS AQUELLAS PROPOSICIONES QUE MODIFIQUEN MEJORANDO - EN CIERTO GRADO UNO O VARIOS DE LOS PROCEDIMIENTOS DE MONTAJE- EXPUESTOS EN EL MANUAL Y PREVIO ESTUDIO Y APROBACIÓN EN SU CA- SO POR LA COORDINACIÓN EJECUTIVA DE CONSTRUCCIÓN DE LA SPCO DE PEMEX, SE INCORPORARÁN AL MISMO.

HECHO POR:

ING. IGNACIO JARAMILLO LOPEZ
GCIA. DE PROYS. INDUSTRIALES
S.P.C.O.

APROBADO POR:

ING. JAIME HERNANDEZ BALBOA
COORD. EJECUTIVO DE CONST'N.
S.P.C.O.

FUENTES DE INFORMACION:

CHICAGO BRIDGE AND IRON CO. (CBI)

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KAWASAKI HEAVY INDUSTRIES LTD. (KHI)

AMERICAN PETROLEUM INSTITUTE. API-650 STD.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS. ASME V.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS. ASME IX.

AMERICAN WELDING SOCIETY (AWS)

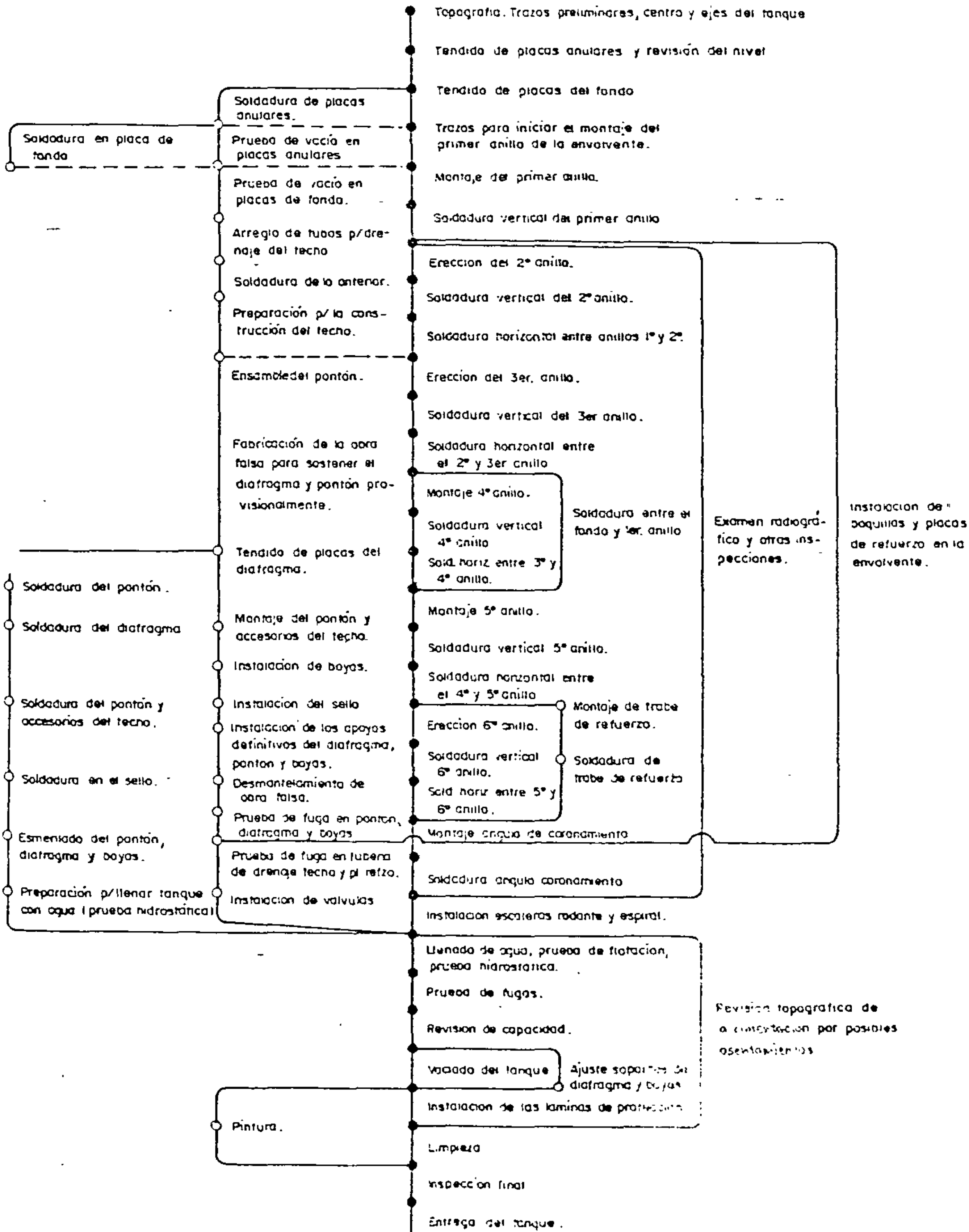
INSTRUCTIVO DE MONTAJE DE TANQUES DE T.F. DE 500 M. BLS. (PEMEX)

PROCEDIMIENTO DE MONTAJE PARA TANQUES DE 500 M. BLS. (CISA)

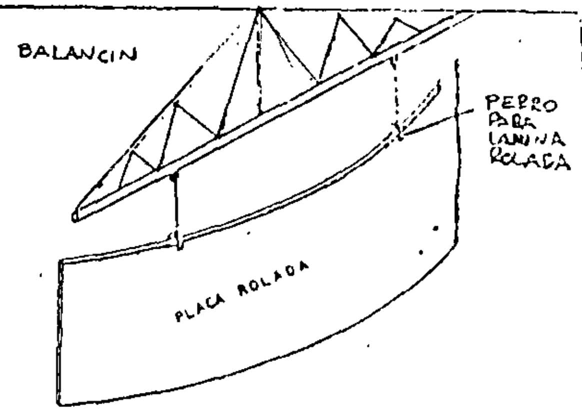
NOTAS Y ESPECIFICACIONES DE MONTAJE DE TANQUES (I.J.L.)

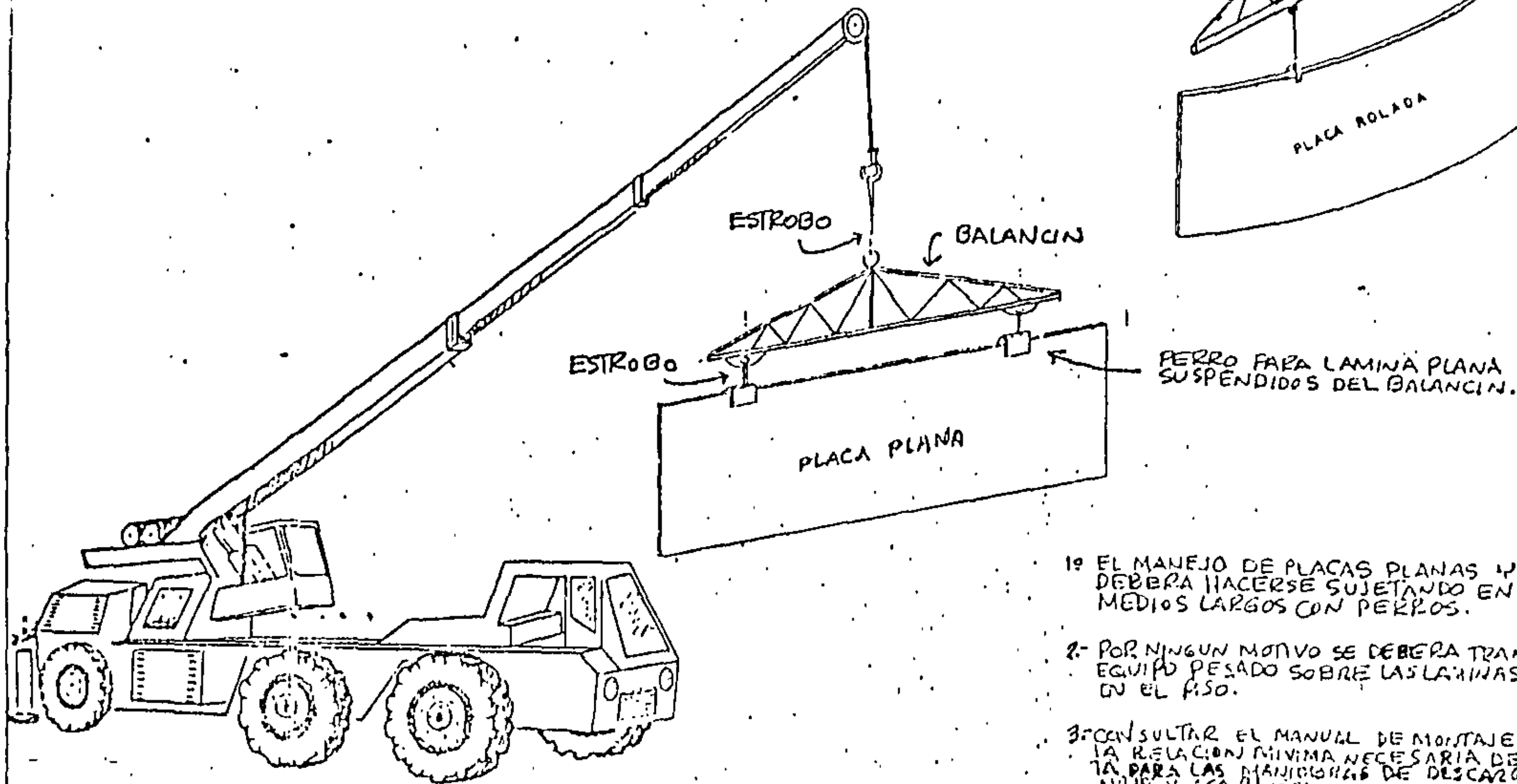
P. E. M. E. X.	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE.	HECHO POR Ing. I. J. L.	FECHA	HOJA
	APROBADO POR. Ing. J. H. B.	IV-86	13 DE 13
PRUEBAS, INSPECC. FINAL. CONCLUSIONES	MANUAL DE MONTAJE N° 1		

DIAGRAMA DE LA SECUENCIA DE LAS OPERACIONES DE MONTAJE DE UN TANQUE



MANEJO DE PLACAS



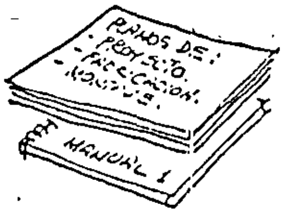
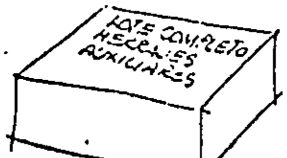
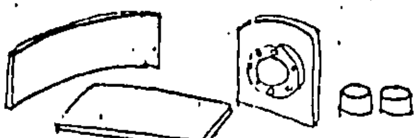


1º EL MANEJO DE PLACAS PLANAS Y ROLADAS DEBERA HACERSE SUJETANDO EN LOS TERCIOS MEDIOS LARGOS CON PERROS.

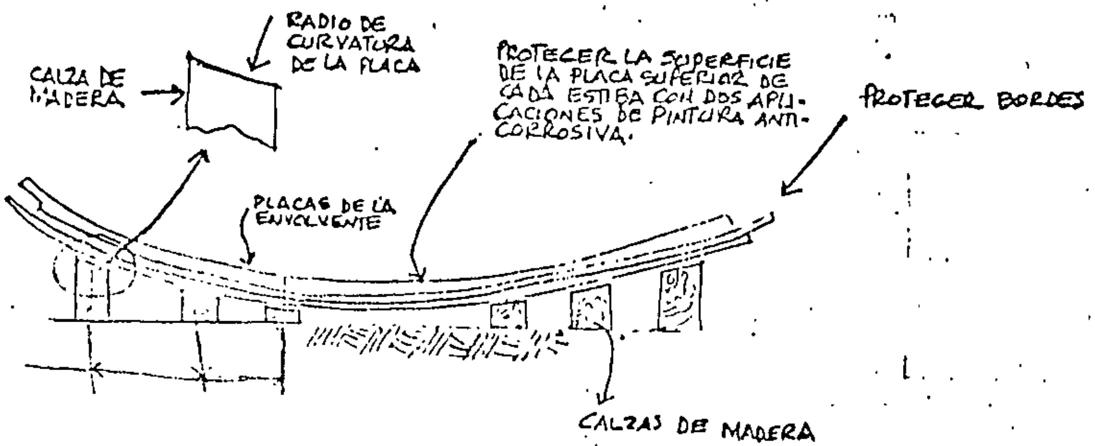
2.- POR NINGUN MOTIVO SE DEBERA TRANSITAR CON EQUIPO PESADO SOBRE LAS LAMINAS COLOCADAS EN EL PISO.

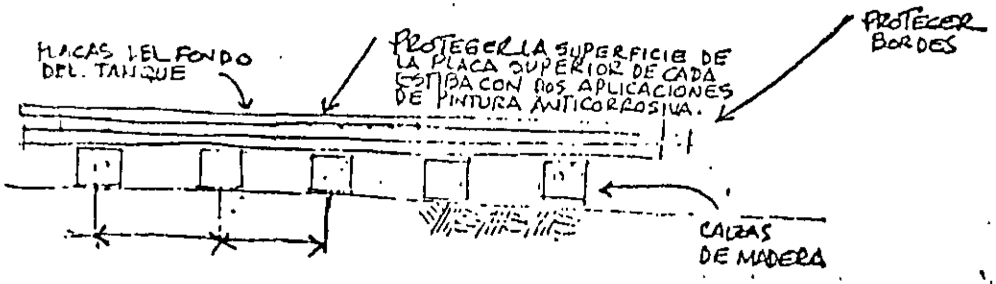
3.- CONSULTAR EL MANUAL DE MONTAJE No. 1 PARA LA RELACION MINIMA NECESARIA DE HERRAMIENTA PARA LAS MANIOBRAS DE DESCARGA, ALMACENAJE Y ACARREOS.

PEMEX I S.P.C.O COORDINACION EJECUTIVA DE CONSTRUCCION	
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE

	ACCION DE LA COMPAÑIA	SUPERVISION PEMEX
	EL RESIDENTE DE LA CONSTRUCTORA JUNTO CON SU MAESTRO MONTADOR Y EL SUPERVISOR DE PEMEX, DEBERAN REUNIRSE ANTES DE INICIAR LA CONSTRUCCION, PARA ESTUDIAR Y ANALIZAR CONCIENZUDAMENTE EL MANUAL Y LOS PLANOS.	DEBERA CANALIZAR LAS DUDAS Y LOS FALTANTES DE PROYECTO POR CONDUCTO DE LA SUPERINTENDENCIA LOCAL DE CONSTRUCCION.
	LA CIA. CONTRATISTA DEBERÁ EXHIBIR UN LOTE COMPLETO DE HERRAJES AUXILIARES	EL SUPERVISOR REVISARA EL LOTE DE ESTA HERRAMIENTA Y NO AUTORIZARA EL INICIO DE LA CONSTRUCCION SI NO ESTA COMPLETO.
	TOMAR NOTA	REVISAR Y REPORTAR LA LLEGADA DE LOS MATERIALES Y ACCESORIOS, ANOTANDO EN BITACORA EL ESTADO DE LOS MISMOS, PRINCIPALMENTE EL MATERIAL DE PLACAS DE FONDO, ENVOLVENTE Y TECHO. PASAR EL REPORTE A LA SUPCIA. LOCAL DE CONSTN.
	REVISAR EL MATERIAL DEL TANQUE QUE SE VA A MONTAR.	MISMA ACCION QUE LA COMPAÑIA.

INICIAR ERECCION DEL TK

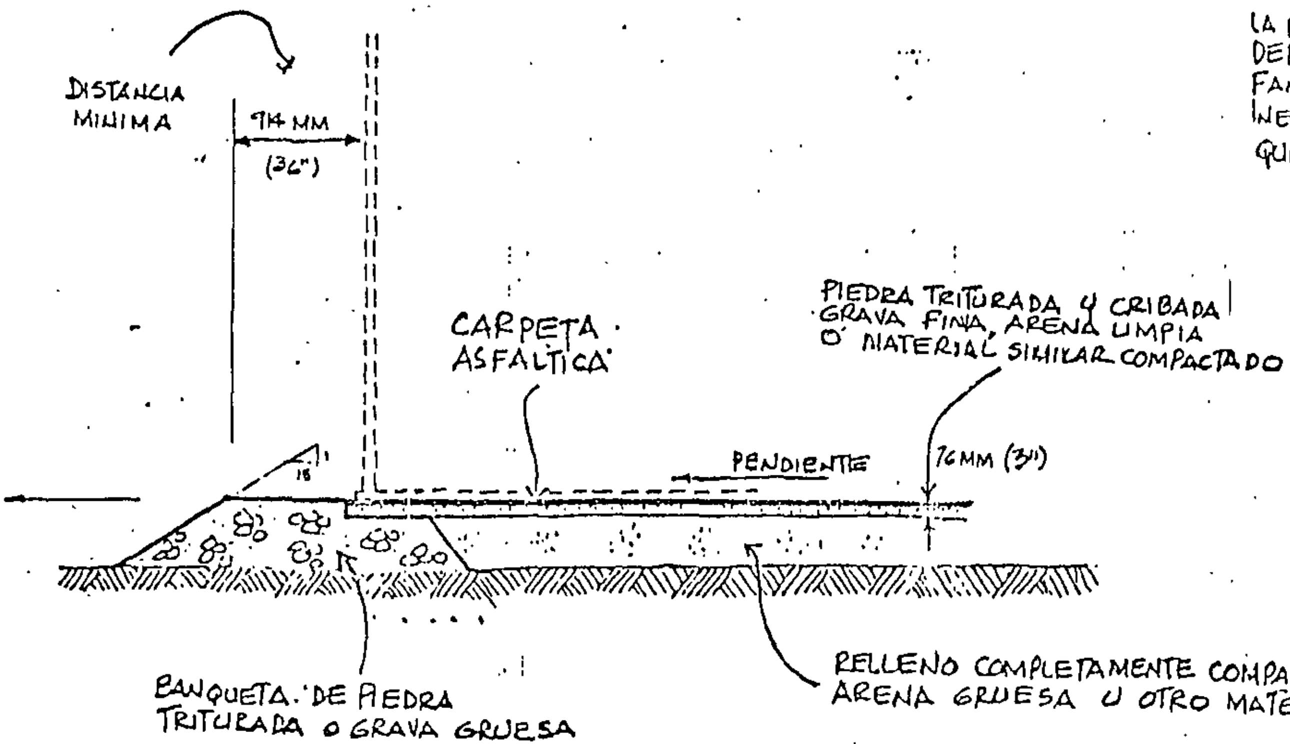




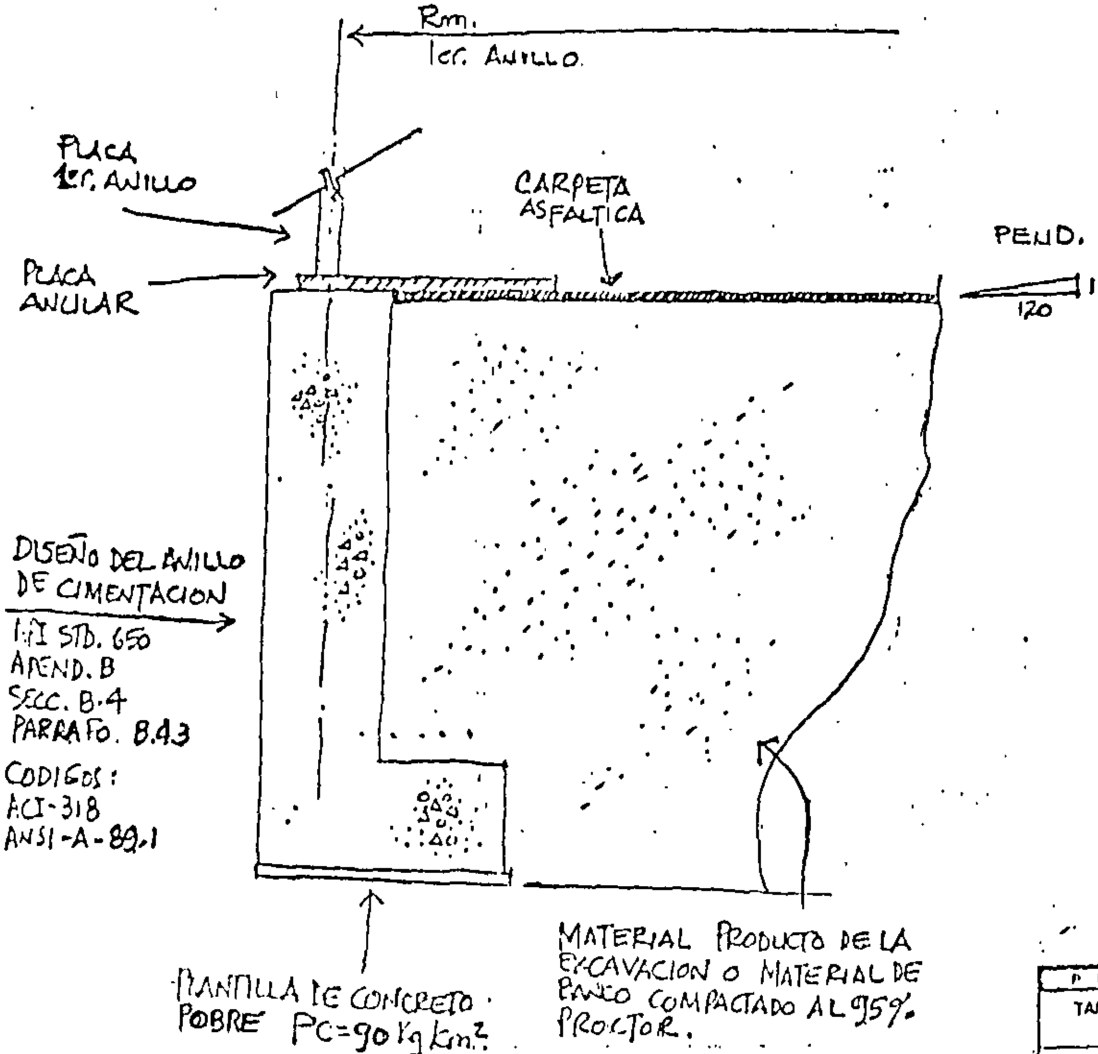
CONSERVACION DE MATERIALES: SE PROCURARA ALMACENAR A LA INTEMPERIE, LAS PLACAS DE LA ENVOLVENTE, EN LA FORMA MAS ADECUADA PARA EVITAR QUE PIERDAN SU CURVATURA; EN LA MISMA FORMA SE ALMACENARAN LAS PLACAS PLANAS DEL FONDO Y TECHO PARA QUE NO SE DEFORMEN. ESTOS MATERIALES SE PROTEGERAN DE LA INTEMPERIE, APLICANDO A TODA LA SUPERFICIE DE LA PLACA SUPERIOR DE CADA ESTIBA, DOS MANOS DE PINTURA ANTICORROSIVA, IGUALMENTE DEBERA HACERSE LA PROTECCION DE LOS BORDES Y BISELES DE LAS PLACAS. LOS DEMAS MATERIALES COMO EL ESTRUCTURAL, BOQUILLAS, TORNILLOS, HERRAJES, ETC, TAMBIEN SE ALMACENARAN CONVENIENTEMENTE PARA SU PROTECCION Y CONTROL.

CIMENTACION DE PIEDRA
API-650 B.4.3.2.

LA PARTE INFERIOR DE EXCAVACION DEBE ESTAR NIVELADA, REMOVER FANGO, VEGETACION Y MATERIALES INESTABLES HASTA LA PROFUNDIDAD QUE SEA NECESARIA.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	[illegible]	[illegible]	PLANO 1-006
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		



LAS CIMENTACIONES CONSTRUIDAS SEAN DE CONCRETO O PIEDRA, ESTARAN SUJETAS A LAS SIGUIENTES REVISIONES ANTES DE PROCEDER A LA ERECCION DEL TANQUE, CONJUNTAMENTE POR EL RESIDENTE DE LA CONTRATISTA Y POR EL SUPERVISOR DE PEMEX:

1.- EL RADIO MEDIO DEL ANILLO DEBERA SER EL CORRECTO, SEGUN DISEÑO CON UNA TOLERANCIA DE ± 25 mm. (1").

2.- LAS DIMENSIONES DEL ANILLO SERAN REVISADAS, ASI COMO LA LOCALIZACION DE REBAJES PARA LAS PUERTAS DE LIMPIEZA, (VEASE SECCION 3.7.7 FIGURA 3-9 DEL API-650).

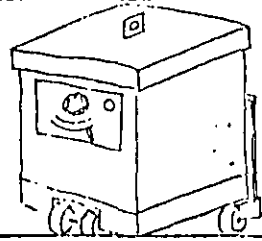
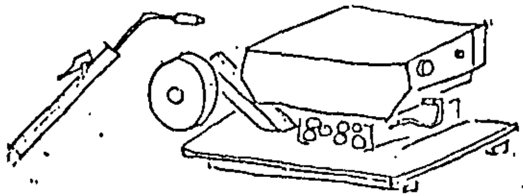
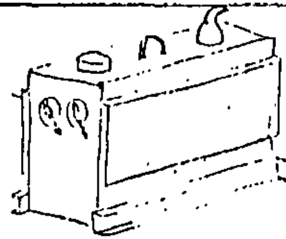
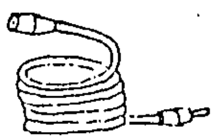
3.- SE EXAMINARAN LAS DIMENSIONES, LOCALIZACION Y ELEVACION DE LAS TUBERIAS SUBTERRANEAS Y LAS EXCAVACIONES.

4.- LA PENDIENTE DE LA BASE (PENDIENTE DEL FONDO DEL TANQUE) Y LA ELEVACION DE LA CORONA EN EL CENTRO DEL TANQUE, SERAN REVISADAS Y PROBADAS DE ACUERDO A LOS PLANOS DE CIMENTACION.

5.- LA BASE DEBERA SER COMPACTADA, UNIFORME Y CONFIGURADA APROPIADAMENTE. LA SUPERFICIE DEBERA ESTAR LIBRE DE PIEDRAS DE DIAMETROS MAYORES DE 25 mm (1"). VEASE API STD 650, APENDICE B SECCION 3.3.

6.- SI LA BASE ESTA PETROLIZADA Y EL ACEITE CUBRE LA SUPERFICIE O ESTA SATURADA A TAL PUNTO QUE PUDIERA MANAR O FLUIR A TRAVES DE LAS JUNTAS DEL FONDO, DEBERA SER CORREGIDA POR EL CONTRATISTA DE LA CIMENTACION, ANTES DE INICIAR EL MONTAJE DE LA ENVOLVENTE.

P E M E X	S P C O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	[illegible]	[illegible]	PLANO 1-005
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS.	SUMINISTRA
1	MAQUINA DE SOLDAR ROTATORIA O DE RECTIFICADOR PARA SOLDADURA MANUAL.		300 AMP.	20	COMPAÑIA CONTRATISTA
2	MAQUINA AUTOMATICA DE ARCO SUMERGIDO PARA SOLDAR JUNTAS HORIZONTALES.			2	"
3	PLANTA GENERADORA CON MOTOR DE COMBUSTION INTERNA.		400 KVA	2	"
4	CABLE DE COBRE FLEXIBLE CALIBRE 2/0 PARA PORTA-ELECTRODOS. CABLE DE TIERRA DE 20 M.	 		1200 MTS. 20	"

Equipo y Herramienta de Montaje

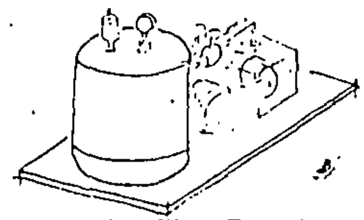
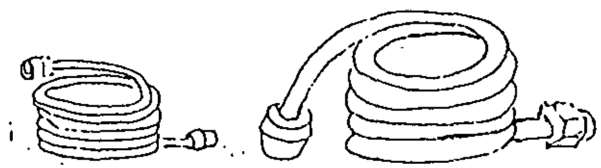
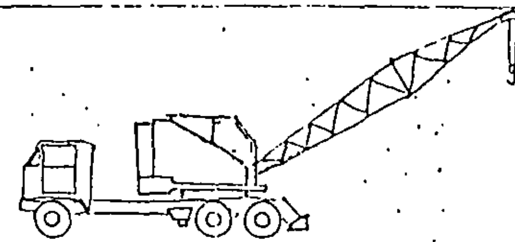
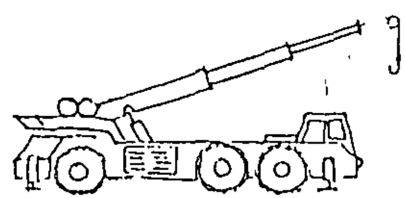
PEMEX | S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION

TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE

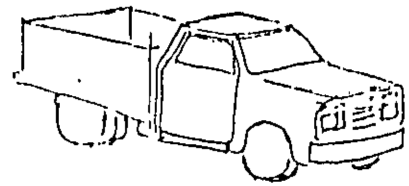
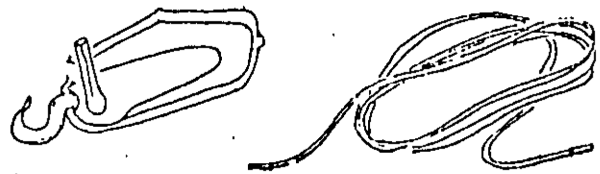
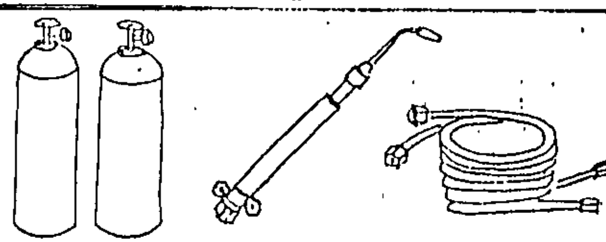
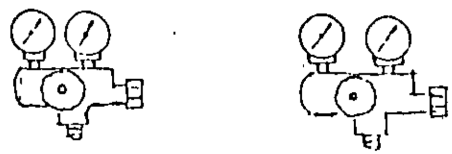
SECCION 1.0 GENERALIDADES

MANUAL DE MONTAJE

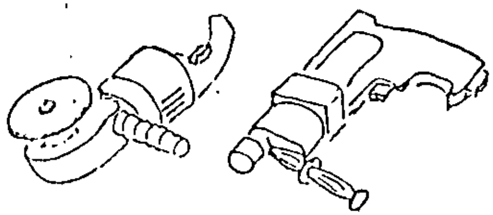
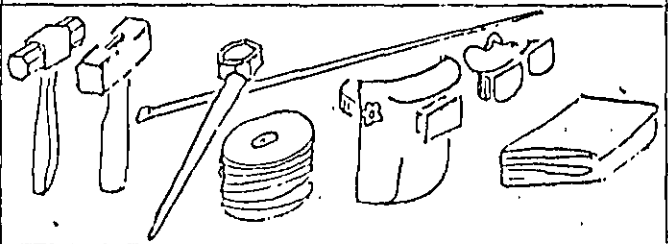
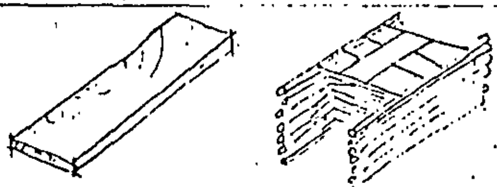
PLANO 1-006

No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS.	SUMINISTRA
5	COMPRESOR DE MOTOR DIESEL PARA SUMINISTRAR AIRE PARA ARCO-AIRE.		300 PIES³/MIN Y 7 KG/CM2	2	COMPAÑIA CONTRATISTA
6	MANGUERA FLEXIBLE		PARA PRESION DE 10 KG/CM2	100 MTS x 51 MM Ø. 300 MTS x 10 MM Ø	"
7	GRUA CON LLANTAS NEUMATICAS Y PLUMA DE 20 MTS.		TON.	2	"
8	GRUA TELESCOPICA		20 TON	1	"

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR ...	FECHA	[illegible]
SECCION 1.0 GENERALIDADES.	MANUAL DE MONTAJE		

No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS.	SUMINISTRA
9	CAMIONETA DE REDILAS		3 TON.	1	COMPAÑIA CONTRATISTA
10	TIRFORDS Y CABLE		2 TON.	5	"
11	EQUIPO DE CORTE PARA OXI-ACETILENO, CON MANGUERAS DE 30 MTS.			5	"
12	MEDIDOR DE ACETILENO MEDIDOR DE OXIGENO			5 5	"

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE			PLANO 1-02[illegible]
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS.	SUMINISTRA
13	SOPLETE ARCO AIRE. ESMERIL NEUMATICO O ELECTRICO. CINCEL NEUMATICO.			10 15 10	COMPAÑIA CONTRATISTA
14	HERRAMIENTA DIVERSA: MARTILLOS DE BOLA, MARROS, MACETAS, BARRETAS (6RIFAS), LLAVES, DISCOS ABRASIVOS, MANGAS, CARETAS, GOGGLES, ETC.			EN NUMERO SUFICIENTE	"
15	TABLONES PARA ANDAMIOS. MENSULAS PARA ANDAMIOS. ANDAMIOS TUBULARES DESMONTABLES O DEL TIPO GONDOLA CORREDIZOS:			EN NUMERO SUFICIENTE	"
16					

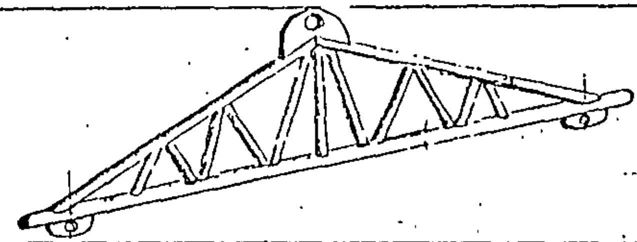
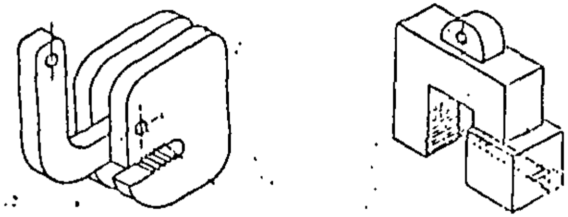
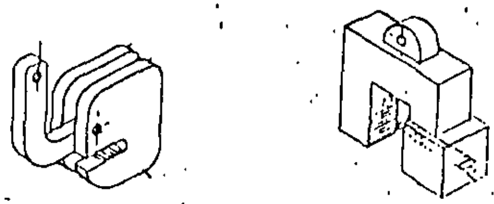
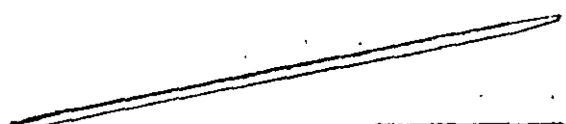
PEMEX | S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION

TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE

SECCION 1.0 GENERALIDADES

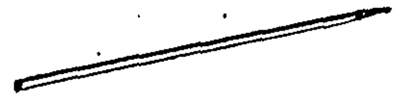
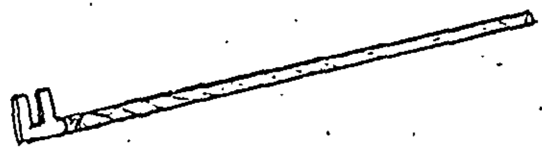
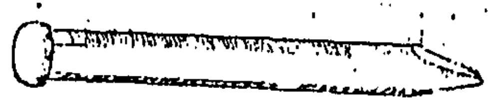
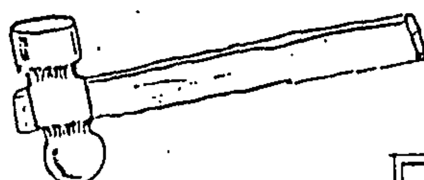
MANUAL DE MONTAJE

PLANO 1-0CC

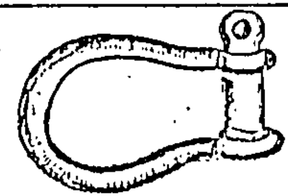
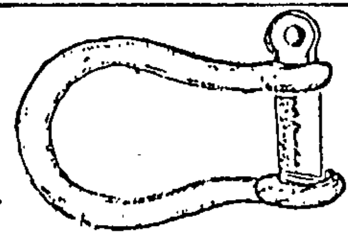
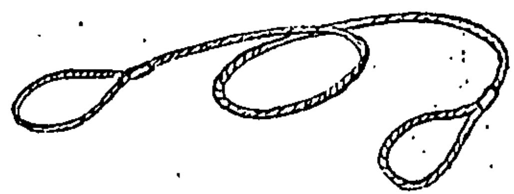
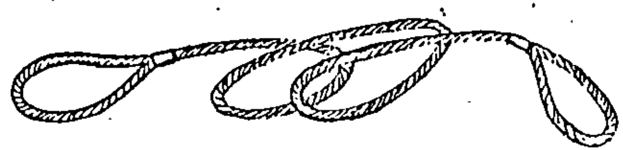
No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS.	SUMINISTRA
1	BALANCIN DE 7.00 M. DE CLARO, ENTRE APOYOS (PREFERIBLE TUBULAR)		8 TONS.	2	COMPAÑIA CONTRATISTA
2	PERROS PARA PLACA DE 38 MM. DE ESPESOR CON MORDAZAS ENDURECIDAS.		1	6	COMPAÑIA CONTRATISTA
3	PERROS PARA PLACA DE 19 MM. DE ESPESOR MAXIMO.			6	COMPAÑIA CONTRATISTA
4	BARRAS REDONDAS DE ACERO LAMINADO Y PUNTA CONICA DE 38 MM. DE DIAMETRO Y 1.50 M. DE LARGO			4	COMPAÑIA CONTRATISTA

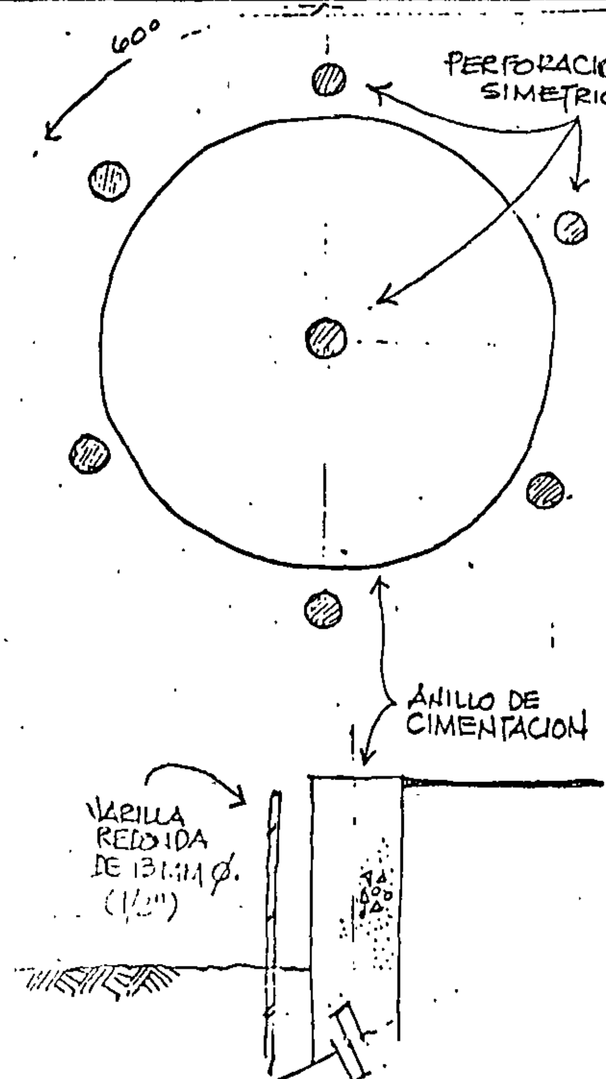
HERRAMIENTA PARA MANIOBRA Y MANEJO DE PLACAS

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	[illegible]	FECHA [illegible]	PLANO 1-010
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

No.	NOMBRE	FIGURA	CAPACIDAD	No. PZAS	SUMINISTRA
5	BARRAS REDONDAS DE ACERO LAMINADO Y PUNTA CONICA DE 19 MM. DE DIAMETRO, Y 0.75 M DE LARGO			4	COMPAÑIA CONTRATISTA
6	GRIFA DE 38 MM. DE DIAMETRO Y 1.50 M DE LARGO			4	COMPAÑIA CONTRATISTA
7	CINCELES DE ACERO LAMINADO DE 25 MM. DE DIAMETRO Y 20 CM. DE LARGO			20	COMPAÑIA CONTRATISTA
8	MARTILLO DE BOLA DE 900 GRAMOS (2 LBS.)			20	COMPAÑIA CONTRATISTA

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I.A.L. / APROBADO POR: Ing. J.R.B.	FECHA IV-86	PLANO 1-011
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

No	NOMBRE	FIGURA	CAPACIDAD	No. PZAS	SUMINISTRA
9	GRILLETE DE TORNILLO DE 19 MM.			10	COMPAÑIA CONTRATISTA
10	GRILLETE DE TORNILLO DE 38 MM.			6	COMPAÑIA CONTRATISTA
11	ESTROBOS DE CABLE DE ACERO DE 19 MM. DE DIAMETRO Y 4 M. DE LONGITUD.			6	COMPAÑIA CONTRATISTA
12	ESTROBOS DE CABLE DE ACERO DE 25 MM. DE DIAMETRO Y 4 M DE LONGITUD.			3	COMPAÑIA CONTRATISTA



Investigacion de Subsuelo

Haganse seis (6) perforaciones simetricas alrededor del perimetro de la envolvente y una (1) en el centro del tanque, con objeto de investigar las irregularidades del subsuelo como: piedras aflorando, cavidades de arcilla, vacios, etc. Esto es importante debido a que esta irregularidades pueden llegar a producir asentamientos desiguales.

Si el área alrededor del tanque es blanda y lodosa, enterrar una varilla redonda de 13mm (½") a un lado del cimiento en distintos lugares para asegurarse que la base no esta desplantada sobre material suelto (basura, abono, etc.) las condiciones del sitio de la ereccion pueden no ser las que se especifican en el diseño, en cuyo caso el residente de PEMEX y el del contratista, no deberan titubear en notificar a la Superintendencia Local de Construccion, si el sitio no es accesible o si no esta de acuerdo con las especificaciones.

PEMEX	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR	FECHA	PLANO 1-013
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

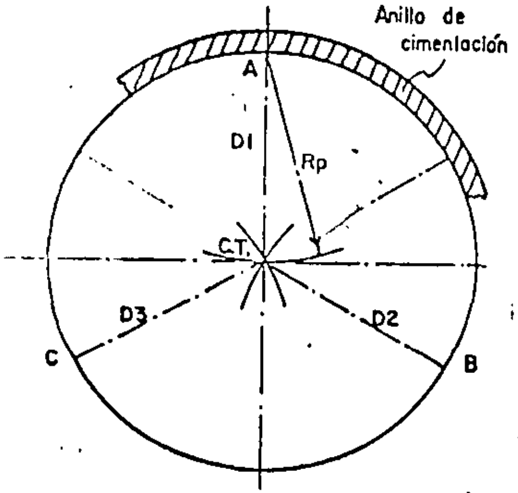


FIGURA
1.3.2.1A

$$Dp = \frac{D1 + D2 + D3}{3}$$

$$Rp = \frac{Dp}{2}$$

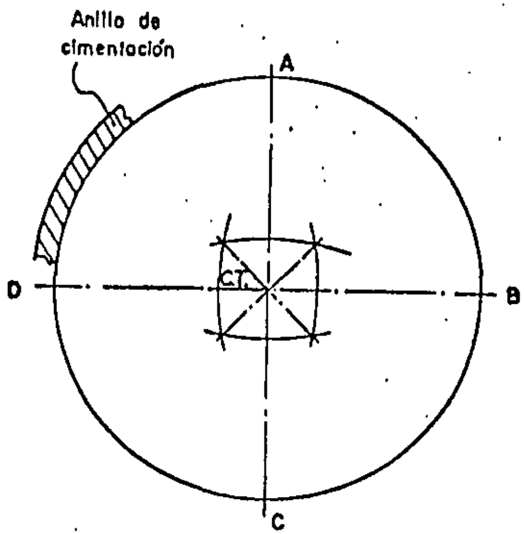


FIGURA
1.3.2.1B.

ES NECESARIO LOCALIZAR EL CENTRO DEL TANQUE EN LA BASE, ANTES DE QUE SEAN TENDIDAS LAS PLACAS DEL FONDO, PARA LO CUAL SE PROCEDERA DE LA SIGUIENTE MANERA:

1. MIDASE EL DIAMETRO DE LA BASE TRES LUGARES APROXIMADAMENTE A 120° (DIAMETRO INTERIOR DEL ANILLO DE CIMENTACION).
2. CALCULAR EL DIAMETRO PROMEDIO DE LAS MEDICIONES ANTERIORES Y DETERMINAR EL RADIO PROMEDIO.
3. SE ATIENDE UN EXTREMO QUE LA CINTA METALICA EN UN PUNTO "A" DEL DIAMETRO INTERIOR DEL ANILLO Y DESCRIBIR UN ARCO CON EL RADIO CALCULADO, CRUZANDO EL CENTRO DE LA BASE.
4. EN OTROS DOS PUNTOS B Y C DE LA PARED ANTERIOR DEL ANILLO A 120° APROXIMADAMENTE DEL PRIMERO, REPETIR EL PASO 3 (FIGURA 1.3.2.1.A).
5. LA INTERSECCION DE LOS TRES ARCOS DA EL CENTRO BUSCADO. MARCARLO CON UNA ESTACA.
6. OTRO PROCEDIMIENTO PARA LOCALIZAR EL CENTRO CUANDO NO EXISTE EN EL SIGUIENTE: FIJAR CUATRO PUNTOS A, B, C Y D APROXIMADAMENTE A 90° DE SEPARACION Y TRAZAR CUATRO ARCOS DESDE ESTOS PUNTOS, CON UN RADIO UN POCO MAYOR QUE EL REAL. EL CRUCE DE LAS DIAGONALES TRAZADAS EN LA INTERSECCION DE LOS ARCOS, DA EL CENTRO DEL TANQUE (FIGURA 1.3.2.1.B).
7. DESPUES QUE HA SIDO LOCALIZADO EL CENTRO, MIDASE EL RADIO DEL TANQUE EN TODAS DIRECCIONES (DEBERA COINCIDIR CON EL EJE DEL ANILLO) PARA CONFIRMAR QUE LAS DIMENSIONES DE LA BASE SON LAS ADECUADAS PARA EL TANQUE QUE SE VA A MONTAR Y QUE EL CENTRO ESTE CORRECTAMENTE FIJADO.

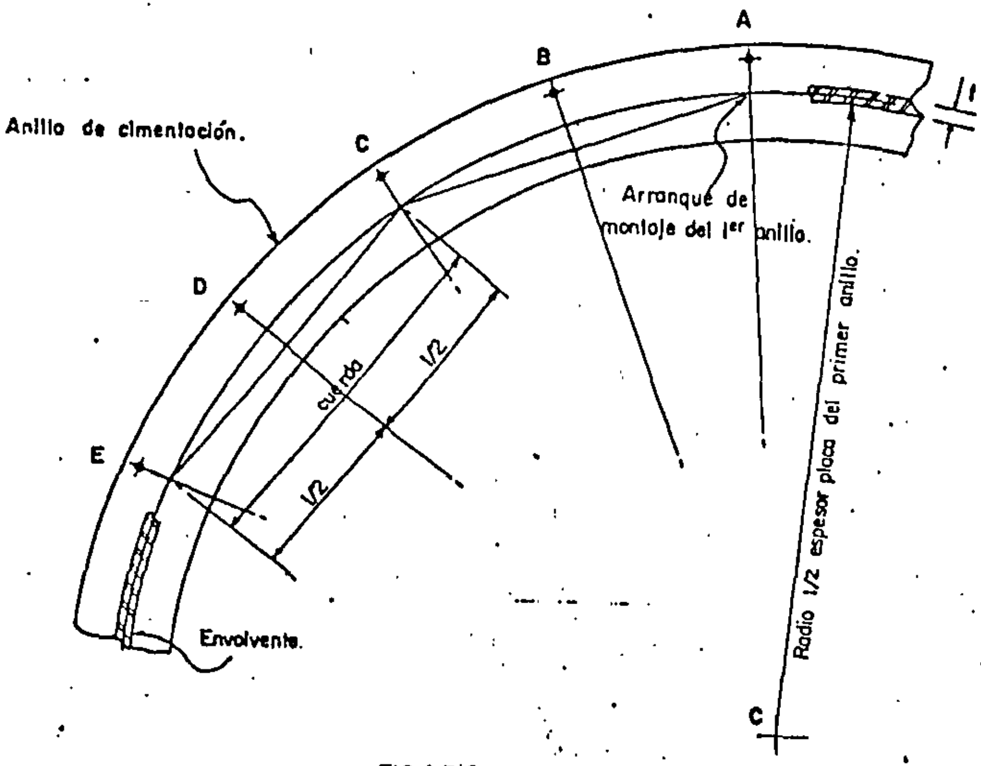


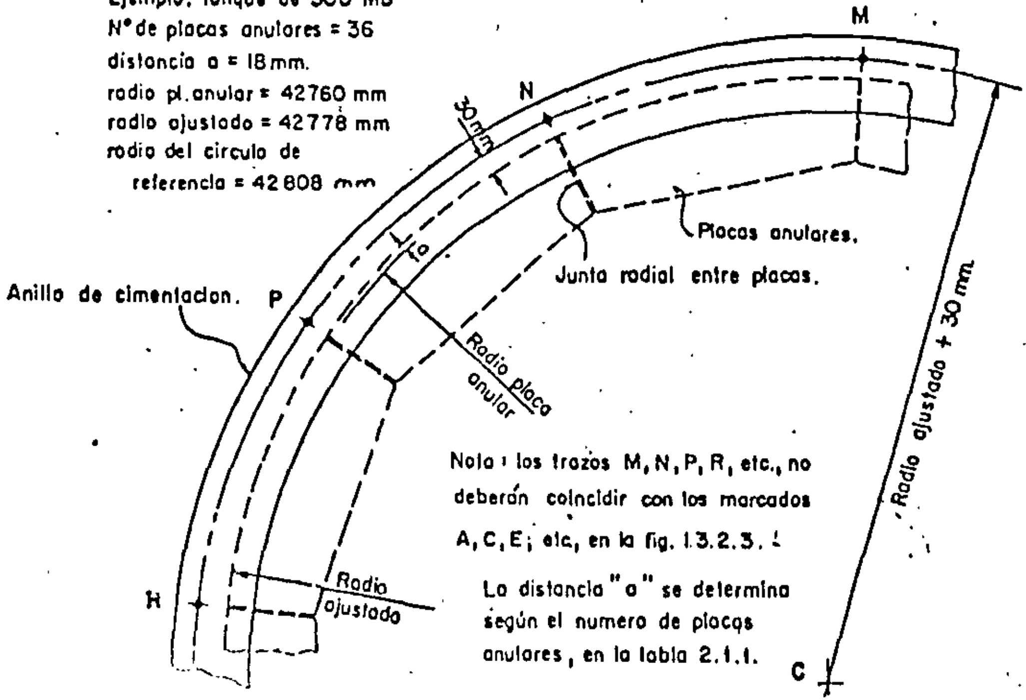
FIG.1.3.2.3A

TRAZOS EN EL ANILLO DE CIMENTACION

CON EL RADIO CORRESPONDIENTE AL MEDIO ESPESOR DE LAS PLACAS DEL PRIMER ANILLO DE LA ENVOLVENTE Y AUXILIADO CON LA CINTA METALICA, TRAZAR UN CIRCULO SOBRE LA CARA SUPERIOR DEL ANILLO. DESDE EL PUNTO DE INICIO DEL MONTAJE DE LA ENVOLVENTE INDICADO EN LOS PLANOS, TRÁCELE EN EL CIRCULO DE LAS CUERDAS DE CADA PLACA DEL PRIMER ANILLO CUYA LONGITUD DEBERÁ VENIR CALCULADA EN LOS DIBUJOS DE LA ENVOLVENTE Y LOCALIZAR EN DICHAS CUERDAS LA MITAD DE CADA PLACA. PROYECTAR ESTOS PUNTOS RADIALES EN EL ANILLO DE CONCRETO O PIEDRA, CALCULANDO QUE QUEDEN FUERA DE LAS PLACAS PERIFÉRICAS YA SEAN ANULARES O IRREGULARES. MARCARLOS CON PINTURA O EN OTRA FORMA DE TAL MANERA QUE NO SE BORREN.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HIZO POR [illegible] APROBADO POR [illegible]	FECHA [illegible]	PLANO 1-015
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

Ejemplo: tanque de 500 MB
N° de placas anulares = 36
distancia a = 18 mm.
radio pl. anular = 42760 mm
radio ajustado = 42778 mm
radio del circulo de
referencia = 42808 mm



Nota: los trazos M, N, P, R, etc., no deberán coincidir con los marcados A, C, E; etc, en la fig. 1.3.2.3. A

La distancia "a" se determina según el numero de placas anulares, en la tabla 2.1.1.

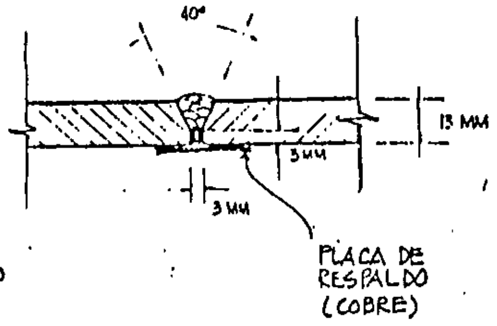
FIG. 1.3.2.3B

TRAZOS DE ANILLO DE CIMENTACION

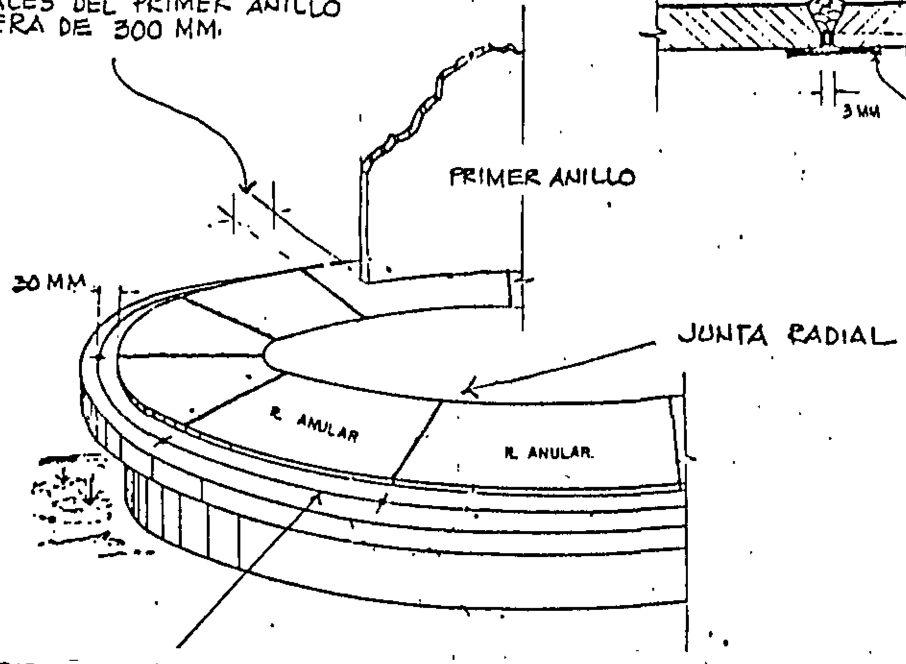
1. Con un radio igual al radio ajustado de la periferia de las placas anulares, aumentando 30 mm. (Véase párrafo 2.1.1 para este radio ajustado), trácese un círculo de referencia sobre el anillo de concreto. Sobre este círculo marcar la posición correcta de las juntas radiales entre placas anulares, cuidando de no hacer coincidir los trazos de las juntas verticales de la envolvente marcados según el párrafo anterior con éstos últimos trazos. La distancia mínima entre ambas juntas es de 300 mm. (Véase la figura 1.3.2.3.B).

2. Como ejemplo, apliquese el instructivo anterior a un tanque de 500,000 Bls. de capacidad identica a las fabricadas y montadas para PEMEX con ingeniería PDM (Véase la figura 1.3.2.3.B para los calculos).

CORTE DE JUNTA RADIAL



LA DISTANCIA MINIMA ENTRE LAS JUNTAS RADIALES DE LAS PLACAS ANULARES Y LAS JUNTAS VERTICALES DEL PRIMER ANILLO SERA DE 300 MM.



RADIO AJUSTADO + 30 MM. MARCAR LOS PUNTOS CON PINTURA O EN OTRA FORMA DE TAL MANERA QUE NO SE BORREN DURANTE LAS MANIOBRAS DE TENDIDO.

TENDIDO DE PLACAS

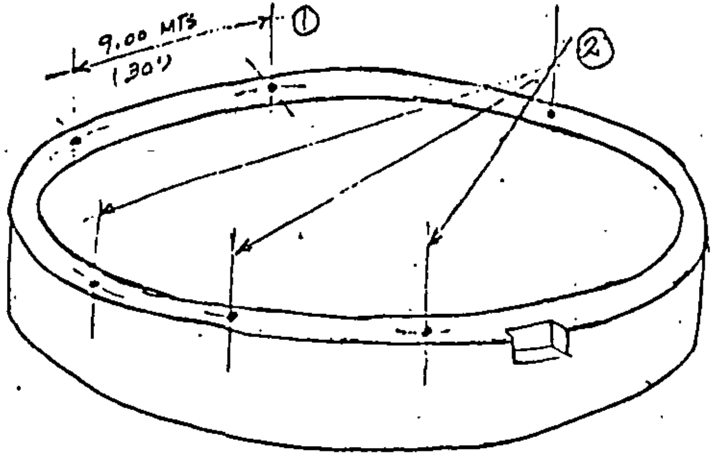
1. LAS PLACAS ANULARES PARA APOYO DEL PRIMER ANILLO, SE TENDERAN A CONTINUACION REPARTIDAS EN LA CIRCUNFERENCIA DE LA CIMENTACION, PARA PROCEDER A SOLDARLAS EN SECCIONES DE DOS EN DOS.

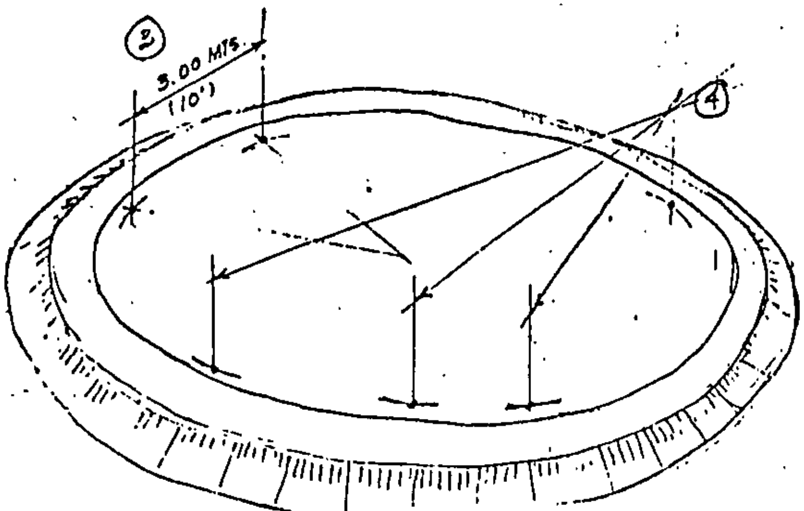
2. LAS UNIONES RADIALES DEBERAN TENER 100% DE PENETRACION Y CON RADIOGRAFIADO TOTAL UTILIZAR PLACA DE RESPALDO DE COBRE, PARA SOLDAR SOLO POR LA PARTE SUPERIOR.

3. EL ANILLO ANULAR DEBERA QUEDAR CONCENTRICO CON EL TRAZO DE RADIO EXTERIOR, QUE SE ENCUENTRA SOBRE LA CARA SUPERIOR DE LA CIMENTACION.

4. LA SOLDADURA RADIAL SUPERIOR DE LAS PLACAS ANULARES DEBERAN SER ESMERILADAS EN EL PASO DE APOYO DEL PRIMER ANILLO ENVOLVENTE.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	[illegible]	[illegible]	[illegible] 1-017
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		





TOLERANCIAS DE NIVEL

I.- ANILLOS DE CONCRETO:

SE CONSIDERARA A NIVEL LA CARA SUPERIOR O ENRASE DE DICHO ANILLO CUANDO ESTE DENTRO DE LOS SIGUIENTES LIMITES.

1.- DE ± 3MM (± 1/8") EN UNA LONGITUD DE CIRCUNFERENCIA DE 9.00 MTS. (30') TOMADA ARBITRARIAMENTE EN CUALQUIER PARTE DE LA MISMA.

2.- Y DE UN DESNIVEL DE ± 6MM. (± 1/4") DESDE UN PUNTO TOMADO COMO REFERENCIA A CUALQUIERA EN TODA LA CIRCUNFERENCIA.

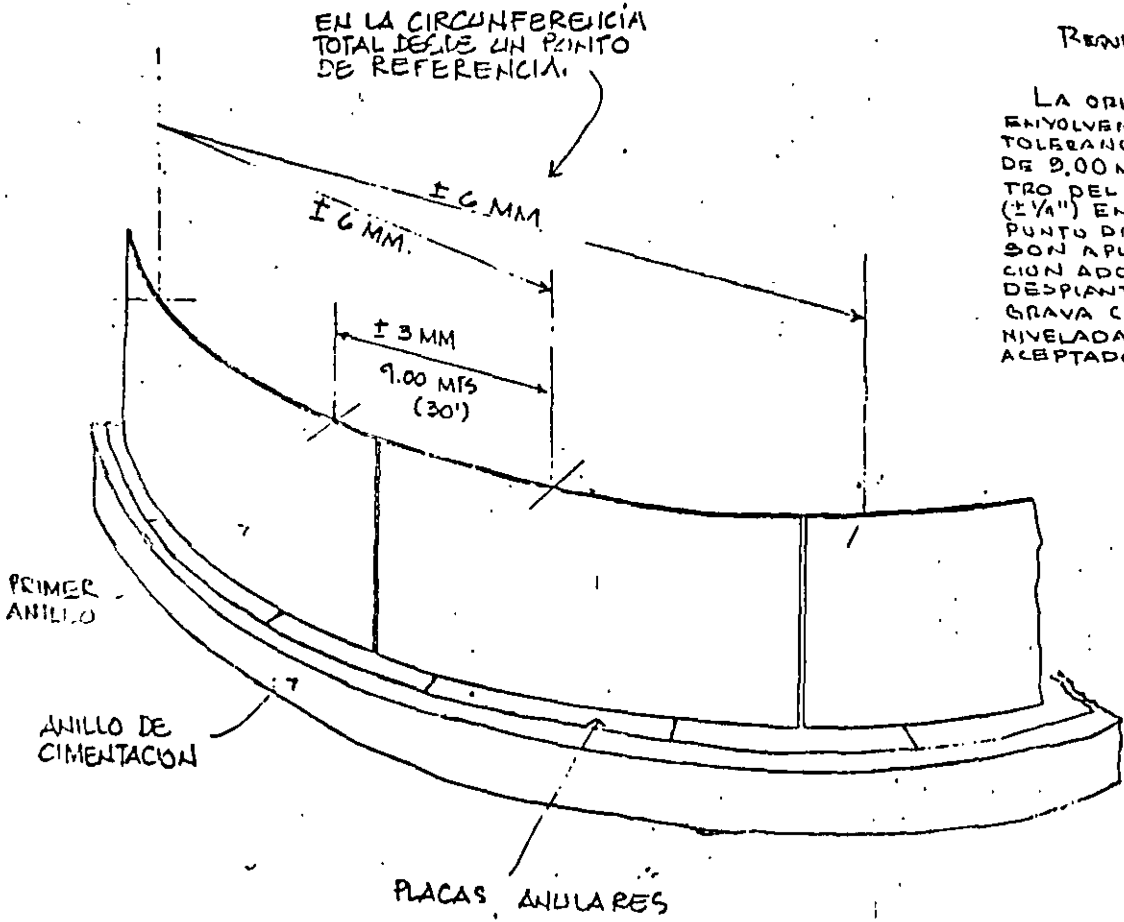
II.- ANILLOS DE PIEDRA O GRAVA:

LAS TOLERANCIAS ADMISIBLES PARA ESTE TIPO DE CIMENTACIONES SON LAS SIGUIENTES:

3.- DE ± 3MM. (1/8") EN 3.00 M. (10') DE LONGITUD DE CUALQUIER PARTE DEL ANILLO.

4.- Y DE ± 13 MM. (1/2") EN TODA LA CIRCUNFERENCIA DESDE UN PUNTO DE REFERENCIA.

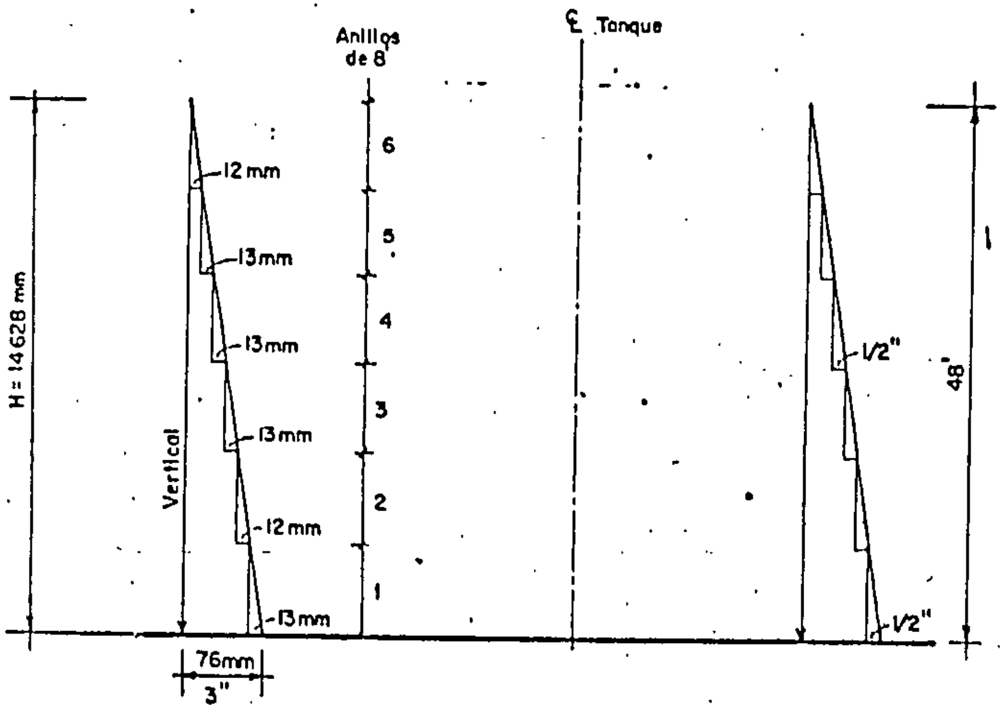
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	[illegible]	FECHA [illegible]	PLANO 1-018
SECCION 10 GENERALIDADES	MANUAL DE MONTAJE		



REQUERIMIENTOS DE HORIZONTALIDAD DE LA ENVOLVENTE.

LA ORILLA SUPERIOR DE CADA ANILLO DE LA ENVOLVENTE DEBERA ESTAR A NIVEL CON UNA TOLERANCIA DE ±3mm (±1/8") EN UNA LONGITUD DE 9.00 M. (30') EN CUALQUIER PARTE DEL PERIMETRO DEL TANQUE Y UNA TOLERANCIA DE ±6mm (±1/4") EN LA CIRCUNFERENCIA TOTAL DESDE UN PUNTO DE REFERENCIA. ESTAS TOLERANCIAS SON APLICABLES A CUALQUIER TIPO DE CIMENTACION ADOPTADO. SIN EMBARGO, UNA ENVOLVENTE DESPLANTADA SOBRE UN ANILLO DE PIEDRA O GRAVA CASI SIEMPRE TENDRA QUE SER RENIVELADA PARA ALCANZAR LOS CRITERIOS ACEPTADOS DE NIVEL.

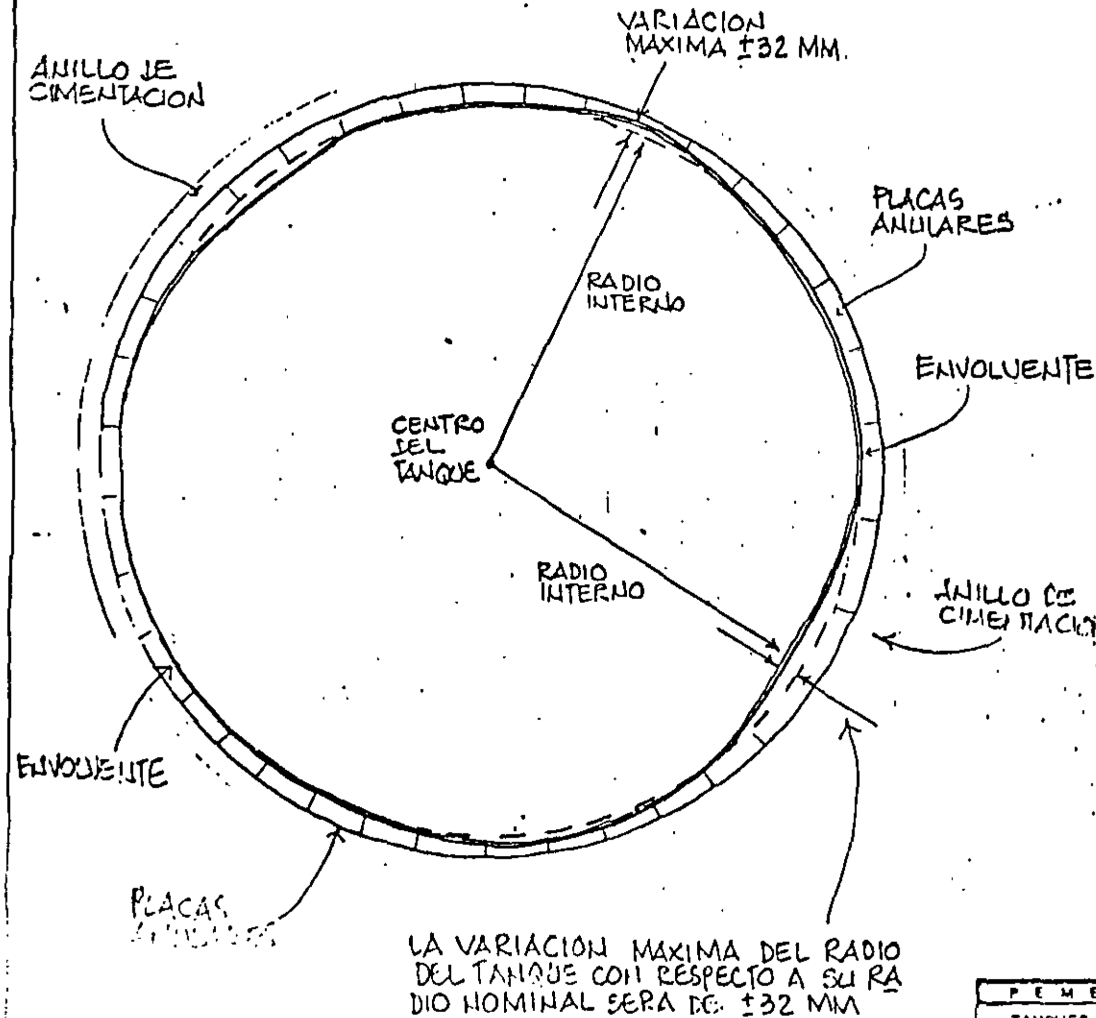
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR [illegible] APROBADO POR [illegible]	FECHA [illegible]	PLANO 1-013
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		



VERTICALIDAD.

LA MAXIMA DESVIACION DE LA VERTICAL DESDE LA PARTE MAS ALTA DE LA ENVOLVENTE A UN PUNTO SITUADO A 30 MM. ARRIBA DEL FONDO, NO DEBERA EXEDER DE 1/200 DE LA ALTURA TOTAL H DE LA ENVOLVENTE; LA DESVIACION EN CADA ANILLO, SERA PROPORCIONAL A LA MAXIMA. POR EJEMPLO; EN TODOS LOS TANQUES CON 6 ANILLO DE 2438 MM (8') DE ANCHO CADA UNO, LA ALTURA TOTAL H VALDRA 14,628 MM (48'). LA DESVIACION TOTAL SERA DE 76MM (3") EN NUMEROS REDONDOS Y EN CADA ANILLO, LA TOLERANCIA SE INCREMENTARA 12.5 MM (1/2") COMO MAXIMO. (VEASE LA FIGURA).

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE		FECHA IV-85	PLANO 1-520
SECCION 10 GENERALIDADES	MANUAL DE MONTAJE		



REDONDEZ

Los radios de la envolvente medidos a 300 mm (1') arriba del fondo, no exederan de las tolerancias indicadas en la tabla 1.4.5. Vease la seccion 5, parrafo 5.5.3 del API 650.

TABLA 1.4.5

Diámetro de Tanque	Tolerancia en el Radio
Hasta 12 metros (40')	± 13 mm. (± 1/2")
De 12 a 45 metros (40' a 150')	± 19 mm. (± 3/4")
De 45 a 76 metros (150' a 250')	± 25 mm. (± 1")
Mayor de 76 metros (mayor de 250')	± 32 mm. (± 1 1/4")

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE		FECHA IV-86	1-021
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

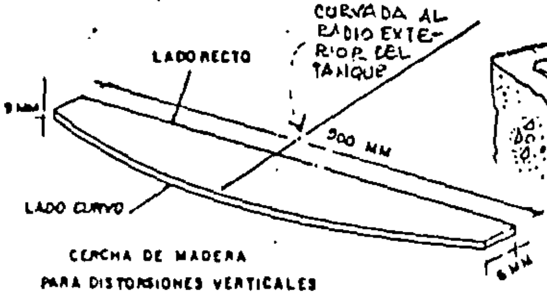
1.- SE CONOCE COMO "PEAKING" Y "BANDING", LOS EFECTOS QUE PROVOCAN DISTORSION DE LA CURVATURA HORIZONTAL Y DISTORSION DE LA RECTA VERTICAL DEL TANQUE.

2.- SE DEBERAN TOMAR LECTURAS DE LOS EFECTOS MENCIONADOS EN FORMA CONTINUA Y DURANTE TODA LA ETAPA DE CONSTRUCCION, DE ACUERDO AL SIGUIENTE CRITERIO:

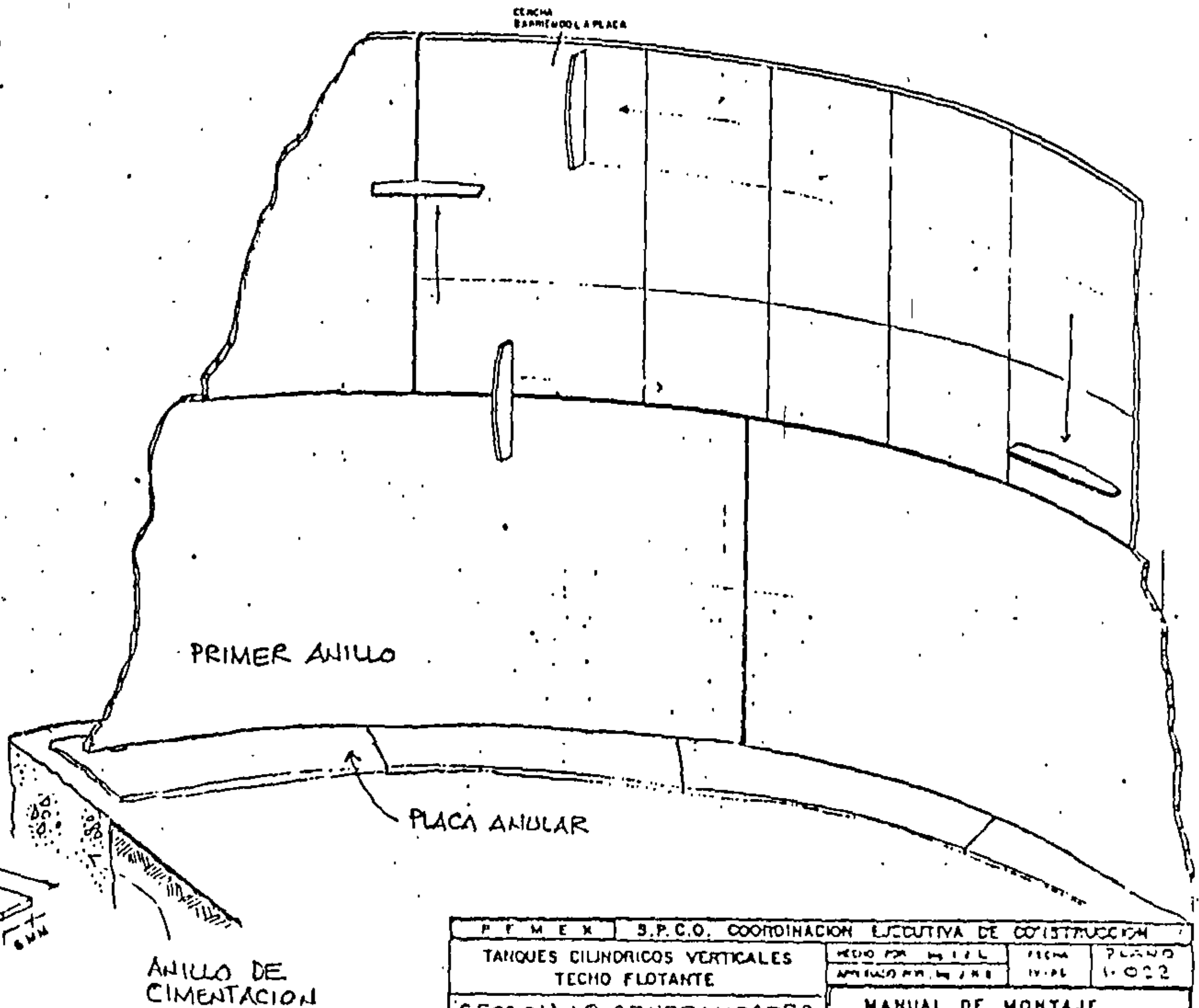
"MEDIR LA TOLERANCIA DE 13 [illegible] CON UNA CERCHA DE 100 MM DE LONG."

3.- ES RECOMENDABLE QUE ESTAS MEDICIONES SE EFECTUEN "BARRIENDO" CON LA CERCHA A TODO LO LARGO Y A LO ANCHO DE LAS PLACAS Y EN LAS UNIONES SOLDADAS VERTICALES Y HORIZONTALES DE LAS MISMAS (VER FIGURA)

4.- ESTE TIPO DE MEDICION DEBERA EFECTUARSE EN TODAS LAS PLACAS DEL TANQUE Y APUNTARSE EN UN REGISTRO EN EL QUE SE CONTROLE Y DETECTEN LOS PUNTOS FUERA DE TOLERANCIA, EN ESTOS CASOS SE HARAN LAS CORRECCIONES NECESARIAS A LAS PLACAS DEFORMADAS.



CERCHA DE MADERA PARA DISTORSIONES VERTICALES Y HORIZONTALES



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR [illegible] APROBADO POR [illegible]	FECHA [illegible]	PLANO 1-022
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

NIVELACIÓN DE LOS ANILLOS DE LA ENVOLVENTE.

LLEVAR REGISTROS ADECUADOS DE LAS LECTURAS DE NIVELACIÓN DE LA ENVOLVENTE, DESPUÉS QUE CADA UNO DE LOS PRIMEROS TRES -- ANILLOS HA SIDO MONTADO. SI HA OCURRIDO UN ASENTAMIENTO DIFERENCIAL MIENTRAS SE ESTÁ MONTANDO EL SEGUNDO Y EL TERCER ANILLO, CONTINUAR REVISANDOLOS HASTA QUE DOS ANILLOS CONSECUTIVOS NO REGISTREN HUNDIMIENTOS DIFERENCIALES. ASENTAR -- LECTURAS ANTES Y DESPUÉS DE CADA RE-NIVELACIÓN. TAMBIÉN REGISTRAR LOS DIÁMETROS DE TANQUES DE TECHO FLOTANTE EN TODOS LOS ANILLOS QUE REQUIERAN LECTURAS DE NIVEL. VÉASE LA TABLA 1.4.8 PARA LAS DIFERENCIAS ADMISIBLES EN DICHOS DIÁMETROS.

TABLA 1.4.8

DIÁMETRO DEL TANQUE M - (PIES)		DIFERENCIA ADMISIBLE DIAM. MAX. A DIAM. MINIM. MM.		PULG.
0-12	(0-40)	25	-	(1)
12-45	(40-150)	38	-	(1 1/2)
45-76	(150-250)	51	-	(2)
MAYOR DE 76	(MAYOR DE 250)	64	-	(2 1/2)

NOTA : Todos los cintas métricas adhesivas estarán a la misma distancia abajo de la orilla superior de las placas de la envolvente con una tolerancia de ± 1.5 mm (1/16")

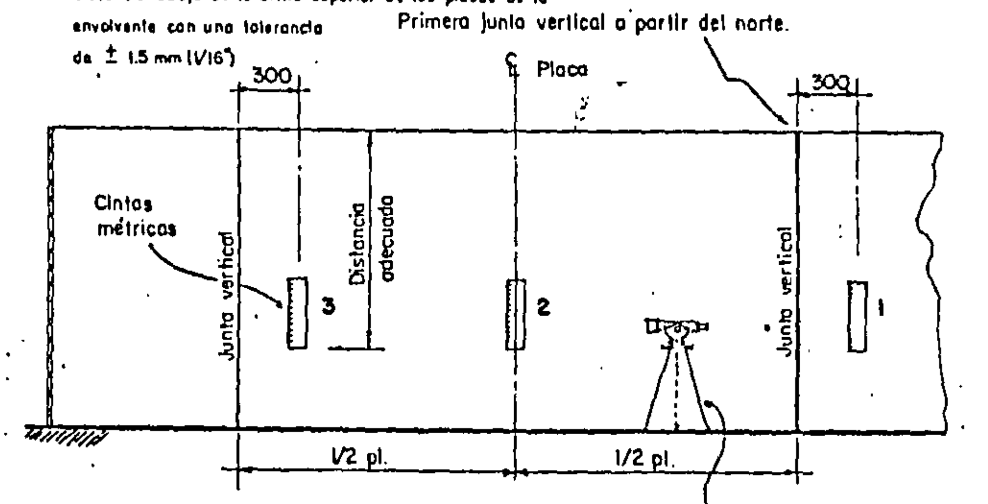


FIG. 1.4.8

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR	FECHA	PLANO 1-023
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

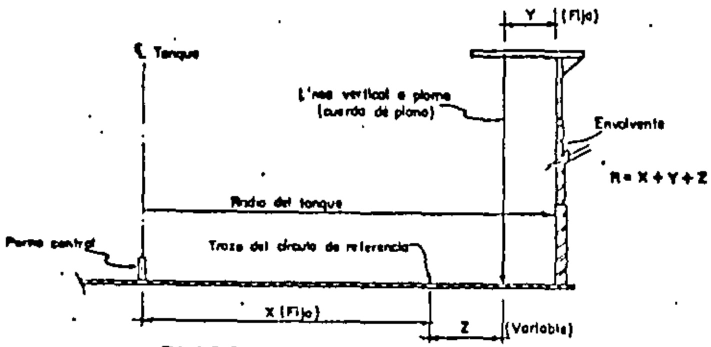


FIG. 1.5.2a

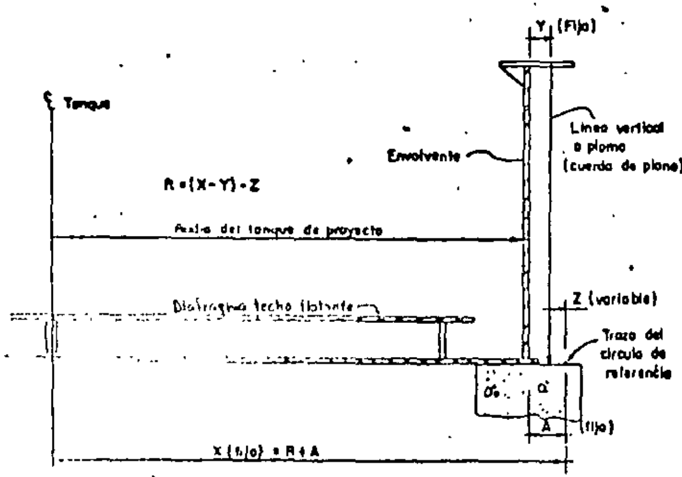


FIG. 1.5.2b

LA MEDICIÓN DIRECTA DEL RADIO DE LOS ANILLOS DE LA ENVOLVENTE PARA FINES DE REVISIÓN DE LA REDONDEZ DE LA MISMA, DA RESULTADOS SATISFACTORIOS EN TANQUES HASTA ALREDEDOR DE - - - 45.00 M. (150') DE DIÁMETRO. PARA RADIOS MAYORES HAY DOS -- PROCEDIMIENTOS DE MEDICIÓN PARA OBTENER RADIOS REALES.

. MÉTODO CONSISTE EN TRAZAR UN CÍRCULO DE REFERENCIA EN EL FONDO CON UN RADIO X Y USAR UNA PLOMADA CON ALAMBRE CUERDA DE PIANO CON UNA MEDIDA Y, FIJA EN EL EXTREMO SUPERIOR DEL ANILLO CORRESPONDIENTE; VER FIGURA - (1.5.2A). MEDIR LA DISTANCIA Z EN TODA LA PERIFERIA, CADA 5° A PARTIR DEL ORIGEN Ó NORTE CONVENCIONAL Y SIGUIENDO UN MOVIMIENTO CONTRARIO AL DE LAS MANECILLAS DEL RELOJ. EL RADIO BUSCADO ES IGUAL A LA SUMA X+Y+Z. TEÓRICAMENTE, SI LA REDONDEZ ES PERFECTA, LA DISTANCIA Z SERÁ LA MISMA EN TODAS LAS MEDICIONES Y POR LO TANTO, LA SUMA X+Y+Z SERÁ EL RADIO DEL TANQUE INDICADO EN EL PLANO DEL FONDO. SIN EMBARGO, SI Z VARÍA DE UNA MEDICIÓN A LAS OTRAS, LOS RADIOS CALCULADOS TAMBIÉN VARÍAN Y EL TANQUE NO ESTÁ REDONDO. COMPARAR CON LAS TOLERANCIAS ADMISIBLES Y SI HAY DISCREPANCIA CORREGIR LA REDONDEZ.

SUMERGIR LA PLOMADA EN UN RECIPIENTE CON AGUA O ACEITE, PARA IMPEDIR CUALQUIER VARIACIÓN DE LA VERTICAL. EN LA MEDICIÓN DE LOS RADIOS DE LOS ANILLOS SUPERIORES CASI - SIEMPRE SUCEDE QUE YA SE ESTÁ TRABAJANDO EL DIAFRAGMA - DEL TECHO SOBRE EL FONDO, EN CUYO CASO PROCEDER DE ACUERDO CON LA FIGURA 1.5.2B O SEA HÁGANSE LAS MEDICIONES POR EL EXTERIOR DEL TANQUE, PERO SIGUIENDO LAS INDICACIONES CORRESPONDIENTES A LA FIGURA 1.5.2A. AHORA EL RADIO DEL TANQUE SE CALCULA CON LA DIFERENCIA (X-Y)-Z.

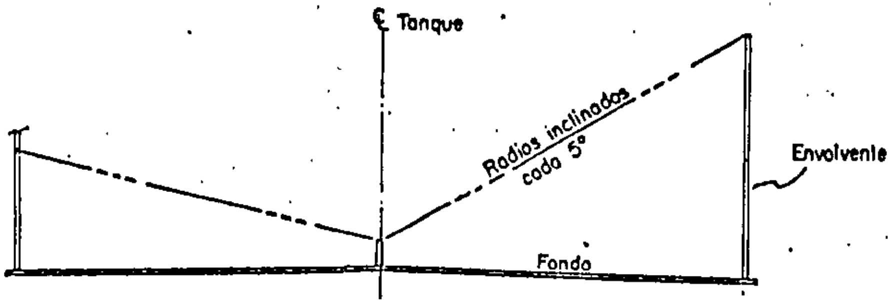


FIG. 1.5.2 c

EL SEGUNDO PROCEDIMIENTO CONSISTE EN MEDIR CON LA CINTA DE ACERO, RADIOS INCLINADOS DESDE EL PERNO CENTRAL DEL TANQUE A LA ORILLA SUPERIOR DE CADA ANILLO (VÉASE FIGURA 1.5.2c) EN TODA LA PERIFERIA DE LA ENVOLVENTE, CADA 5° A PARTIR DEL ORIGEN Ó N CONVENCIONAL. LLEVAR UN REGISTRO DE MEDICIONES DE CADA RADIO INCLINADO Y COMPARARLAS. SI HAY DISCREPANCIA ENTRE DOS O MÁS MEDICIONES CONSECUTIVAS Y LA DIFERENCIA ENTRE LA MÁS LARGA Y LA MÁS CORTA NO ES ACEPTABLE, CALCULAR EL RADIO REAL INCLINADO Y CORREGIR LA ENVOLVENTE SI ES NECESARIO. CUANDO SE HAGA EL REGISTRO DE MEDICIONES, ANOTAR QUE SE TRATA DE RADIOS INCLINADOS. ESTE PROCEDIMIENTO REQUIERE QUE HAYA LA MISMA TENSIÓN EN LA CINTA EN CADA MEDICIÓN.

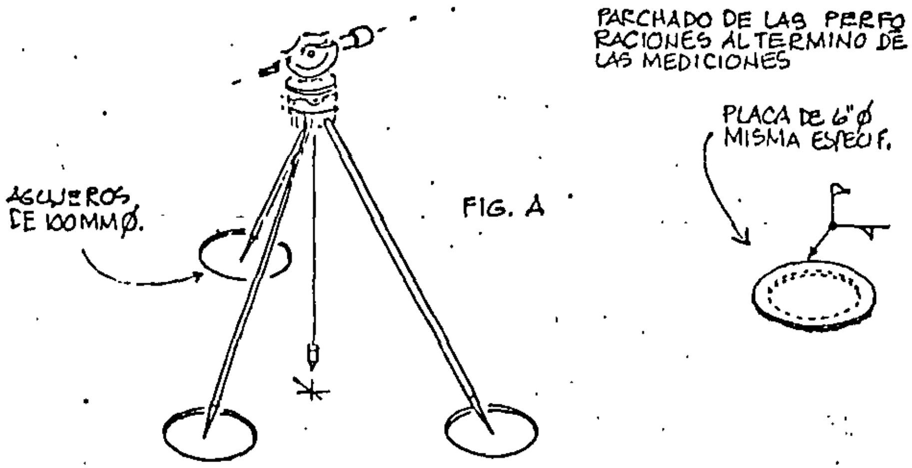


FIG. A

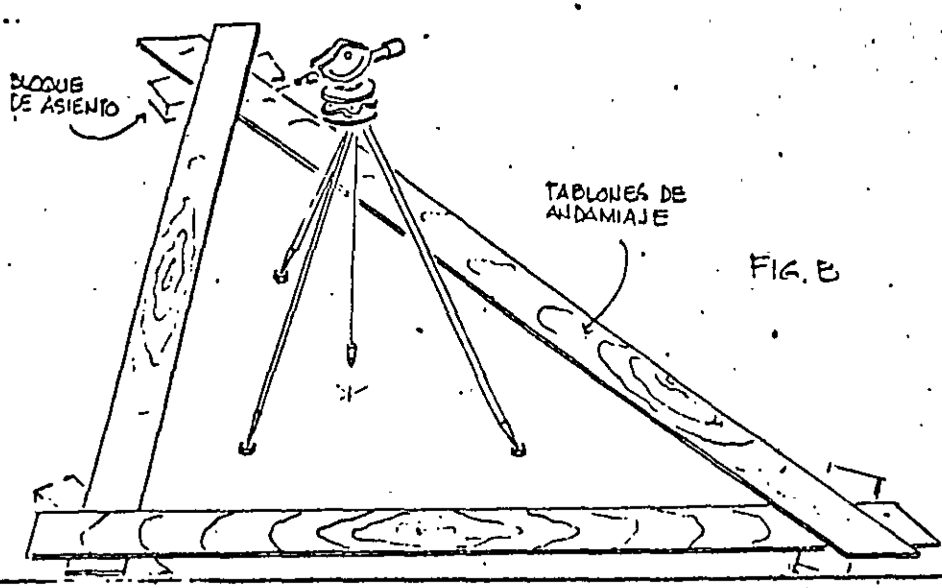


FIG. B

TRÁNSITO Y NIVEL: ESTOS INSTRUMENTOS DEBERÁN ESTAR APOYADOS EN UNA BASE SÓLIDA Y TAN CERCA DEL CENTRO DEL TANQUE COMO SEA POSIBLE. EXISTEN ALGUNOS MÉTODOS PARA INSTALARLOS EN UNA BASE SÓLIDA.

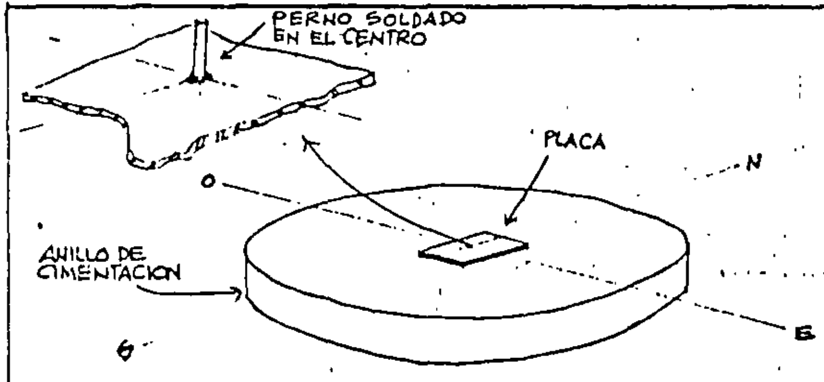
1. UNICAMENTE, CON LA APROBACIÓN DE LA SUPTCIA. LOCAL DE CONSTRUCCIÓN, SE PODRÁ APLICAR EL SIGUIENTE MÉTODO: CORTAR TRES (3) AGUJEROS DE 100 MM. (4") DE DIÁMETROS EN LA PLACA DEL CENTRO PARA DESCUBRIR LA BASE Y APOYAR EN ELLA EL TRÍPODE DEL INSTRUMENTO. DEBERÁ DISPONERSE DE PLACA DE LA MISMA ESPECIFICACIÓN PARA HACER TRES PARCHES CIRCULARES DE 150 MM. (6") DE DIÁMETRO PARA TAPAR LOS ORIFICIOS TRASLAPANDO Y SOLDANDO DESPUÉS DE TERMINAR LA OPERACIÓN TOPOGRÁFICA. HACER A ESTAS SOLDADURAS LAS MISMAS PRUEBAS QUE AL RESTO DEL FONDO. (VER FIGURA A)

2. PUNTEAR TRES (3) TUERCAS LISAS EN EL FONDO PARA SOSTENER EL TRÍPODE DEL INSTRUMENTO Y HACER UN ENTRAMADO TRIANGULAR DE MADERA (TABLONES USADOS EN LOS ANDAMIOS) ALREDEDOR DEL MISMO PARA AISLARLO (VER FIG. 1.5.3.1). COMO MOVIMIENTO EN EL ENTRAMADO PUEDE AÚN DESNIVELAR EL APARATO, HAY QUE ASEGURARSE DE REVISAR EL NIVEL DE LA BURBUJA ANTES DE CADA LECTURA. (VER FIGURA B).

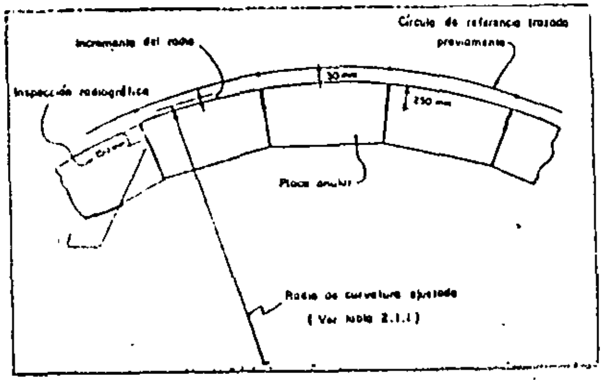
3. TRABAJAR UNA POLIGONAL FUERA DEL TANQUE.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: M. J. L. / APROBADO POR: M. J. R.	FECHA: IV-86	PLANO 1-026
SECCION 1.0 GENERALIDADES	MANUAL DE MONTAJE		

FONDOS CON PLACAS ANULARES
(SOLDADAS A TOPE CON BISEL EN "V" Y LAMINAS DE RESPALDO.)



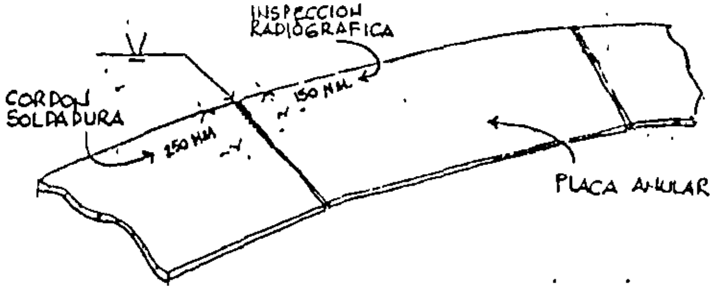
1.- COLOCAR LA PLACA CORRESPONDIENTE AL CENTRO DEL TANQUE Y TRANSPORTAR A LA MISMA EL CENTRO, PREVIAMENTE LOCALIZADO Y MARCADO EN EL ANILLO DE CIMENTACION; SOLDAR EN EL NUEVO CENTRO UN PERNO DE 13 MM Ø X 100 MM DE LONG.



2.- TENDER Y AJUSTAR LAS PLACAS ANULARES, A FIN DE OBTENER UNA SEPARACION APROPIADA ENTRE PLACA Y PLACA, USAR UN RADIO AJUSTADO QUE NO ES OTRO, QUE EL INDICADO EN LOS PLANOS PARA LA PERIFERIA DE LAS PLACAS, AUMENTANDO ALGUNOS MILIMETROS, SEGUN LA TABLA SIGUIENTE.

NUMERO DE PLACAS ANULARES	13	19	25	32	38	44	50
AUMENTO AL RADIO DEL PLANO EN MILIMETROS.	6	10	13	16	19	22	25

3.- INICIAR EL MONTAJE DE LAS PLACAS RECTANGULARES TRASLAPADAS DEL FONDO, SIGUIENDO LAS INSTRUCCIONES DE LOS PARRAFOS 2.1.2 Y 2.1.3 DEL MANUAL DE MONTAJE No.1 Y DE ACUERDO CON SU COLOCACION Y LA SECUENCIA MARCADA EN EL PLANO RESPECTIVO.



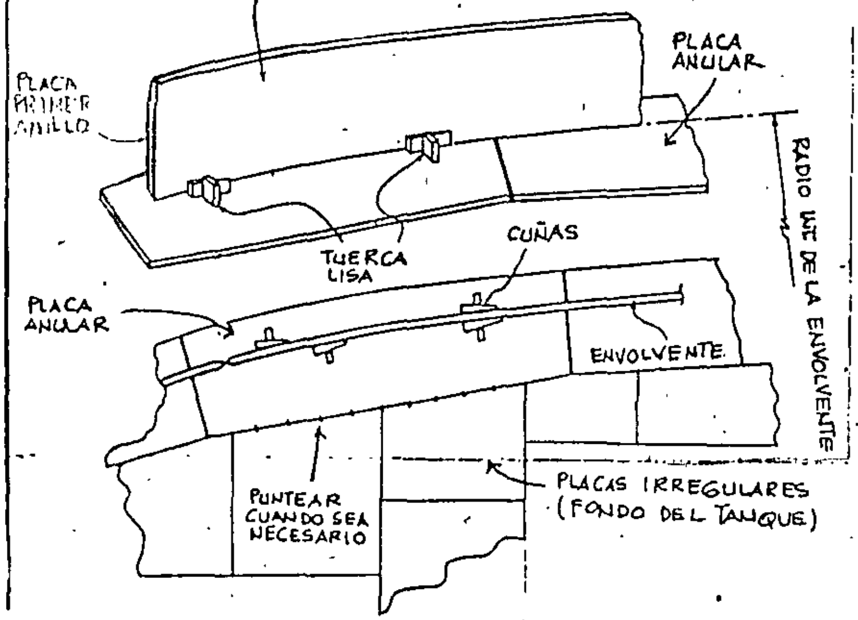
4.- SOLDAR LOS 250 MM. (10") DEL EXTREMO EXTERIOR DE TODAS LAS JUNTAS RADIALES DE LAS PLACAS ANULARES, ESMERILARLAS E INSPECCIONAR LA SOLDADURA CON RADIOGRAFIAS O PARTICULA MAGNETICA DE LOS 150 MM (6") EXTREMOS.

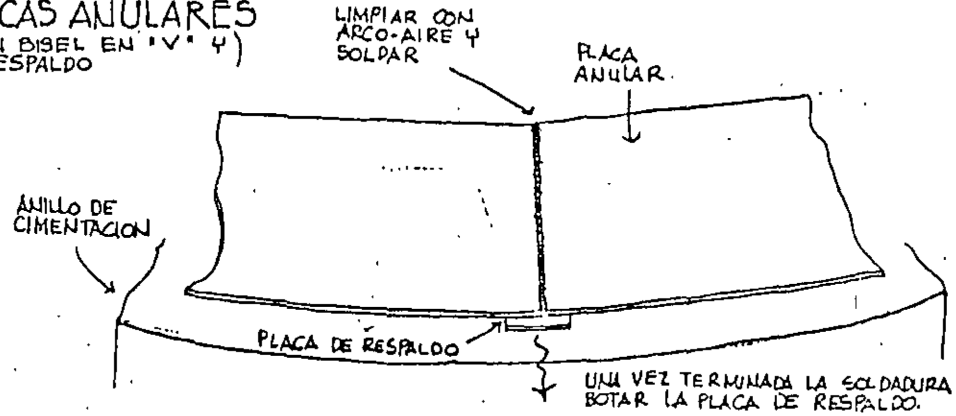
5.- MONTAR EL PRIMER ANILLO DE LA ENVOLVENTE, DE ACUERDO CON LAS INSTRUCCIONES DE LA SECCION 3.0 (CONSULTAR MANUAL DE MONTAJE No.1) Y SOLDAR LAS JUNTAS VERTICALES.

PLANO 1 DE 2

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I J L / APROBADO POR Ing. J H B	FECHA JUN 86	PLANO 2-001
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE N° 1		

FONDOS CON PLACAS ANULARES
(SOLDADAS A TOPE CON BISEL EN "V" Y LAMINAS DE RESPALDO)





6- FIJAR LA JUNTA ENTRE FONDO Y ENVOLVENTE, SEGUN PARRAFO 3.5 (MANUAL DE MONTAJE No. 1). EN LOS TANQUES DE GRAN CAPACIDAD, LAS PLACAS DEL PRIMER ANILLO SON TAN GRUESAS QUE LOS PUNZONES O CUÑAS DE AJUSTE ENTRE LA ENVOLVENTE Y LAS TUERCAS PUNTEADAS EN LAS PLACAS ANULARES, DEFORMAN A ESTAS EN LUGAR DE REDONDEAR LA ENVOLVENTE. PUNTEAR LAS PLACAS IRREGULARES (FONDO DEL TQ) A LAS ANULARES PARA AGREGAR RESISTENCIA CUANDO SEA NECESARIO.

7- SOLDAR LA JUNTA CIRCULAR ENTRE LA PLACA ANULAR Y EL PRIMER ANILLO DE LA ENVOLVENTE CON LAS INDICACIONES CONTENIDAS EN EL PARRAFO 3.5 (CONSULTAR MANUAL DE MONTAJE No. 1)

8- CON ARCO-AIRE CORTAR LAS JUNTAS RADIALES NO SOLDADAS ENTRE LAS PLACAS ANULARES ABRIENDOLAS A LA SEPARACION APROPIADA. TERMINAR DE SOLDAR ESTAS JUNTAS SIN INTERRUPCION Y BOTAR LA LAMINA DE RESPALDO.

9- FIJAR Y SOLDAR LAS PLACAS IRREGULARES A LAS ANULARES. CUANDO SE FIJE ESTA JUNTA, ASEGURARSE DE MANTENER LA MINIMA DISTANCIA ENTRE LA ENVOLVENTE Y LA PLACA IRREGULAR, COMO SE MUESTRA EN EL PLANO DE MONTAJE.

10- FIJAR Y SOLDAR LAS PLACAS IRREGULARES UNAS A OTRAS. CUANDO HAYAN SIDO SOLDADAS TODAS LAS PLACAS RECTANGULARES, SOLDAR LAS IRREGULARES A AQUELLAS.

PLANO 2 DE 2

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN. 86	PLANO 2-002
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

FONDOS CON PLACAS TRASLAPADAS RECTANGULARES

(TIPOS MAS COMUNES DE TENDIDO DE PLACAS DE FONDO).

1.- FONDOS CON LAS PLACAS FORMANDO HILERAS LONGITUDINALES Y FILAS TRANSVERSALES, LAS PLACAS PERIFERICAS DE CIERRE SON ANULARES E IRREGULARES (FIG. "A").
ESTE ARREGLO ES PARA TANQUES DE GRAN CAPACIDAD.
(DE 100 A 500,000 BLS).

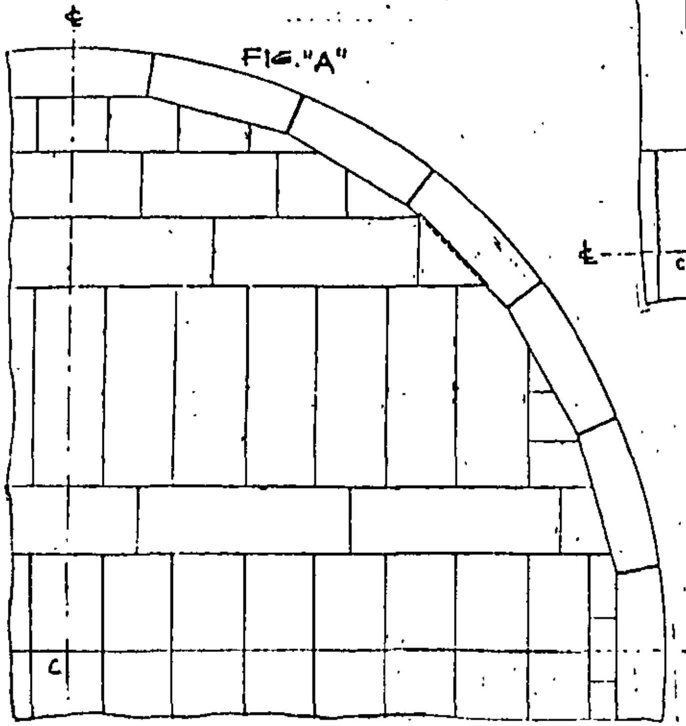
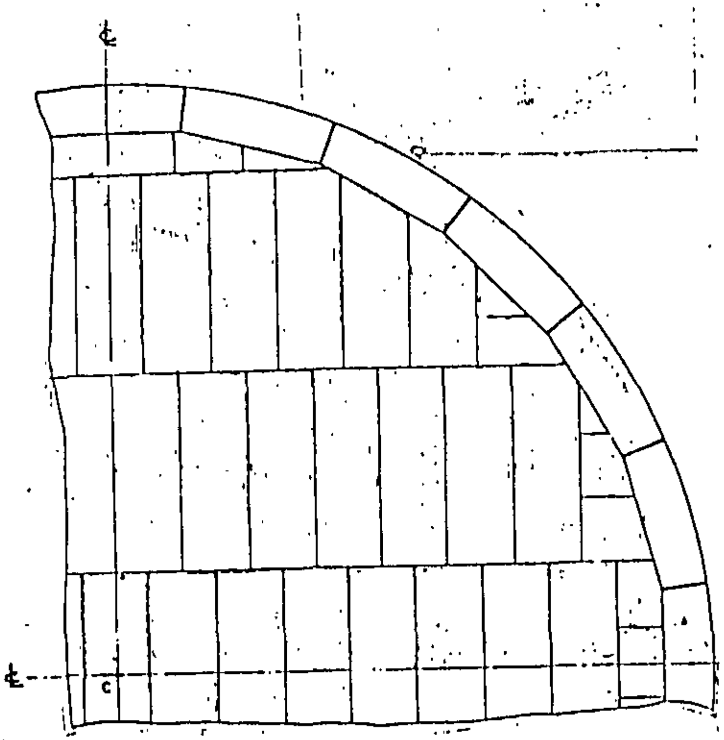
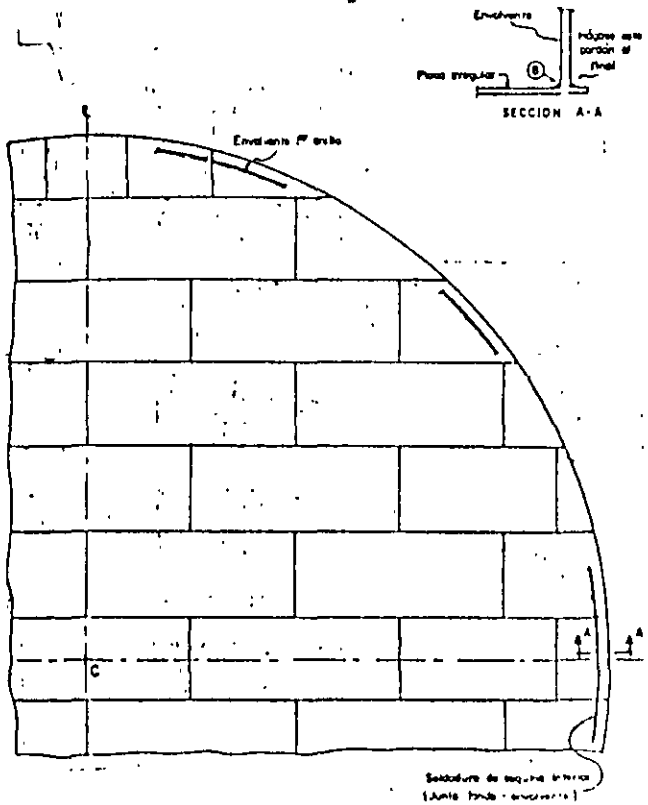


FIG. "A"



2.- FONDOS CON LAS PLACAS RECTANGULARES DISPUESTAS SOLAMENTE EN HILERAS LONGITUDINALES CON PLACAS ANULARES EN LA PERIFERIA Y PLACAS IRREGULARES TRANSVERSALES (FIG. "B") PARA TANQUES DE MEDIANA CAPACIDAD.
(DE 55 A 100,000 BLS.)

3.- FONDOS TIPO PLATAFORMA CON TODAS LAS PLACAS RECTANGULARES E IRREGULARES EN UN SOLO SENTIDO Y SIN PLACAS ANULARES. ESTE ARREGLO SE USA EN TANQUES DE MEDIANA A BAJA CAPACIDAD.
(DE 55,000 A 500 BLS.)

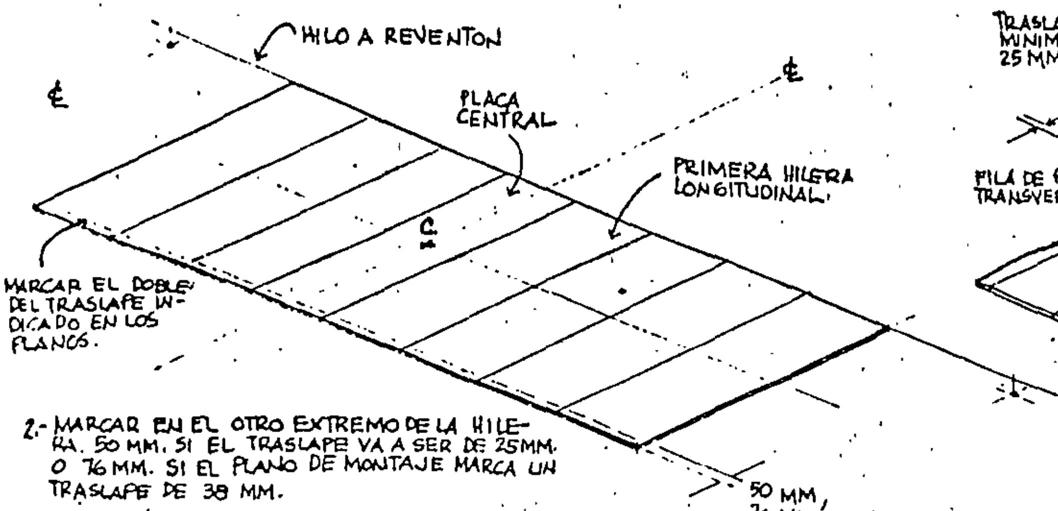


P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. APROBADO POR: Ing. J. H. B.	FECHA JUN. 86	PLANO 2-003
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

METODO PARA TENDIDO DE PLACAS

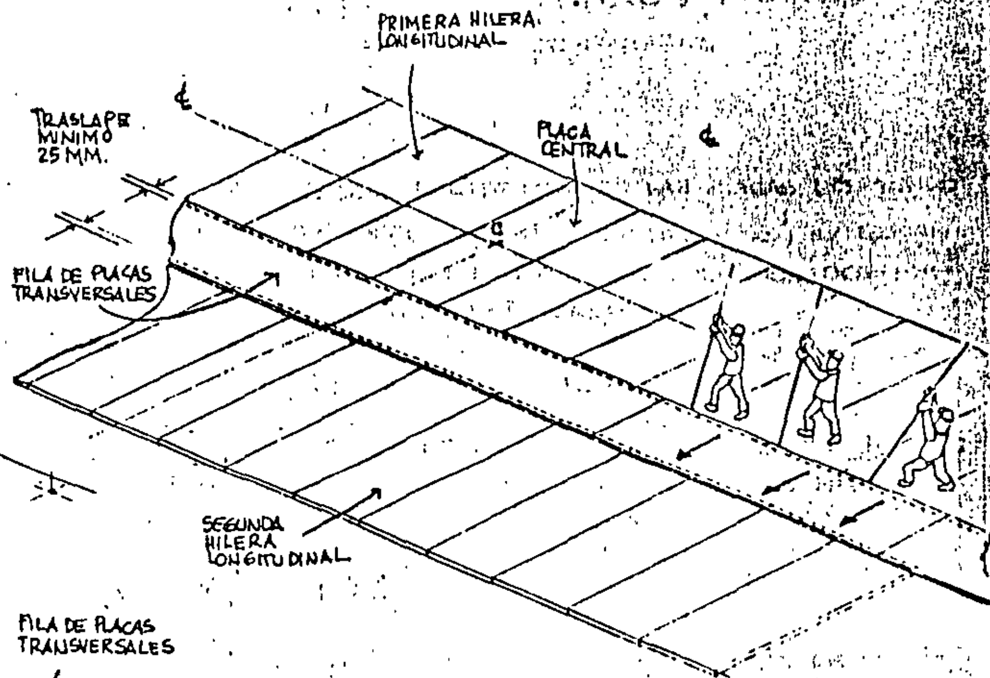
ARREGLO PARA TANQUES DE GRAN CAPACIDAD (DE 100 A 500,000 BLS).

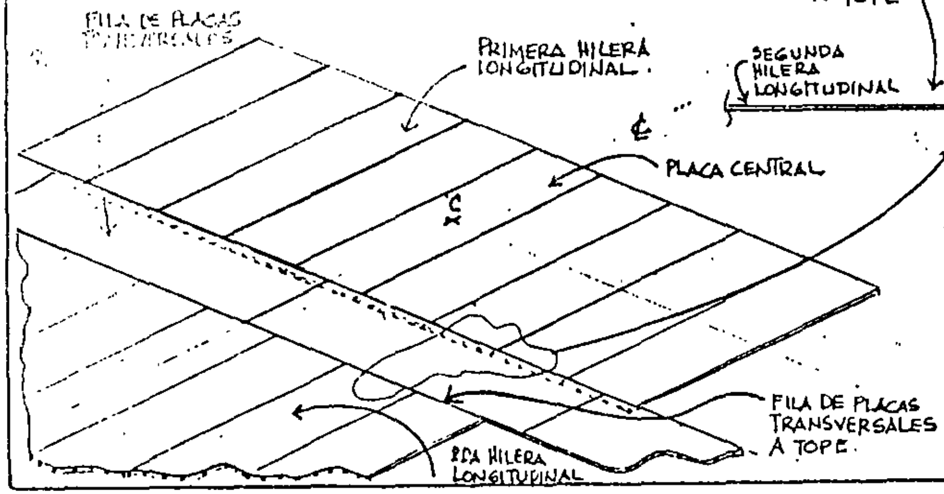
1.- TENDER LA HILERA CENTRAL DE PLACAS RECTANGULARES A UNO Y OTRO LADOS DE LA PLACA CENTRAL PREVIAMENTE COLOCADA Y MEDIANTE UN HILO A REVENTON, MANTENER UN EXTREMO RECTO.

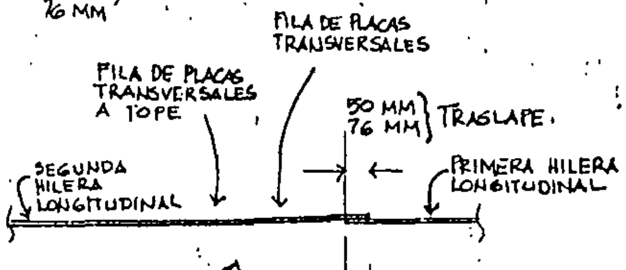


2.- MARCAR EN EL OTRO EXTREMO DE LA HILERA. 50 MM. SI EL TRASLAPE VA A SER DE 25MM. O 76 MM. SI EL PLANO DE MONTAJE MARCA UN TRASLAPE DE 38 MM.

3.- TENDER LA PRIMERA FILA DE PLACAS TRANSVERSALES DE MODO QUE SE TRASLAPEN LOS 50 o LOS 76 MM., MARCADOS EN LA PRIMERA HILERA UNIENDOLA LIGERAMENTE A ESTA.







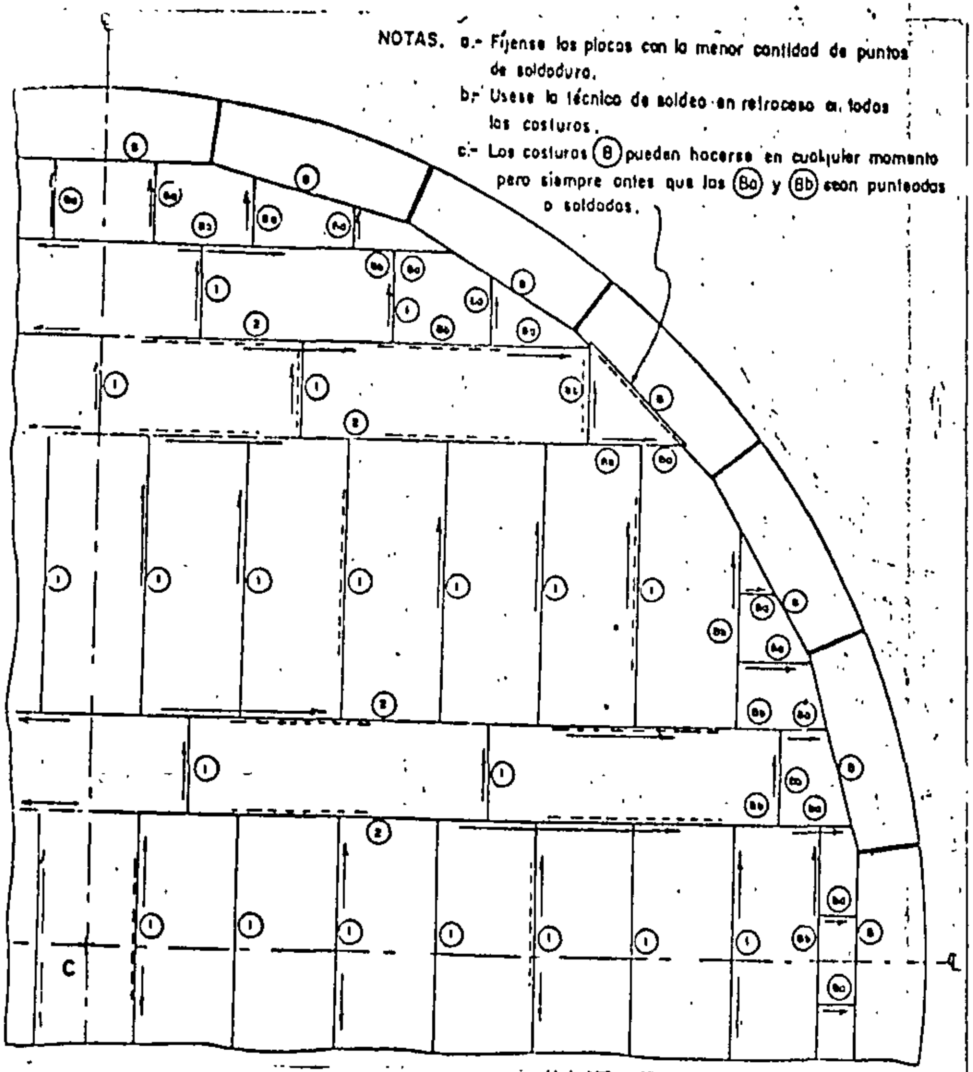
4.- TENDER LA SIGUIENTE HILERA DE PLACAS A TOPE CON LAS DE LA FILA TRANSVERSAL.

5.- DESTRUIR LOS PUNTOS DE SOLDADURA Y EMPUJAR CON BARRETAS LAS PLACAS DE LA FILA TRANSVERSAL POR ARRIBA DE LA SEGUNDA HILERA HASTA OBTENER UN TRASLAPE DE AMBAS HILERAS DE 25 A 38 MM. SEGUN EL CASO.

6.- REPETIR LAS OPERACIONES ANTERIORES EN EL LADO OPUESTO SIMETRICO.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN. 86	PLANO 2-004
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

NOTAS. a.- Fíjense las placas con la menor cantidad de puntos de soldadura.
b.- Usese la técnica de soldeo en retroceso a todas las costuras.
c.- Las costuras B pueden hacerse en cualquier momento pero siempre antes que las Ba y Bb sean punteadas o soldadas.



TENDIDO DEL FONDO CON PLACAS TRASLAPADAS RECTANGULARES.

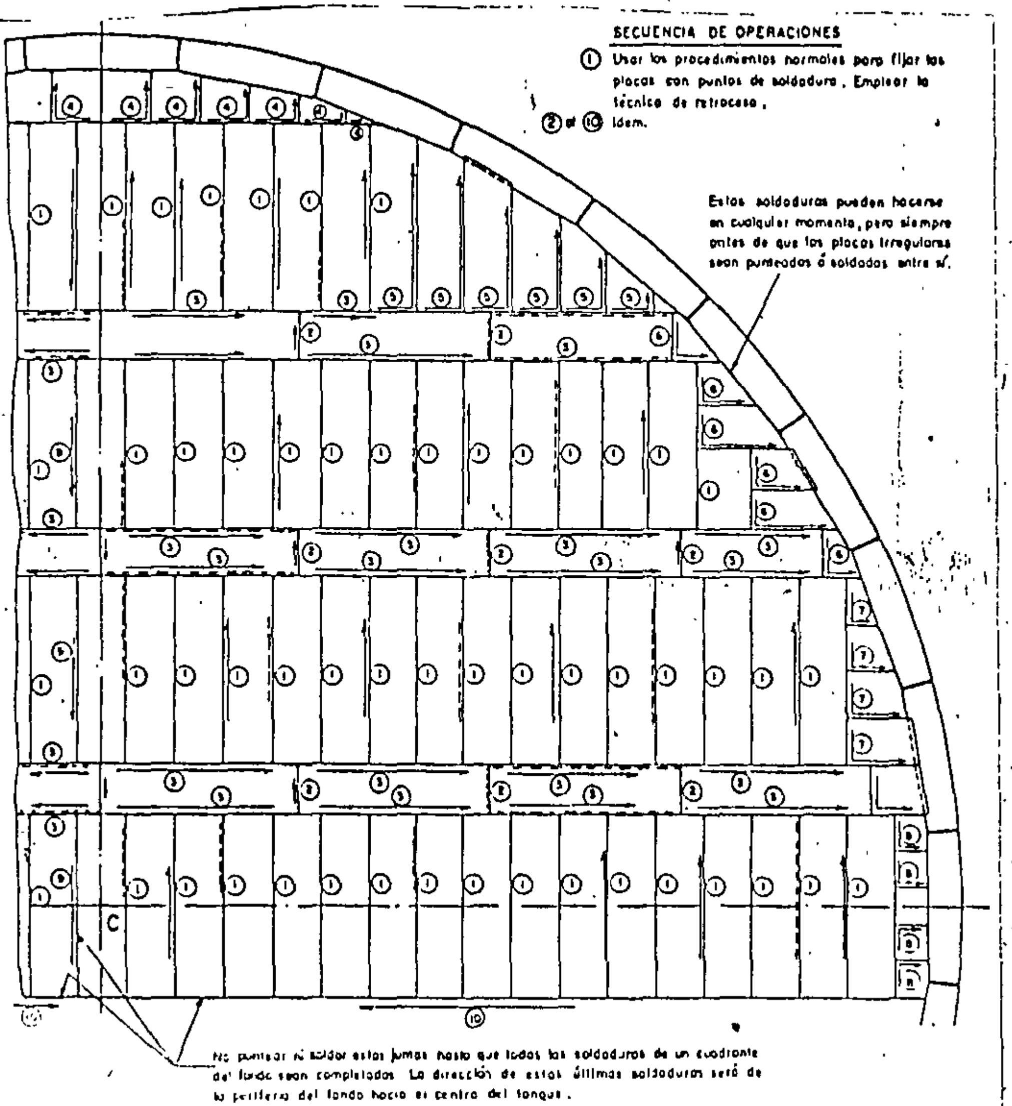
EL ARREGLO DEL TENDIDO DE LAS PLACAS DEL FONDO PUEDE ADOPTAR VARIAS FORMAS. A CONTINUACION SE DESCRIBEN TRES DE LOS TIPOS MAS COMUNES SELECCIONADOS POR LOS MAS IMPORTANTES DISEÑADORES DE TANQUES.

ARREGLO DEL TENDIDO Nº 1

FONDOS CON LAS PLACAS FORMANDO HILERAS LONGITUDINALES Y FILAS TRANSVERSALES. LAS PLACAS PERIFERICAS DE CIERRE SON ANULARES E IRREGULARES (COMO SE VE EN LA FIGURA). ESTE ARREGLO ES PARA TANQUES CON CAPACIDAD APROX. DE 100,000 BLS.

LA TECNICA DE SOLDEO PARA ESTE TIPO DE TENDIDO SE INDICA EN EL MANUAL DE MONTAJE Nº 1, SECCION 2.1.3.1 (TECNICA DE SOLDEO C.B.I).

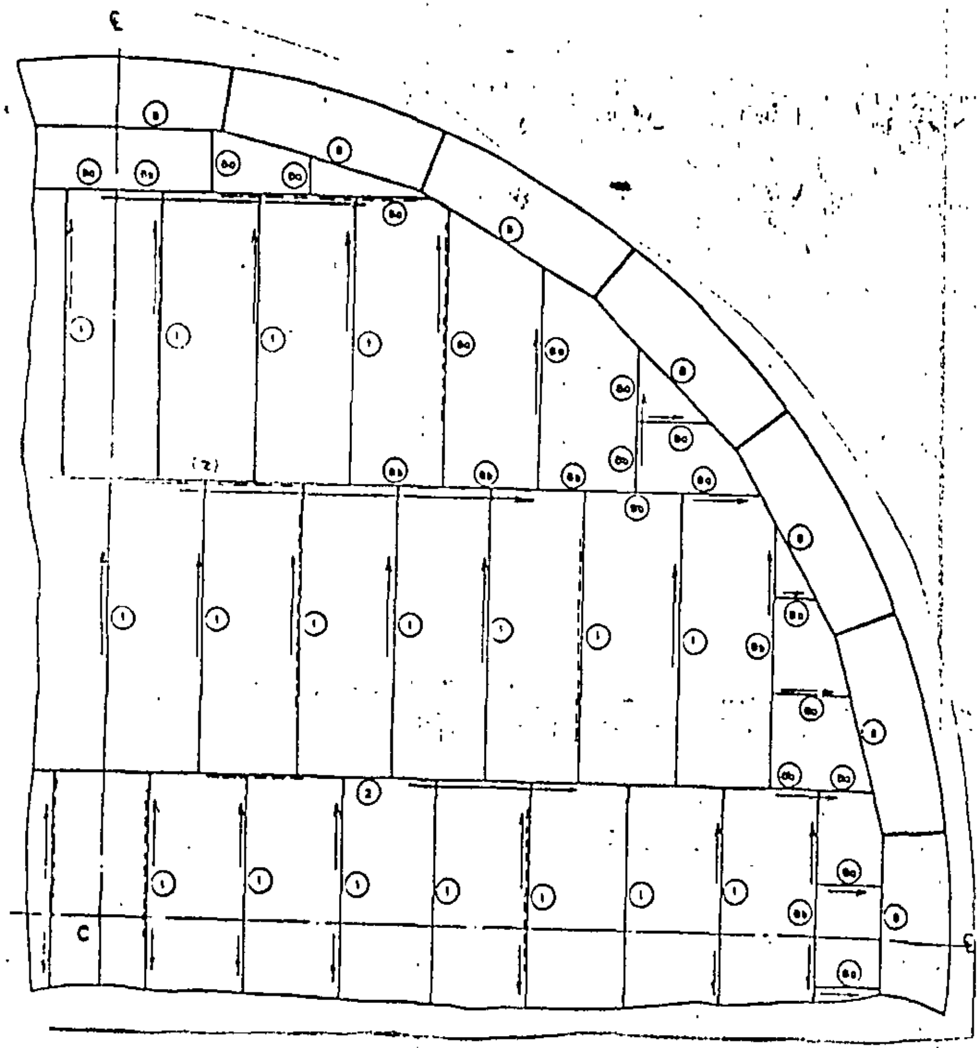
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN. 86	PLANO 2.005
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		



ESTE TIPO DE TENDIDO DE PLACAS ES IGUAL AL ANTERIOR (FORMANDO HILERAS LONGITUDINALES Y FILAS TRANSVERSALES); LA UNICA DIFERENCIA ES QUE ESTE TENDIDO ES PARA TANQUES DE MAYOR CAPACIDAD (250,000 a 500,000 BLS).

LA TECNICA DE SOLDEO SE DESCRIBE EN EL MANUAL DE MONTAJE Nº 1, SECCION 2.1.3.1 (TECNICA C.B.1)

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA Jun-76	PLANO 2-006
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		



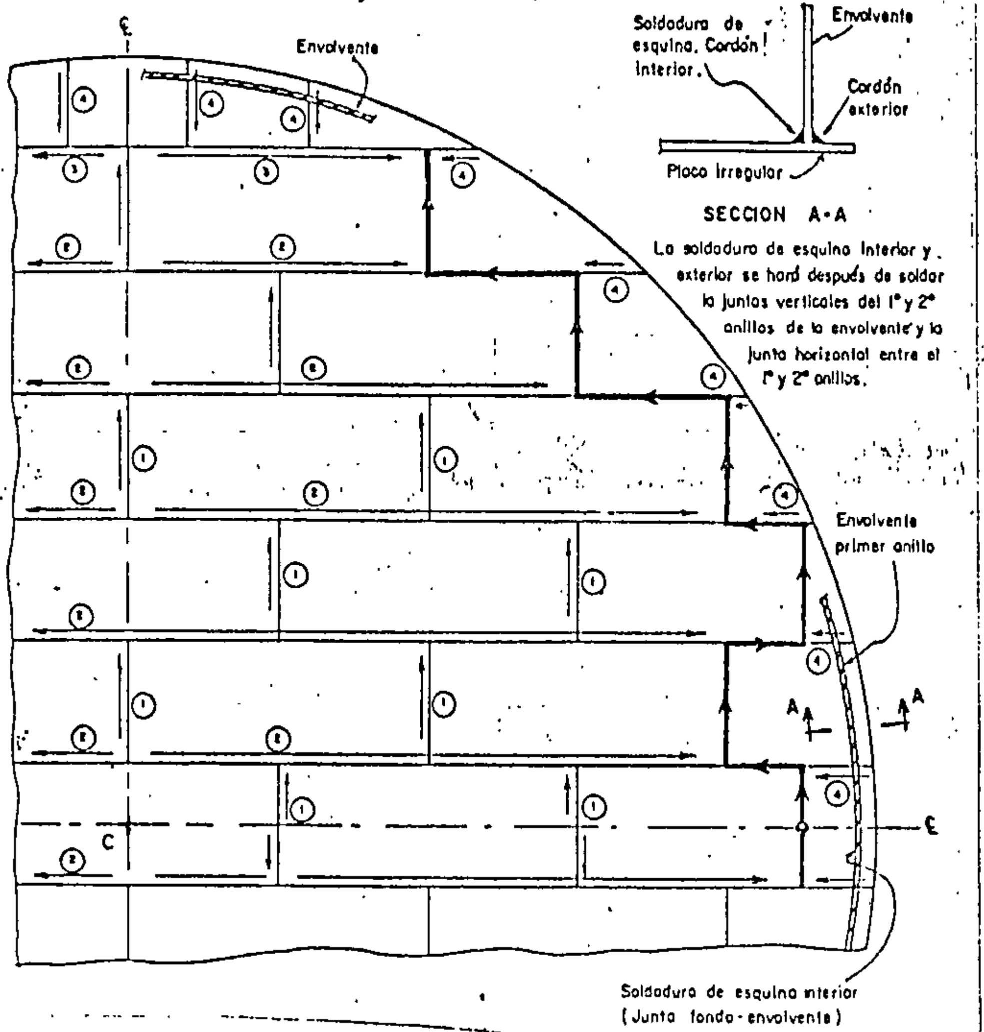
ARREGLO DEL TENDIDO Nº 2

FONDOS CON LAS PLACAS RECTANGULARES DISPUESTA SOLAMENTE EN HILERAS LONGITUDINALES CON PLACAS ANULARES EN LA PERIFERIA Y PLACAS IRREGULARES TRANSVERSALES (COMO SE INDICA EN LA FIGURA) PARA TANQUES DE MEDIANA CAPACIDAD (55,000 A 100,000 BLS.)

LA TECNICA DE SOLDEO ES LA UTILIZADA C.B.I, DESCRITA EN EL MANUAL DE MONTAJE Nº 1 SECCION 2.1.3.1

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN. 86	PLANO 2-007
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

NOTAS, a.- Fíjense las placas con el menor número de puntos de soldadura.
b.- Usese la técnica de retroceso en todas las costuras.
c.- Las soldaduras (4) se harán en cualquier tiempo después que la soldadura de esquina interior ha sido completada.
d.- El cordón marcado con línea grueso se hará al final.



TECNICAS DE SOLDEO PDM.

ESTA TECNICA SE UTILIZA EN EL ARREGLO DEL TENDIDO DE PLACAS CON HILERAS LONGITUDINALES Y FILAS TRANSVERSALES.

PARA EVITAR GRANDES DEFORMACIONES, DEBE SEGUIRSE LA SECUENCIA DE LA SOLDADURA MARCADA EN LAS JUNTAS CON NUMEROS PROGRESIVOS Y RESPETAR LA DIRECCION DEL AVANCE DEL SOLDEO MARCADO CON FLECHAS.

COMO COMPLEMENTO A LO ANTERIOR SIGASE LAS INSTRUCCIONES DADAS A CONTINUACION:

1. LAS COSTURAS ENTRE PLACAS RECTANGULARES MARCADAS (1) Y (2) PODRAN SOLDARSE SIN RELACIONARLAS CON LAS SOLDADURAS 4 ENTRE PLACAS IRREGULARES.

2. LAS COSTURAS (4) DE LAS PLACAS IRREGULARES SE SOLDARAN EN CUALQUIER TIEMPO DESPUES QUE EL CORDON INTERIOR DE LA JUNTA DE FONDO-ENVOLVENTE HA SIDO SOLDADA COMPLETAMENTE.

3. LAS COSTURAS (3) DE LAS PLACAS RECTANGULARES, DEBERAN SOLDARSE ANTES QUE LAS JUNTAS ENTRE LAS IRREGULARES Y LAS RECTANGULARES MARCADAS CON LINEAS MAS GRUESAS SEAN SOLDADAS. LAS DEMAS SOLDADURAS (1), (2) Y (4) SE COMPLETARAN ANTES QUE SE SUELDE LA (3).

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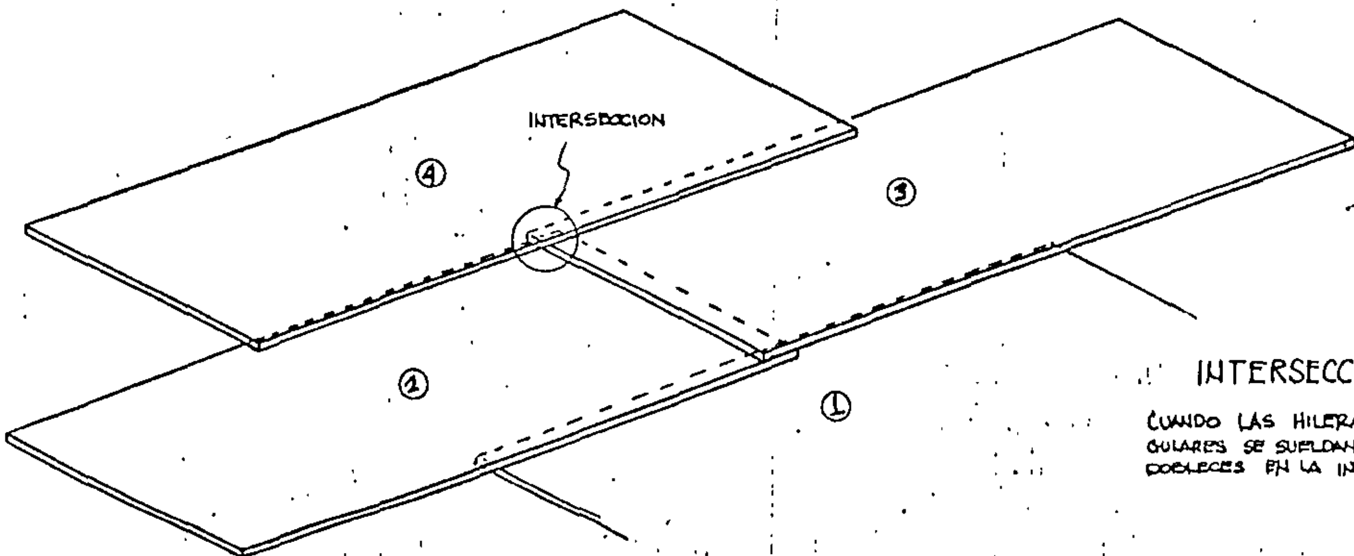
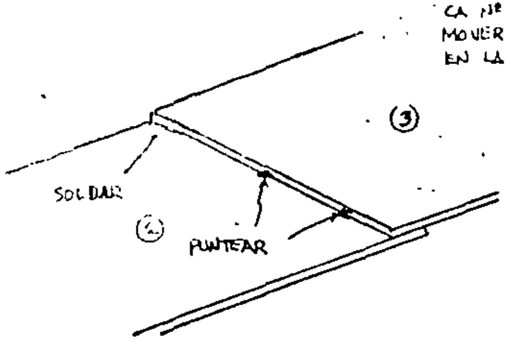
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE N° 1		

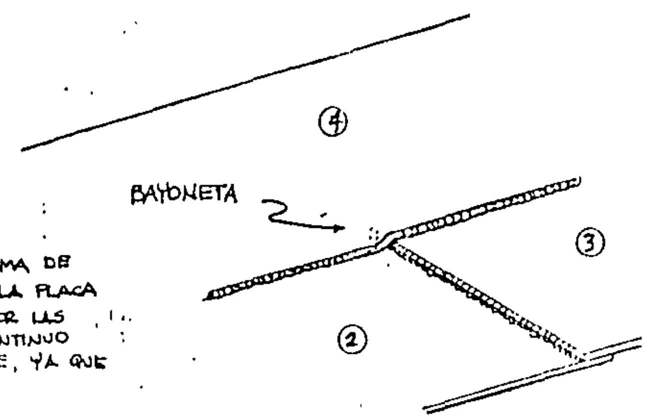


INTERSECCION DE TRES PLACAS

CUANDO LAS HILERAS Y PILAS DE PLACAS RECTANGULARES SE SUELDAN ENTRE SI, SE REQUIERE HACER DOBLECES EN LA INTERSECCION Y DESPUES SOLDAR.

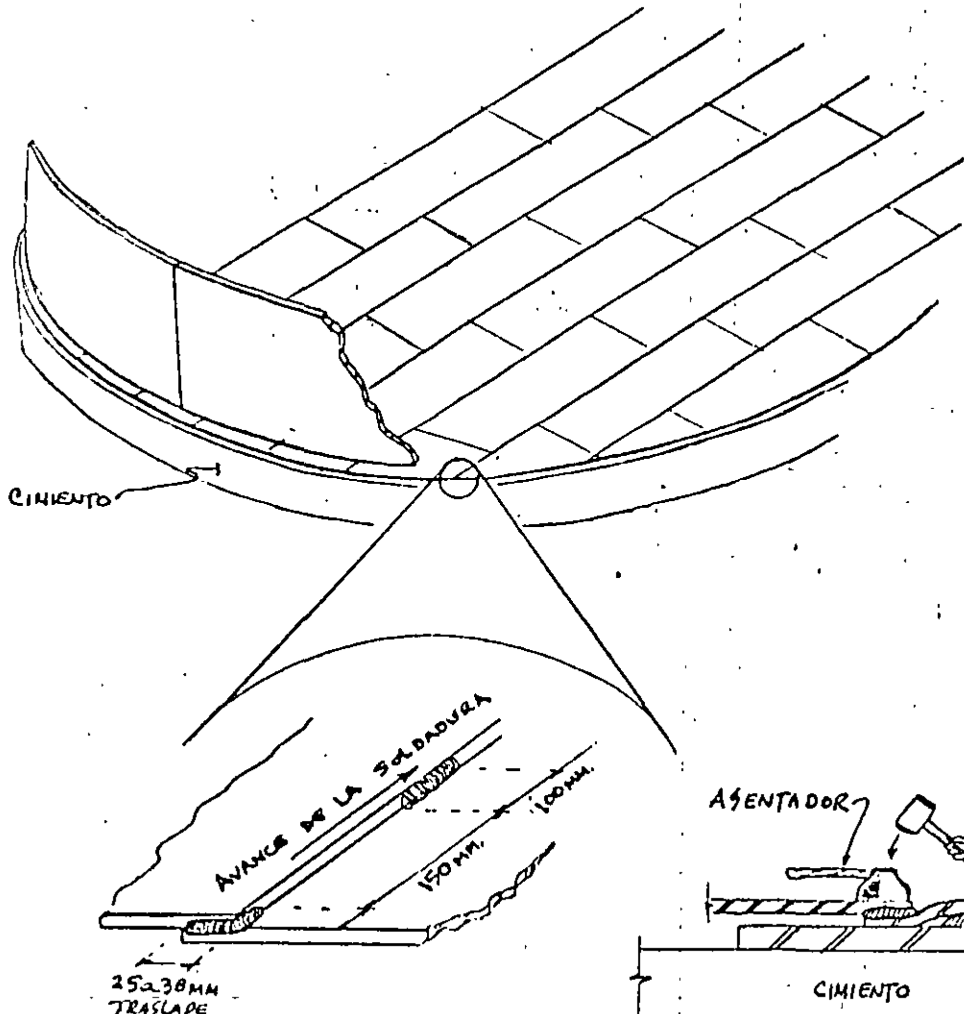
SOLDAR LA PORCION DE LAS PLACAS ② Y ③ QUE QUEDARA CUBIERTA POR LA Nº ④ Y PUNTEAR EL RESTO DE ESTA UNION ANTES DE TENDER LA PLACA Nº ④ ; SI ESTA PORCION NO FUE SOLDADA, REMOVER LA PLACA ④ PARA SOLDAR TODO EL CORDON EN LA UNION DE PLACAS ② Y ③.





DOBLECE LA PLACA Nº ④ EN FORMA DE BAYONETA PARA QUE ASIENTE SOBRE LA PLACA ② Y SUELDECE LA JUNTA FORMADA POR LAS PLACAS ④.③ Y ④.② DE MODO CONTINUO SIN INTERRUMPIRLA EN EL TRASLAPE, YA QUE SE PODRIA ORIGINAR UNA GRIETA.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. / APROBADO POR: Ing. J. H. B.	FECHA JUN. 66	PLANO 2-010
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		



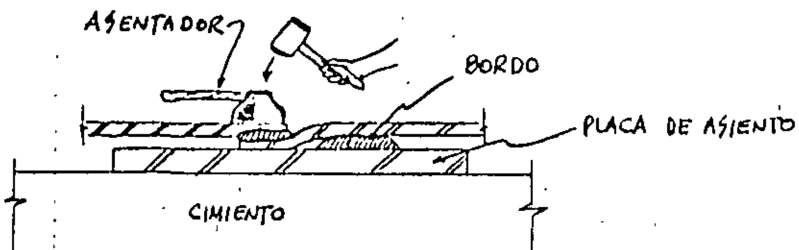
AJUSTE Y SOLDEO DE ESQUINAS BAYONETADAS

CUANDO EL DISEÑO DE UN FONDO NO CONTEMPLA PLACAS ANULARES, LA ENVOLVENTE SE APOYARA EN PLACAS IRREGULARES TRASLAPADAS, PERO PARA MANTENER NIVELADA TODA LA PERIFERIA PARA ASENTAR LA ENVOLVENTE, ES NECESARIO MODIFICAR EL TRASLAPE EN LAS ESQUINAS EXTREMAS EXTERIORES FORMANDO UN CONJUNTO MACHI-HEMBRADO.

SUELDESE EL LADO EXTERIOR DE FUERA HACIA EL CENTRO, OMITIR 150 MM. (6") Y SOLDAR 100 MM. MAS (4") COMO SE INDICA EN LA FIGURA.

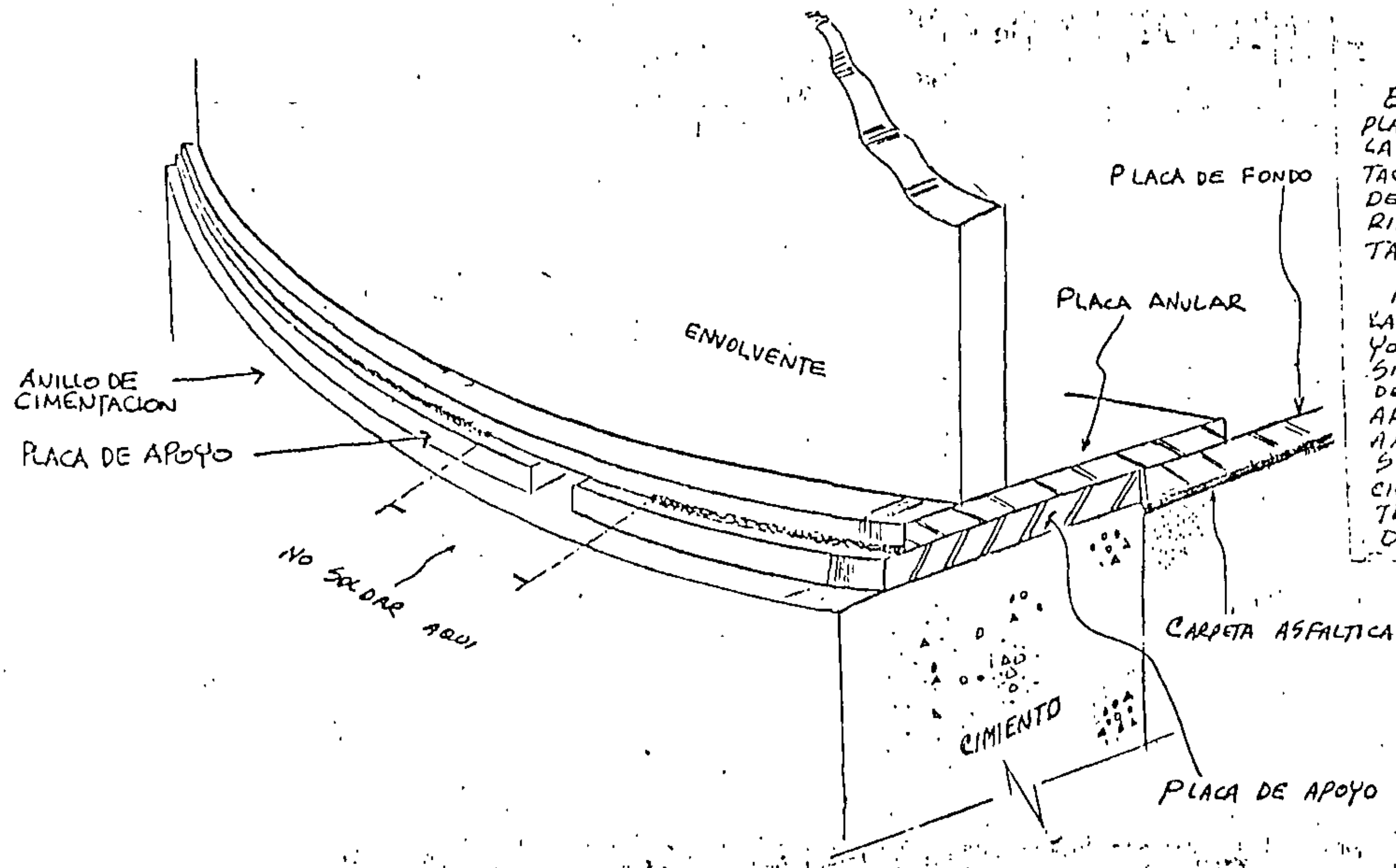
COLOCAR UNA PLACA DE ASIENTO PROVISIONAL COMO UN BORDO COMO LA MOSTRADA EN LA FIGURA Y GOLPEAR EL TRASLAPE HACIA ABAJO HASTA QUE LAS CARAS SUPERIORES DE DOS PLACAS IRREGULARES ADYACENTES ESTEN A NIVEL.

COMPLETAR LA SOLDADURA EN EL AREA DOBLADA, USANDO EL REQUERIDO NUMERO DE PASADAS (DOS MINIMAS) PARA HACER UN TRASLAPE SOLDADO COMPLETO Y RETIRAR LA PLACA PROVISIONAL DE ASIENTO, PARA QUE LAS IRREGULARIDADES SE APOYEN EN EL ANILLO DE CIMENTACION.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I.J.L. / APROBADO POR Ing. J.H.D.	FECHA JUN. 86	PLANO 2-011
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE Nº 1		

PLACAS DE APOYO

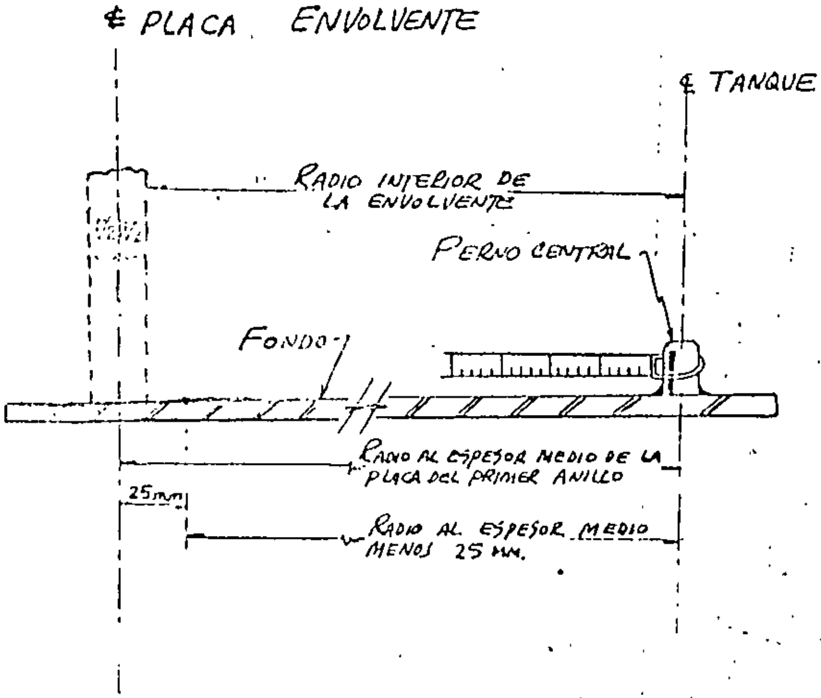


EN OCASIONES SE UTILIZA ESTE TIPO DE PLACAS, LAS CUALES SE COLOCAN ENTRE LA PLACA ANULAR Y EL ANILLO DE CIMENTACION, BAJO LA ENVOLVENTE. EL OBJETO DE ESTE TIPO DE PLACA ES TRANSFERIR LA CARGA DE LA MISMA A LA CIMENTACION.

NO SE SUELDEN LAS PLACAS UNA A LA OTRA AL SOLDAR LA PLACA DE APOYO A LA ANULAR DEL FONDO, DEJESE SIN SOLDAR UNA PARTE EN LA JUNTA DE SEPARACION DE LAS PLACAS DE APOYO. EN LA FIGURA SE INDICA EL AREA DONDE DEBE OMITIRSE LA SOLDADURA PARA EVITAR LA FORMACION DE GRIETAS EN LA ENVOLVENTE CUANDO SE SUELDE DE CORRIDO.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. / APROBADO POR: Ing. J. H. B.	FECHA JUN 86	PLANO 2-012
SECCION 2.0 MONTAJE DEL FONDO	MANUAL DE MONTAJE N° 1		

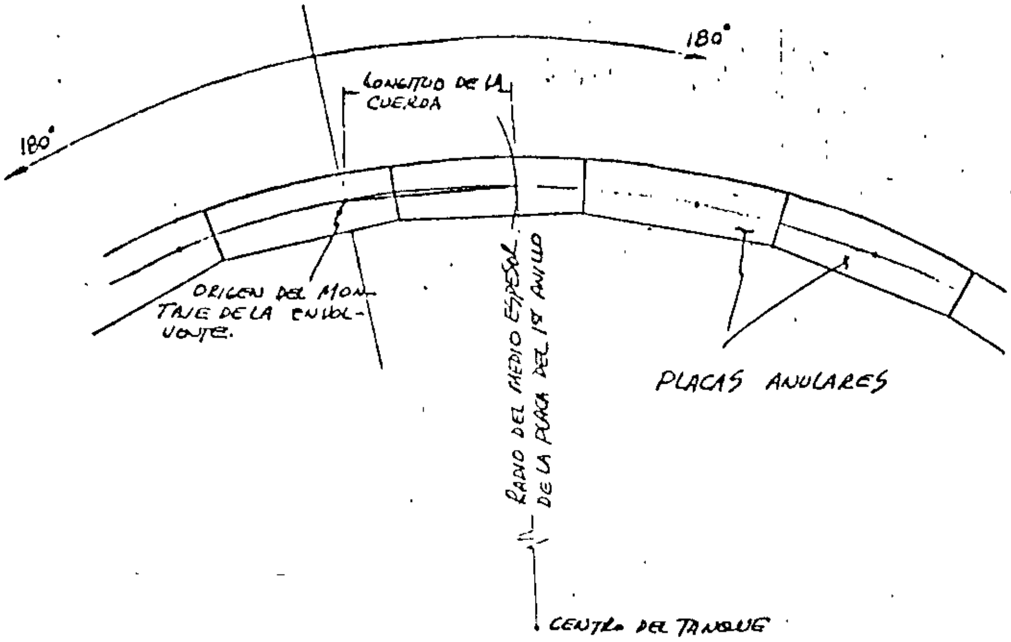
ERECCION DE LA ENVOLVENTE



TRAZOS AUXILIARES EN LA ERECCION

1.- ENGANCHAR LA ARGOLLA EXTREMA DE UNA CINTA METALICA DE MEDIR EN EL PERNO SOLDADO EN LA PLACA CENTRAL DEL FONDO Y TRAZAR TRES CIRCULOS CONCENTRICOS DE REFERENCIA: EL PRIMERO CON UN RADIO AL MEDIO ESPESOR DE LAS PLACAS DEL PRIMER ANILLO DE LA ENVOLVENTE, EL SEGUNDO CON EL RADIO INTERIOR DEL TANQUE Y EL ULTIMO CON UN RADIO DE 25 MM MENOR QUE EL SEGUNDO.

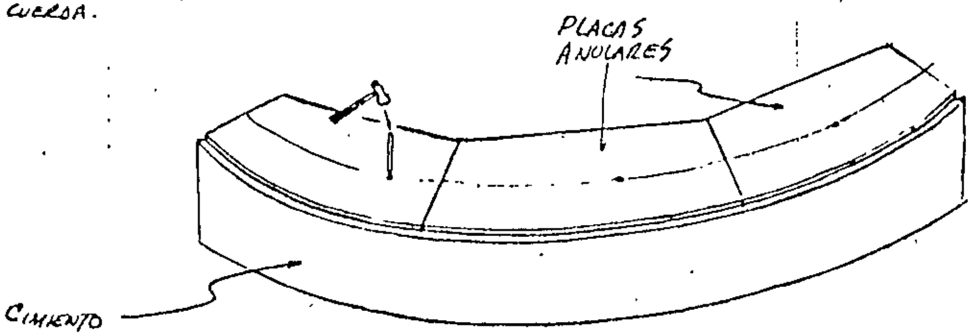
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUNIO 8.	PLANO 3-0.1
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE N° 1		



2.- EL TRAZO DE CUERDAS SE REALIZA MEDIANTE EL EMPLEO DE DOS CINTAS DE MEDIR. MIENTRAS QUE CON UNA SE ESTA MIDIENDO EL RADIO DEL MEDIO ESPESOR SOBRE EL CIRCULO CORRESPONDIENTE, CON LA OTRA SE MIDE AL MISMO TIEMPO LA LONGITUD DE LA CUERDA DESDE EL TRAZO ANTERIOR.

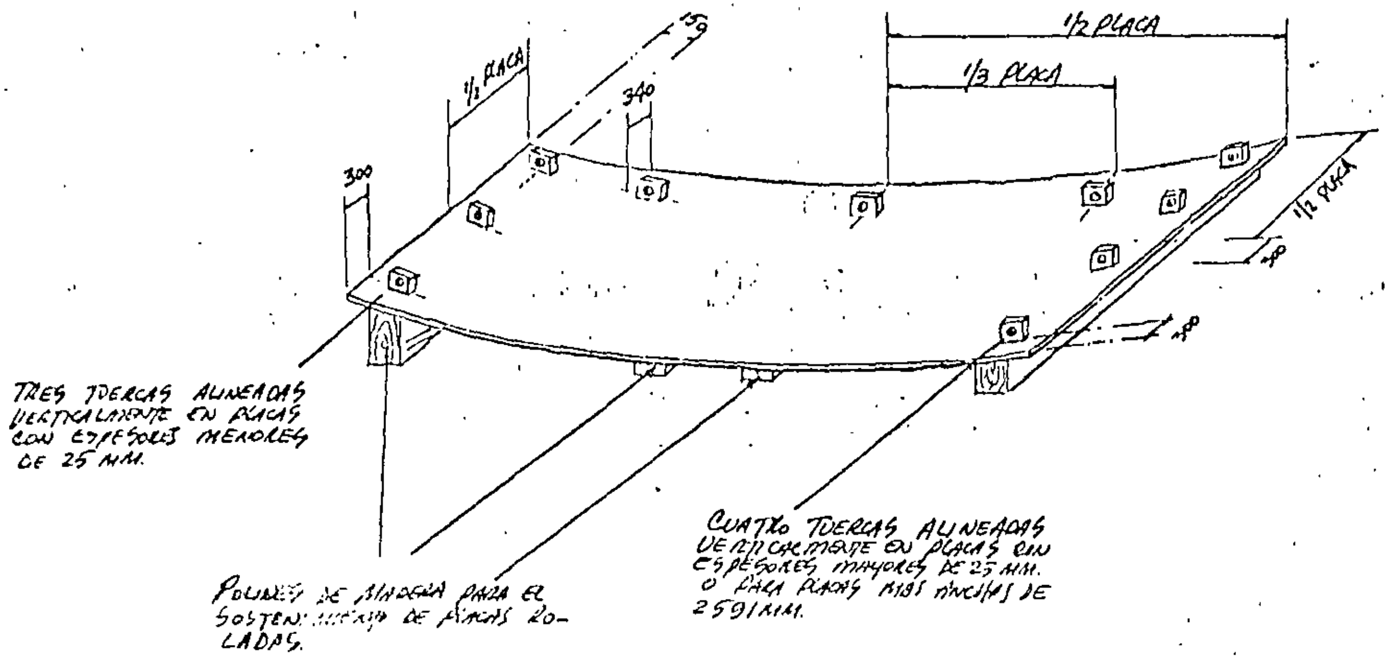
EMPEZANDO EN EL PUNTO DONDE SE INICIA EL MONTAJE DE LA ENVOLVENTE, TRAZAR LOS EXTREMOS DE LAS CUERDAS DE TODAS LAS PLACAS DEL PRIMER ANILLO TRABAJANDO INDEPENDIENTEMENTE LAS DOS MEDIAS CIRCUNFERENCIAS EN DIRECCIONES OPUESTAS. SI LAS LOCALIZACIONES FINALES EN CADA DIRECCION NO COINCIDEN, DIVIDIR EL ERROR ENTRE EL NUMERO DE CUERDAS, INCREMENTAR SU LONGITUD CON EL COCIENTE QUE RESULTE Y TRAZARLAS NUEVAMENTE, REPETIR ESTA OPERACION HASTA QUE NO HAYA ERROR.

MARCAR CON PUNTO Y MARTILLO LOS TRAZOS EXTREMOS DE CADA CUERDA.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I.J.L. / APROBADO POR Ing. J.H.B.	FECHA Jun-'82	PLANO 3 de 2
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE No 1		

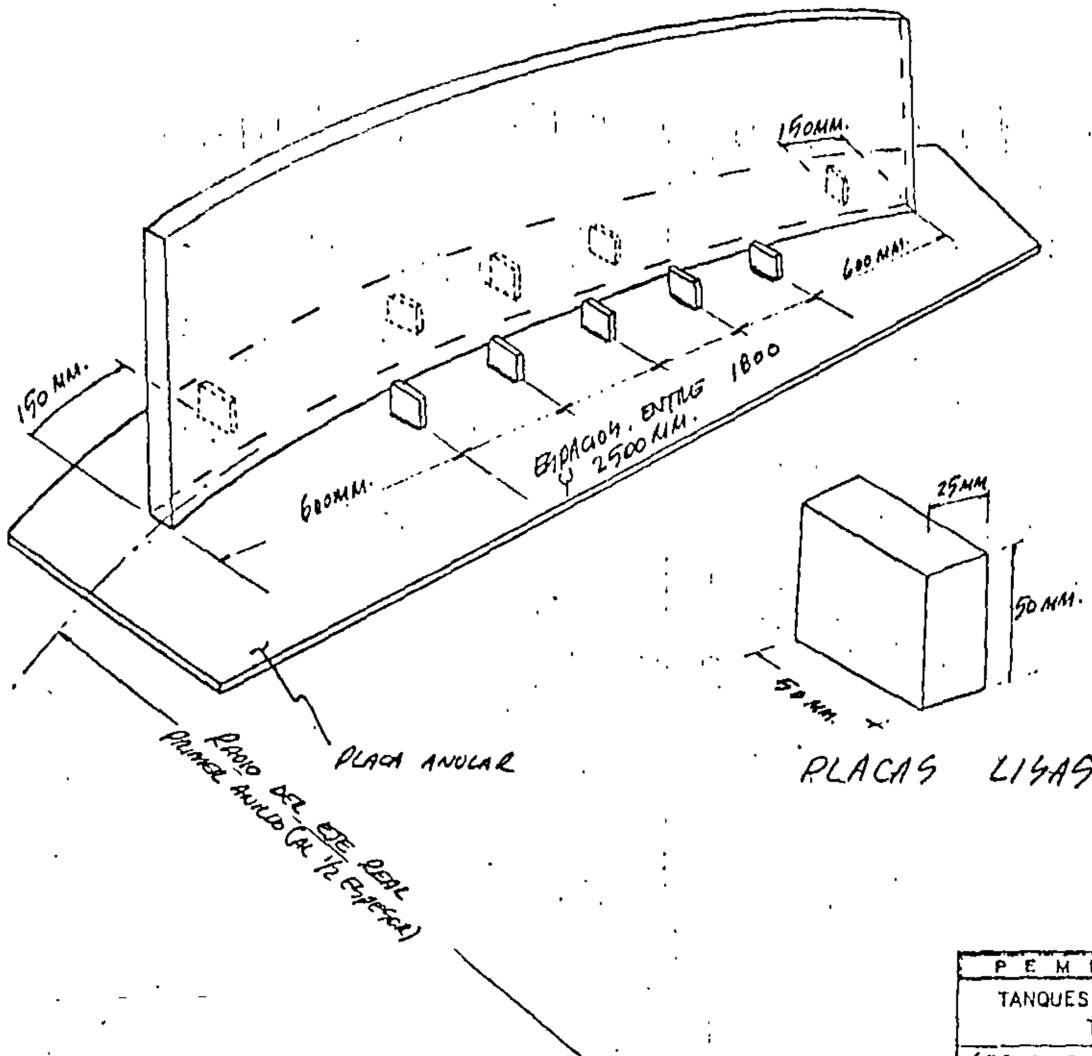
SOLDAR EN CADA PLACA DE LA ENVOLVENTE, LAS TUERCAS LISAS PARA LOS CANDADOS SUJETADORES CORRESPONDIENTES A LAS JUNTAS VERTICALES Y PARA LOS RIGIDIZANTES EN LAS JUNTAS HORIZONTALES.



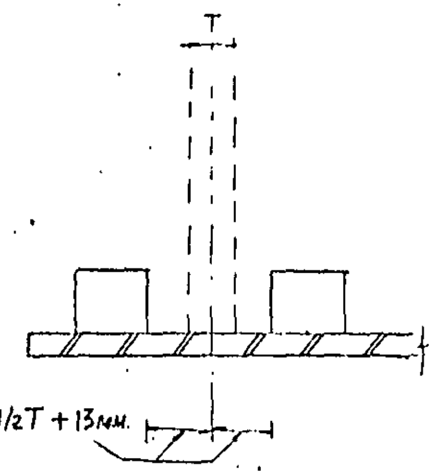
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN-86	PLANO 3-003
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE Nº 1		

PUNTEAR POR PARES EN LAS PLACAS O IRREGULARES DEL FONDO UNA SERIE DE TUERCAS LISAS DE 50x50x25 mm. SEPARADAS DEL EJE DE LA ENVOLVENTE HACIA EL EXTERIOR Y EL INTERIOR, UN MEDIO ESPESOR DE LA PLACA DEL PRIMER ANILLO MAS 13 MM.

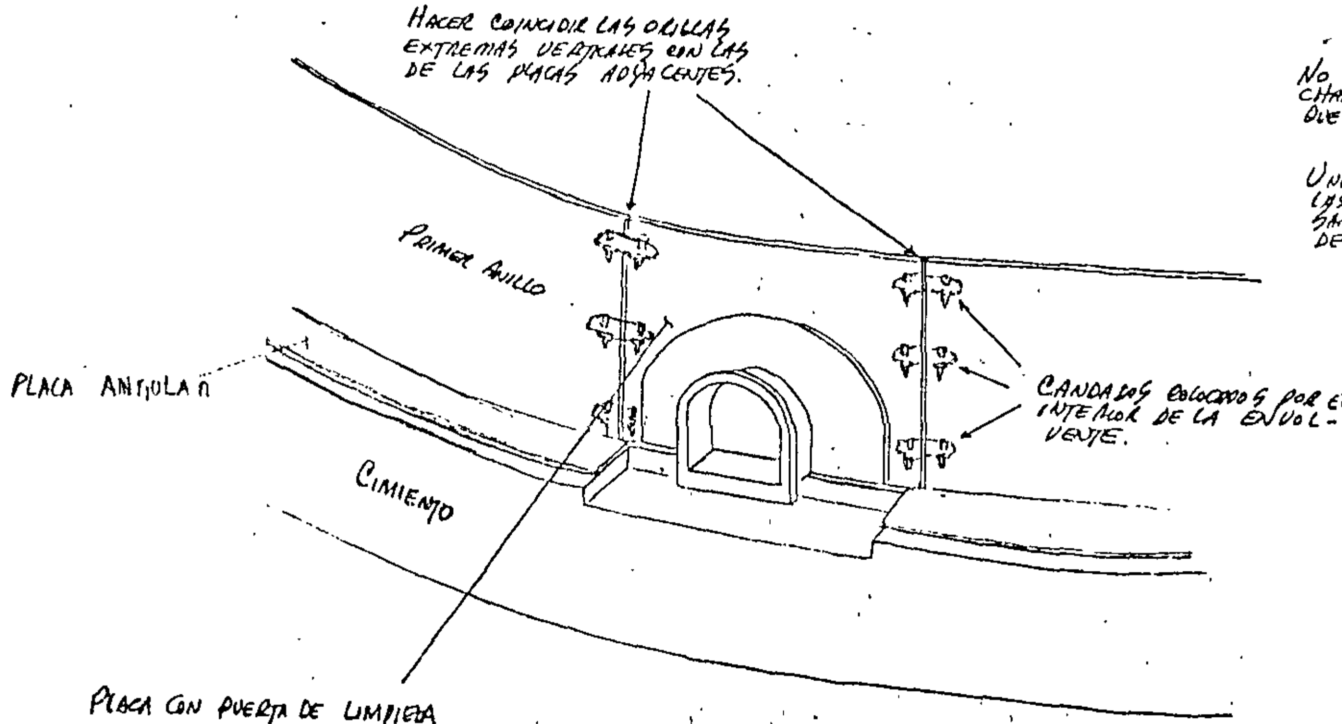
LOCALIZAR PRIMERO DOS TUERCAS A 150 MM DE CADA EJE POR EL LADO EXTERIOR Y A 600MM. POR EL INTERIOR, DESPUES EL RESTO DE LA SERIE A INTERVALOS NO MENORES DE 1800 A 2500 MM.



PLACAS LISAS



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA Jun-86	PLANO 3-004
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE N° 1		



PRESENTAR LAS PLACAS EN SU UBICACION CORRECTA. MANEJESE EN LA MISMA FORMA QUE LAS DEMAS PLACAS DE LOS ANILLOS.

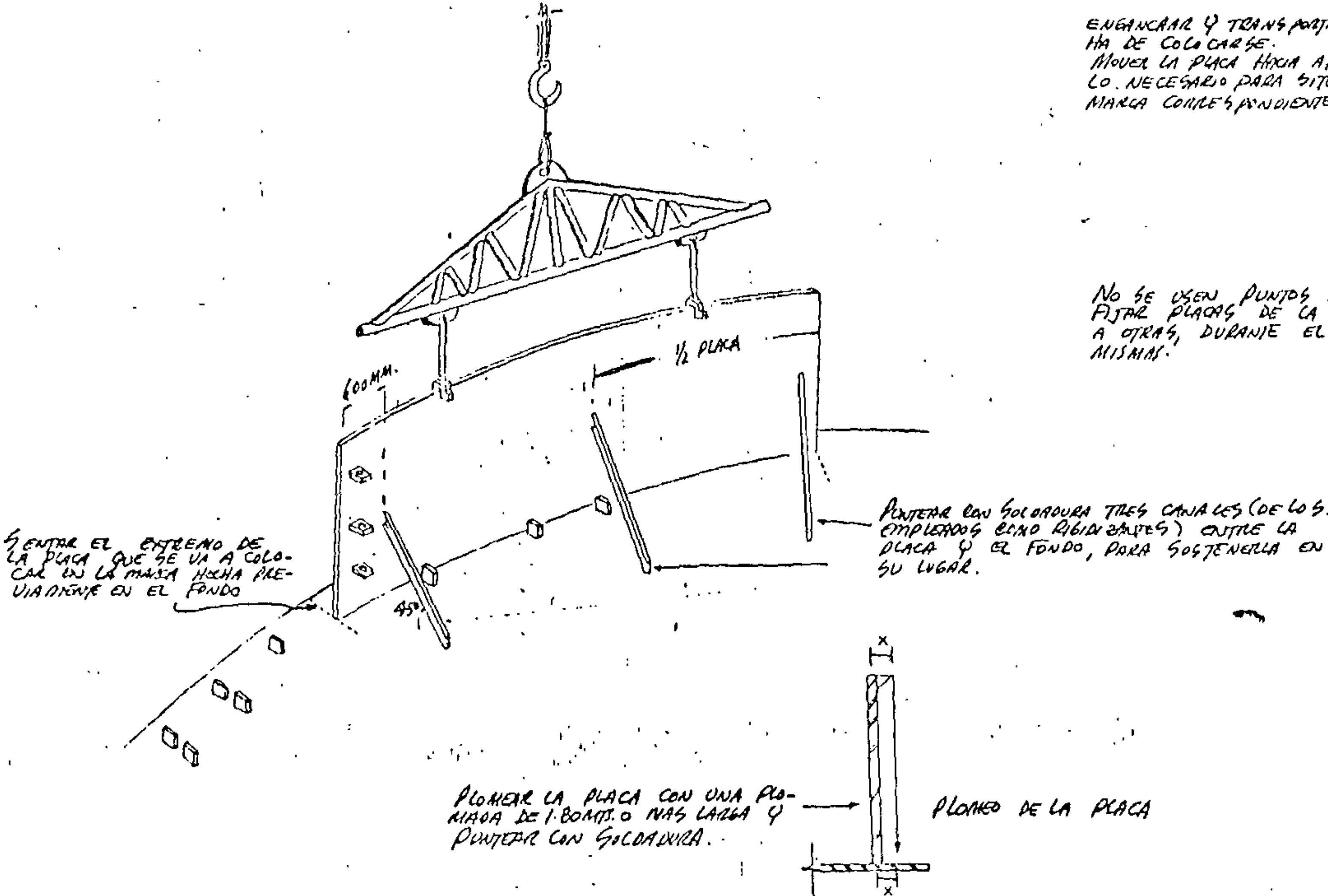
NO USAR PLACAS SEPARADORAS. NO DESENGANCHAR EL EQUIPO DE LEVANTAMIENTO, HASTA QUE LOS CANDADOS ESTEN BIEN APRETADOS.

UNA VEZ PRESENTADAS AJUSTADAS Y SUJETADAS LAS PLACAS DE LA PUERTA DE LIMPIEZA, REVISAR LA INSTALACION Y LIBERESE EL EQUIPO DE MONTAJE.

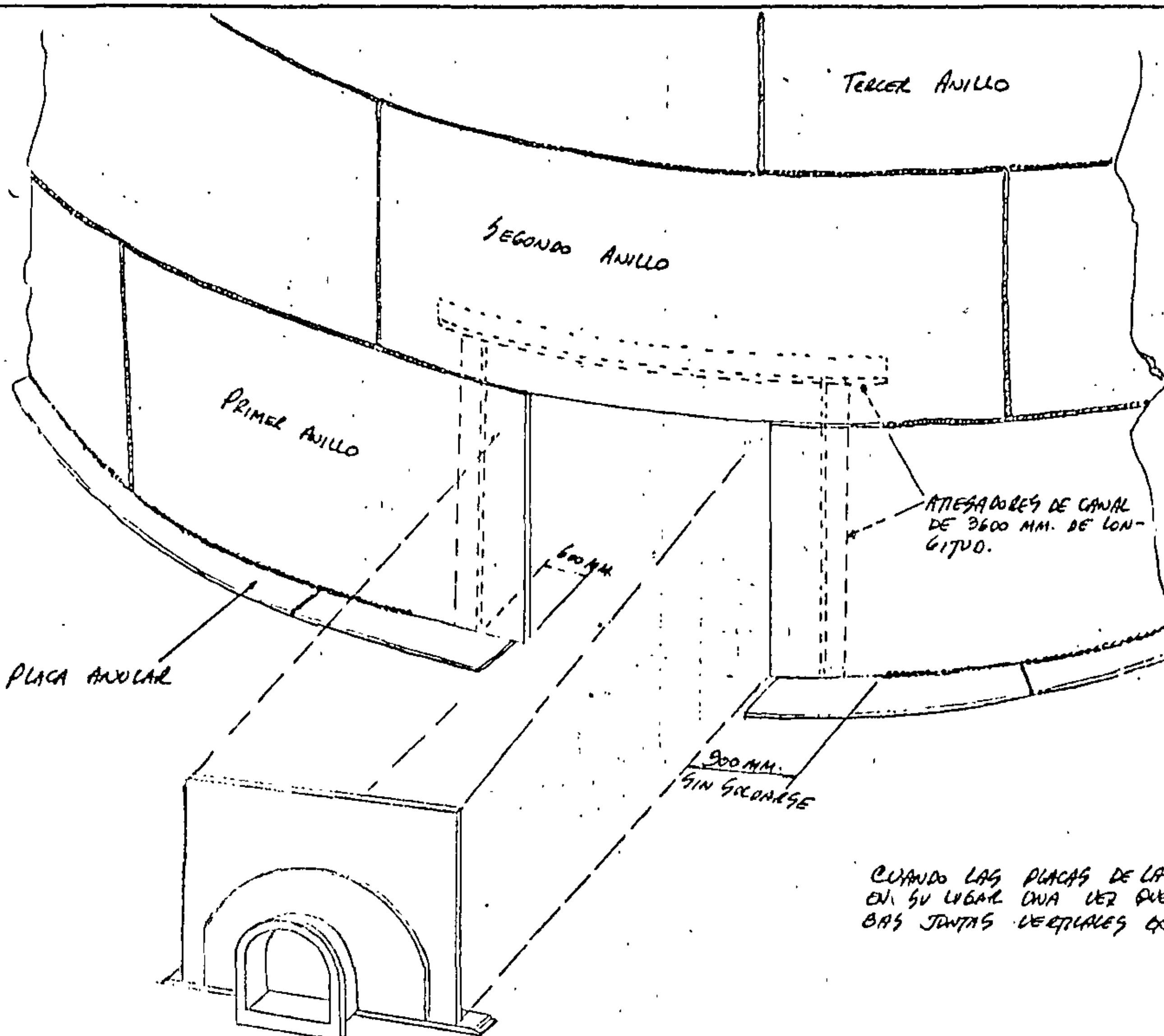
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN-86	PLANO 3-006
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE N° 1		

ENGANCHAR Y TRANSPORTAR LA PRIMERA PLACA QUE HA DE COLOCARSE.
MOVER LA PLACA HACIA AFUERA O HACIA ADENTRO LO NECESARIO PARA SITUAR EL EXTREMO EN LA MARCA CORRESPONDIENTE.

NO SE USEN PUNTOS DE SOLDADURA PARA FIJAR PLACAS DE LA ENVOLVENTE UNAS A OTRAS, DURANTE EL MONTAJE DE LAS MISMAS.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA JUN - 83	ARCHIVO 3-105
SECUENCIA DE ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE Nº 1		



PARA CONTAR CON UNO O MAS ACCESOS HACIA EL INTERIOR DEL TANQUE HAY NECESIDAD DE REMOVER LAS PLACAS DE LAS PUERTAS. ESTAS PLACAS NO SE QUITARAN HASTA QUE LAS OPERACIONES SIGUIENTES HAYAN SIDO EJECUTADAS:

1.- DOS ANILLOS SUPERIORES, CUANDO MENOS, DEBERAN ESTAR TOTALMENTE SOLDADOS.

2.- LA JUNTA CIRCUNFERENCIAL FONDO - ENVOLVENTE Y LA PRIMERA JUNTA HORIZONTAL ENTRE EL PRIMERO Y EL SEGUNDO ANILLO ESTEN SOLDADAS EXCEPTO 900 MM. MINIMOS POR CADA LADO DE LA PLACA. (VER FIGURA).

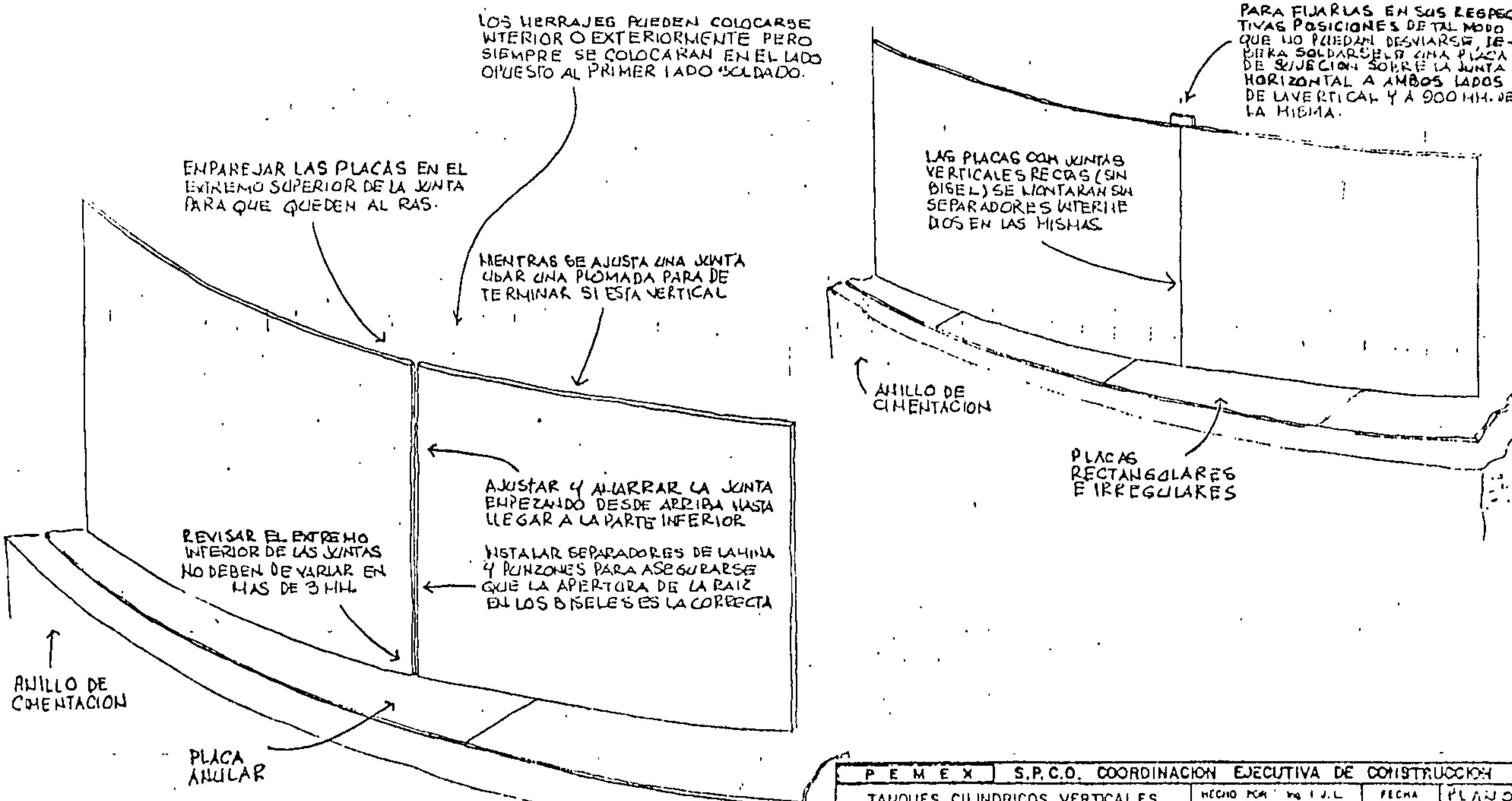
3.- LA ABERTURA QUE DEJA LA PLACA AL RETIRARLA HA SIDO PERFECTAMENTE ATIESADA CON CANALES DE 3.5M DE LONGITUD MINIMOS.

4.- NO EMPEZAR A QUITAR CANDADOS NI PLACAS DE SUJECION, HASTA QUE EL EQUIPO DE IZAJE ESTE ENGANCHADO.

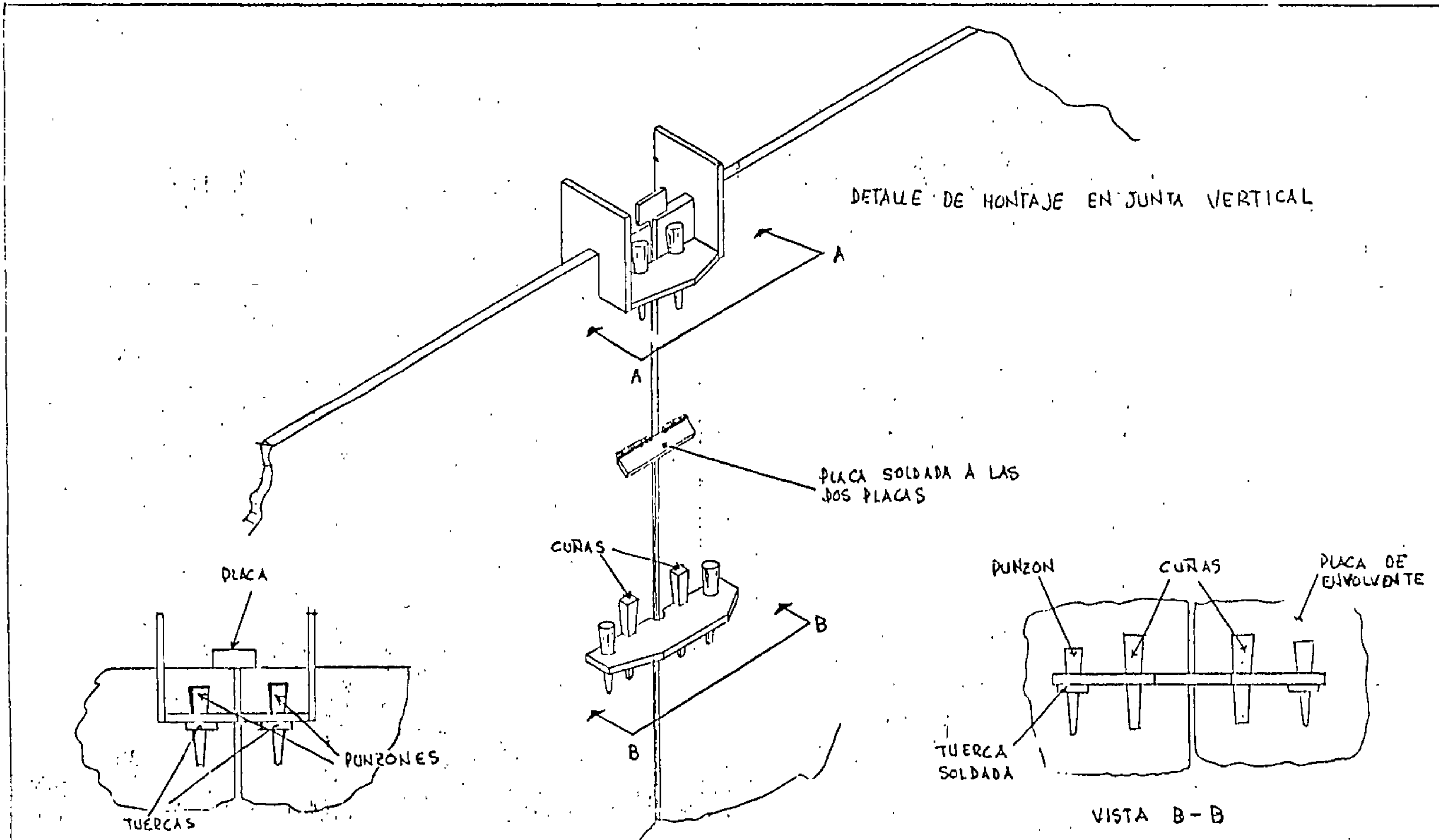
CUANDO LAS PLACAS DE LAS PUERTAS SE COLOCAN EN FORMA DEFINITIVA EN SU LUGAR UNA VEZ QUE SE TERMINO EL MONTAJE, FIJARLAS EN AMBAS JUNTAS VERTICALES EXTREMAS MEDIANTE PLACAS SEPARADORAS.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I.J.L. AVALADO POR Ing. J.H.B.	FECHA JUN-86	FOJA 3-007
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE No 1		

UNION Y SOLDEO DE JUNTAS VERTICALES

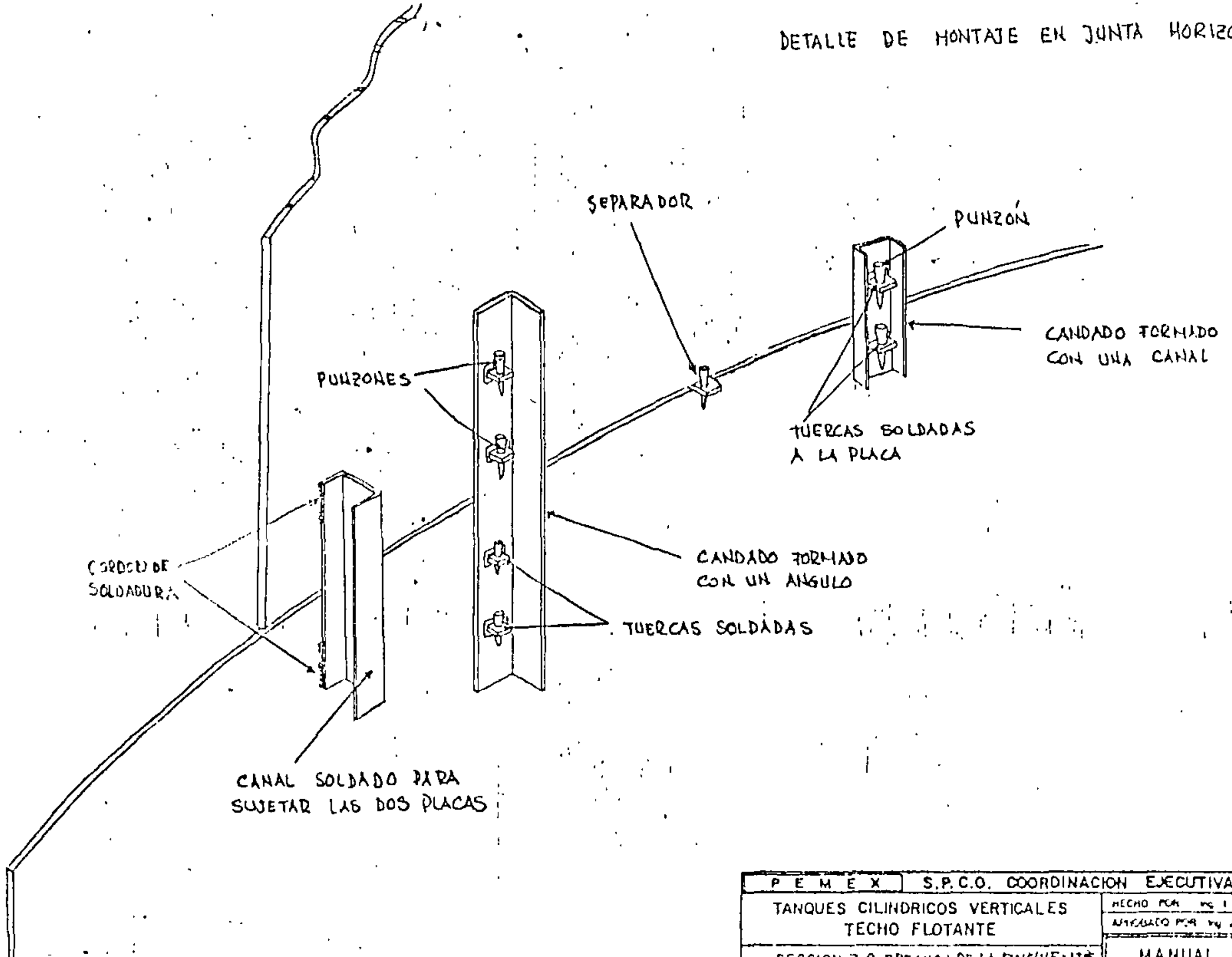


P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION			
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE		HECHO POR: Ing. I. J. L. / APROBADO POR: Ing. J. M. B.	FECHA JUN 86	PLANO 3-008
SECCION 30 ERECCION DE LA ENVOLVENTE		MANUAL DE MONTAJE N° 1		

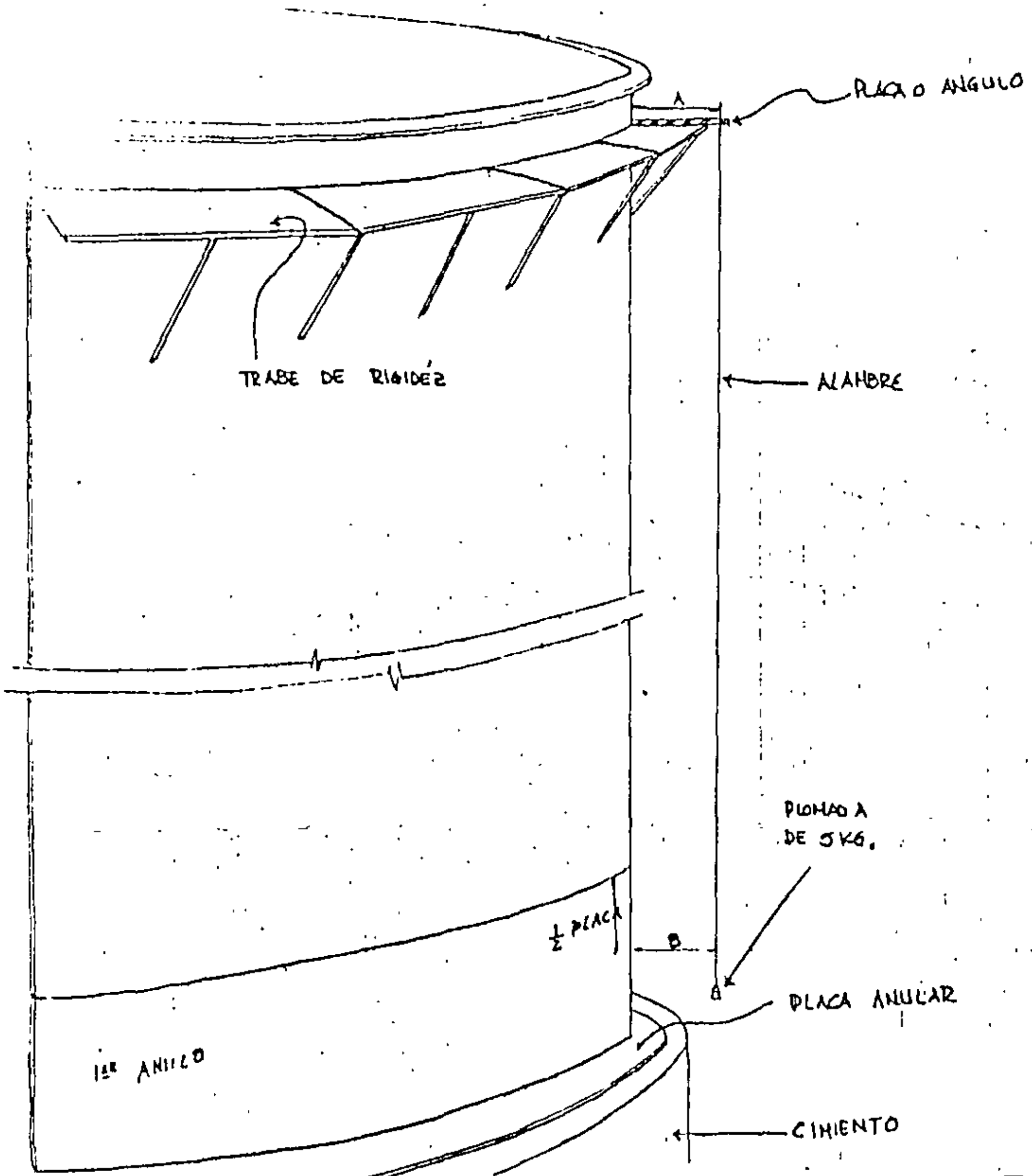


P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. R. B.	FECHA JUN-86	[illegible]
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE Nº 1		

DETALLE DE MONTAJE EN JUNTA HORIZONTAL



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR / REVISADO POR	FECHA JUN-66	PLANO 3 010
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE No 1		



METODO PARA REVISAR LA REDONDEZ DE LA ENVOLVENTE DE TANQUES ABIERTOS EN SU PARTE SUPERIOR DONDE SE INSTALAN LOS TECHOS FLOTANTES.

LA SECUENCIA A SEGUIR PARA LA APLICACION DE ESTE METODO, SE DESCRIBE EN EL MANUAL DE MONTAJE No. 1

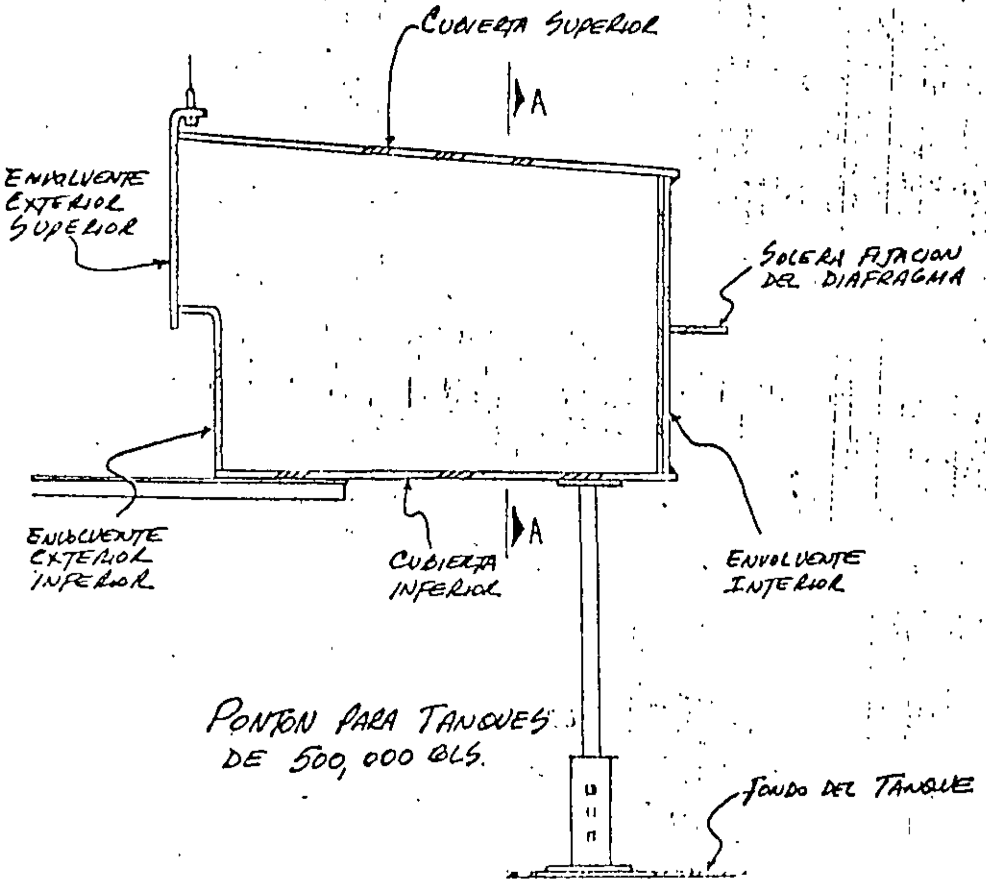
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR, Ing. J. M. B.	FECHA JUN-86	PLANO 3.011
SECCION 3.0 ERECCION DE LA ENVOLVENTE	MANUAL DE MONTAJE Nº 1		

ARMADO DEL PONTON

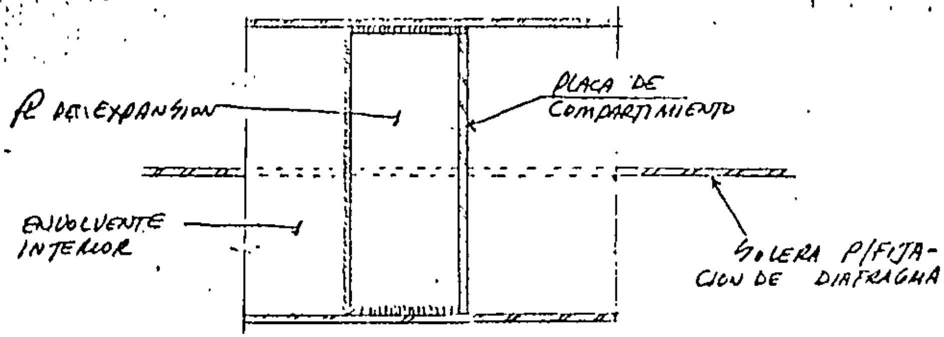
A.- INICIAR EL ENSAMBLE POR SECCIONES, TEDIENDO LAS PLACAS DE LA CUBIERTA INFERIOR DE CADA SECCION, CALZARLAS PARA PONERLAS A NIVEL Y UNIRLAS ENTRE SI PUNTEANDO LAS JUNTAS RADIALES.

B.- COLOCAR LA ENVOLVENTE EXTERIOR INFERIOR PUNTEANDOLA Y EN SEGUIDA LA ENVOLVENTE SUPERIOR.

C.- COLOCAR PLACAS DIVISORIAS DE LOS COMPARTIMIENTOS DEL PONTON PUNTEANDOLAS A SU ENVOLVENTE Y A LA CUBIERTA INFERIOR.

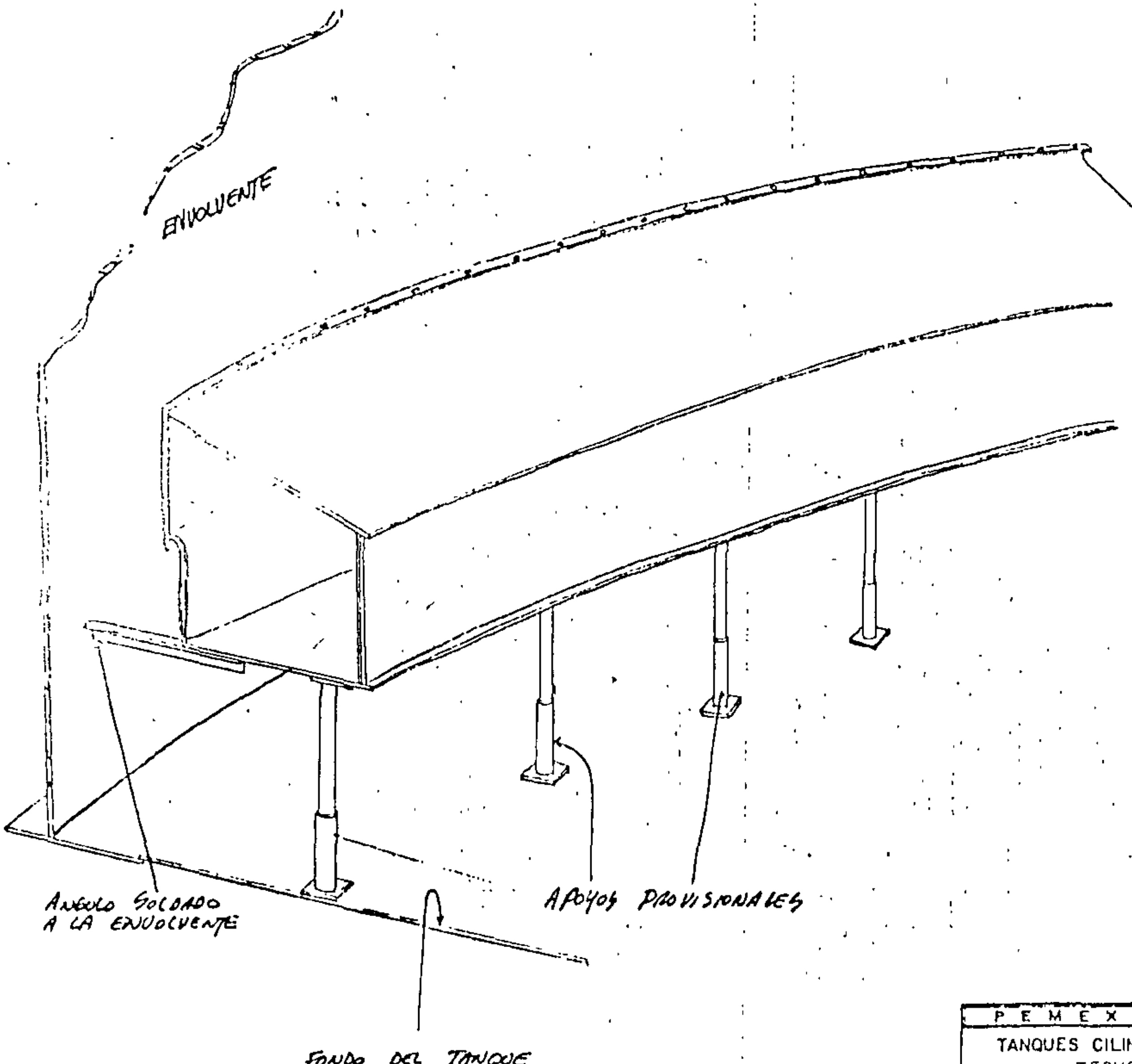


PONTON PARA TANQUES DE 500,000 BLS.



SECCION "A-A"

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / ARMADO POR Ing. J. H. B.	FECHA JUN-86	PLANO J-001
SECCION 4.0 MONTAJE DEL TECHO FLOTANTE	MANUAL DE MONTAJE N° 1		



D.- COLOCAR Y PUNTEAR EN LA MISMA FORMA QUE LA ENVOLVENTE EXTERIOR LA INTERIOR, ASI COMO LAS PLACAS DE EXPANSION.

E.- MONTAR, AJUSTAR Y PUNTEAR EL SECTOR SUPERIOR A LAS ENVOLVENTES EXTERIOR E INTERIOR. PUNTEAR LAS PLACAS DIVISORIAS DE LOS COMPARTIMIENTOS CONFORME SE VAYA CERRANDO EL PONTON. CUIDAR DE QUE NO COINCIDAN LAS JUNTAS VERTICALES DE LAS ENVOLVENTES CON LAS UNIONES RADIALES.

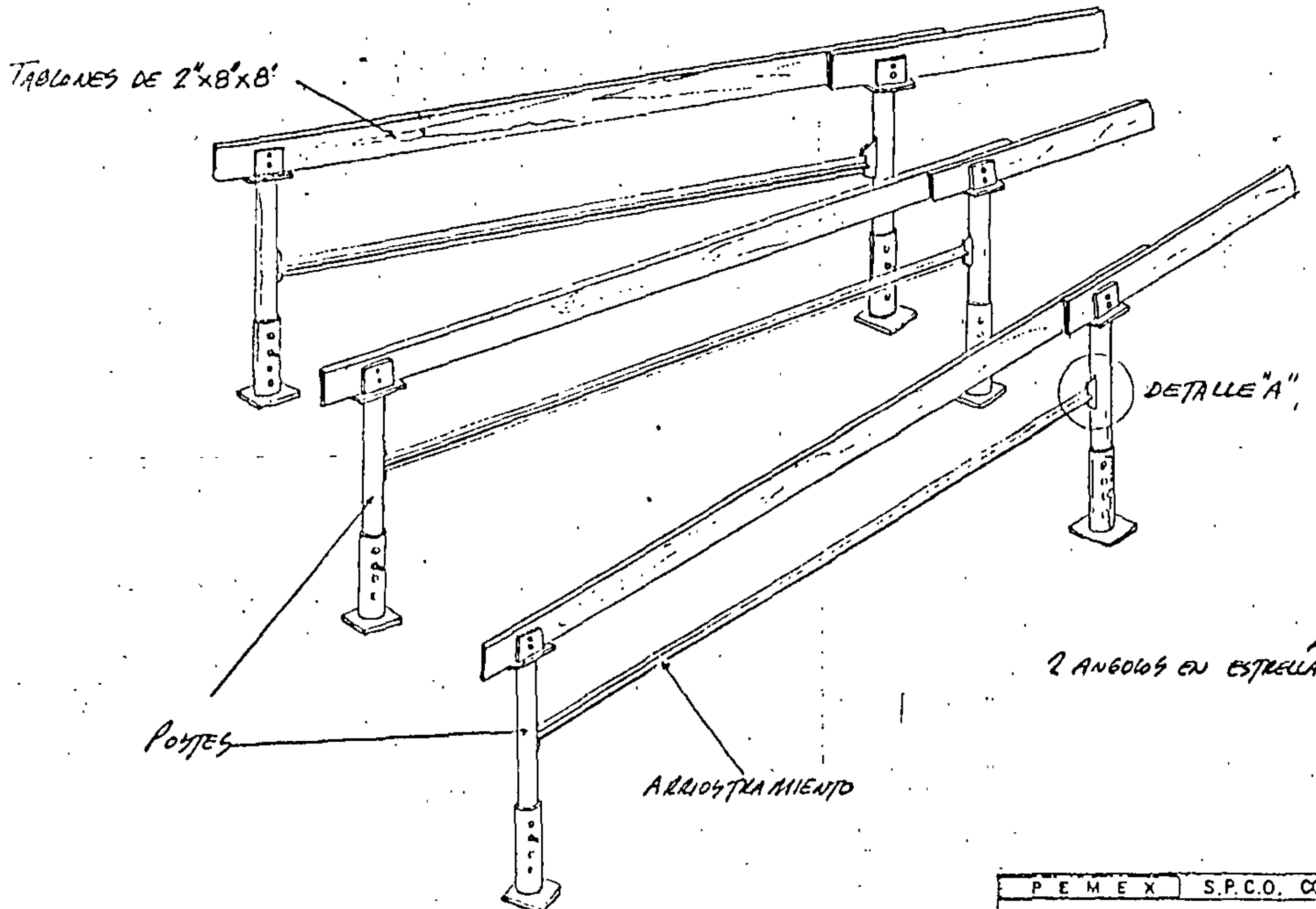
F.- SEGUIR LA MISMA SECUENCIA DE ENSAMBLADO INDICADA PARA LA PRIMERA SECCION DEL PONTON EN LAS RESTANTES.

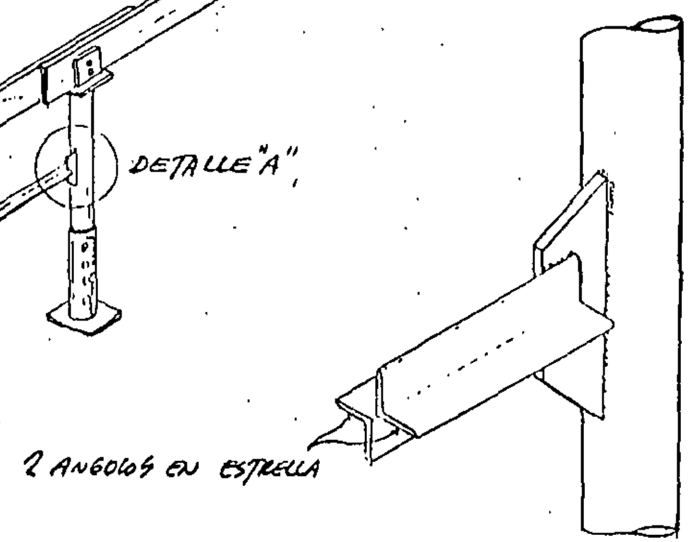
G.- UNA VEZ ENSAMBLADAS Y MONTADAS CON APOYOS PROVISIONALES CADA UNA DE LAS SECCIONES, INICIAR EL SOLDEO DE LAS MISMAS.

PARA MAYOR INFORMACION CONSULTAR EL MANUAL DE MONTAJE No. 1 SECCION 4.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I J L REVISADO POR Ing. J H B	FECHA JUN-86	PLANO 4-002
SECCION 4o MONTAJE DE TECHO FLOTANTE	MANUAL DE MONTAJE N° 1		

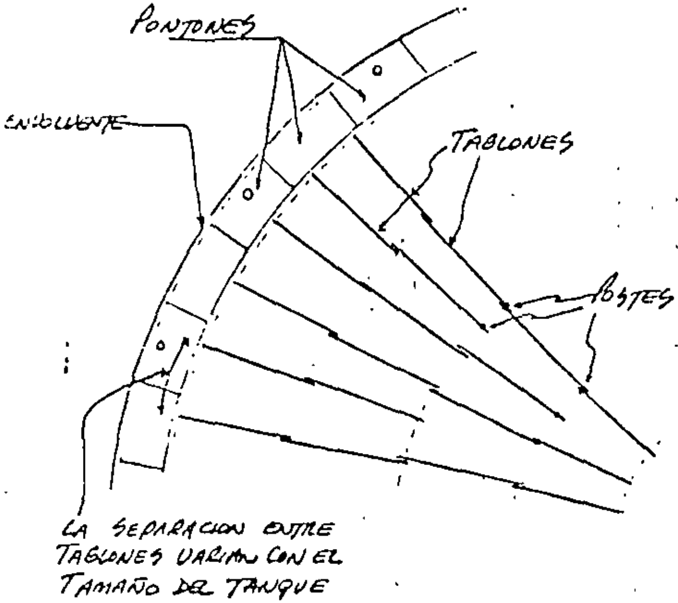
OBRA FALSA
PARA APOYO Y ARMADO DEL TECHO

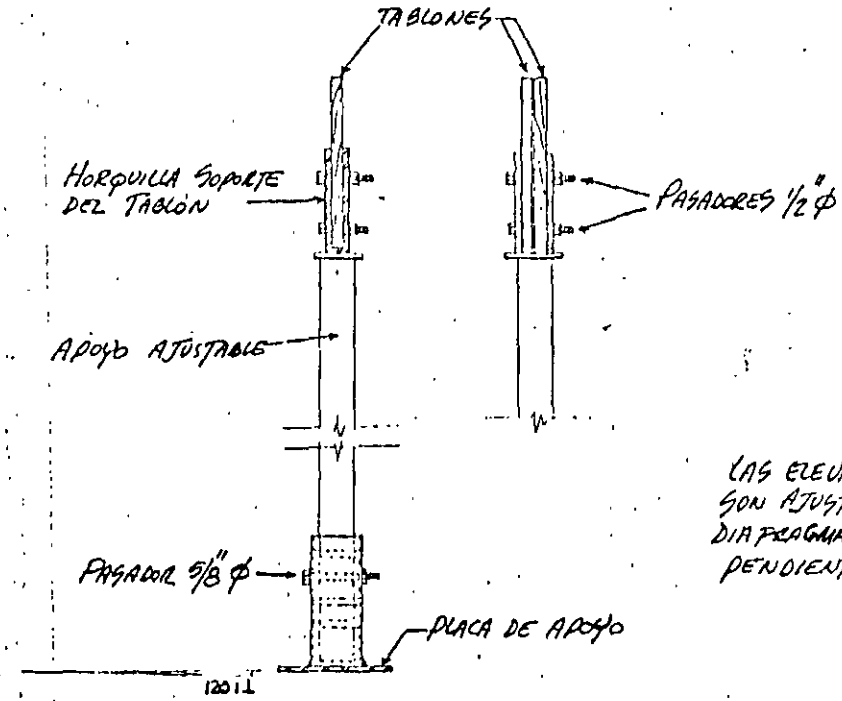




DETALLE "A"

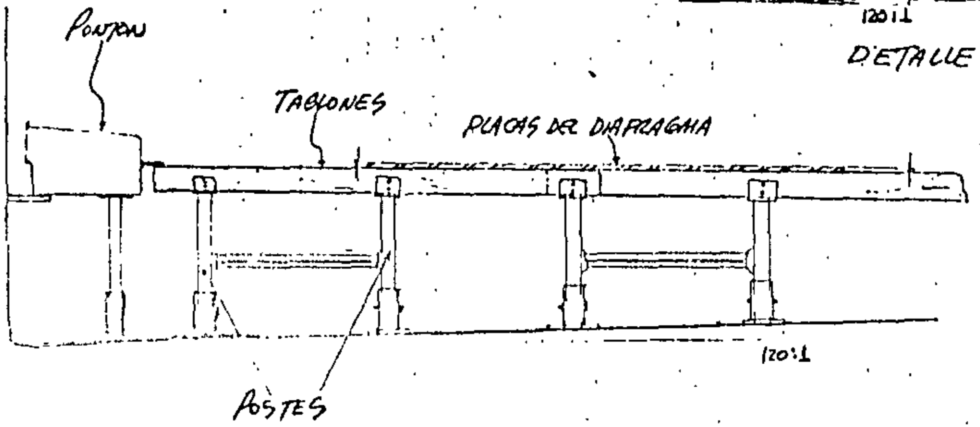
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA JUN-86	PLANO 4-003
SECCIONADO LEVANTATE DEL TECHO FLOTANTE	MANUAL DE MONTAJE No 1		





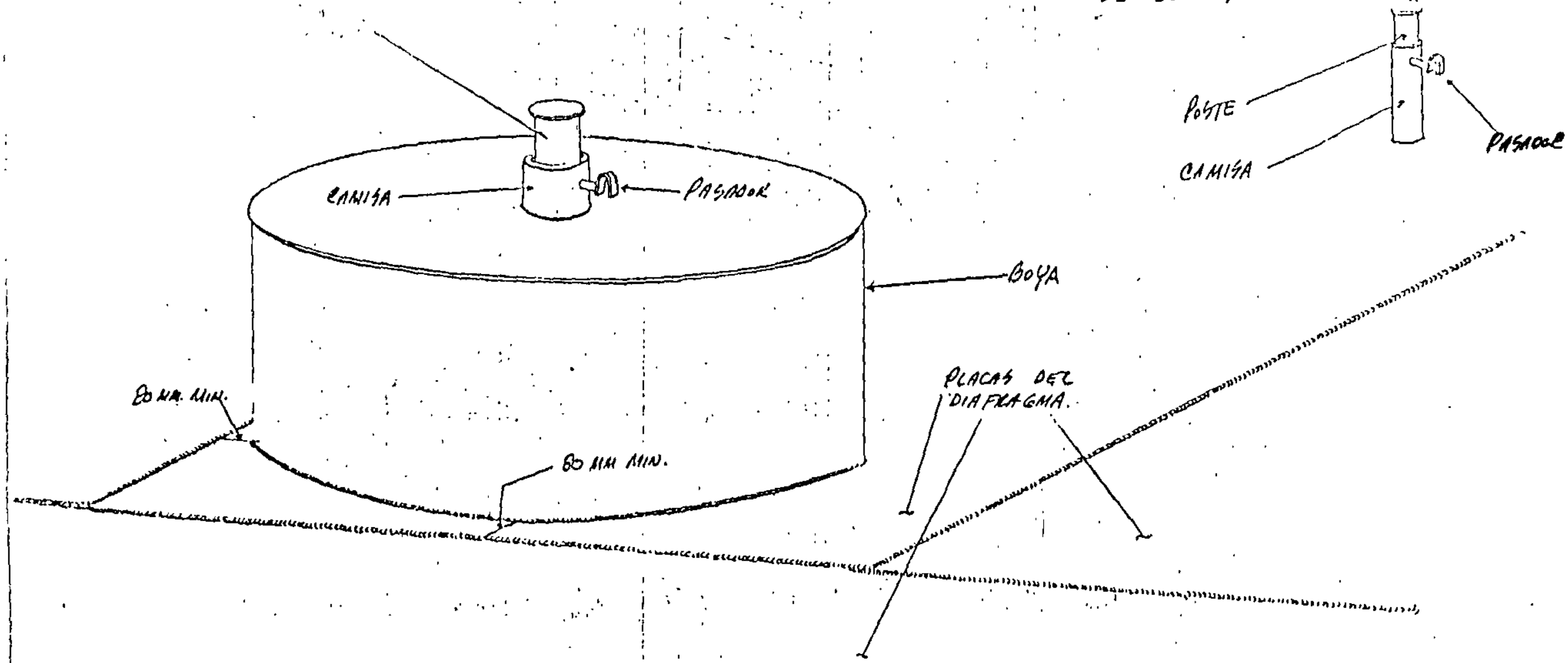
DETALLE DE UN POSTE

LAS ELEVACIONES DE LOS POSTES SON AJUSTADAS DE MODO QUE EL DIAFRAGMA SE TIENDA PLANO SIN PENDIENTES.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA Jun-86	PLANO 1-004
SECCION 4.0 MONTAJE DEL TECHO FLOTANTE	MANUAL DE MONTAJE N° 1		

AJUSTAR EN CAMPO LA LOCALIZACION DE CADA BOYA DE MODO QUE LA DISTANCIA DEL CORDON DE SOLDADURA A SU LINEA DE TANGENCIA NUNCA SEA MENOR DE 80 MM.

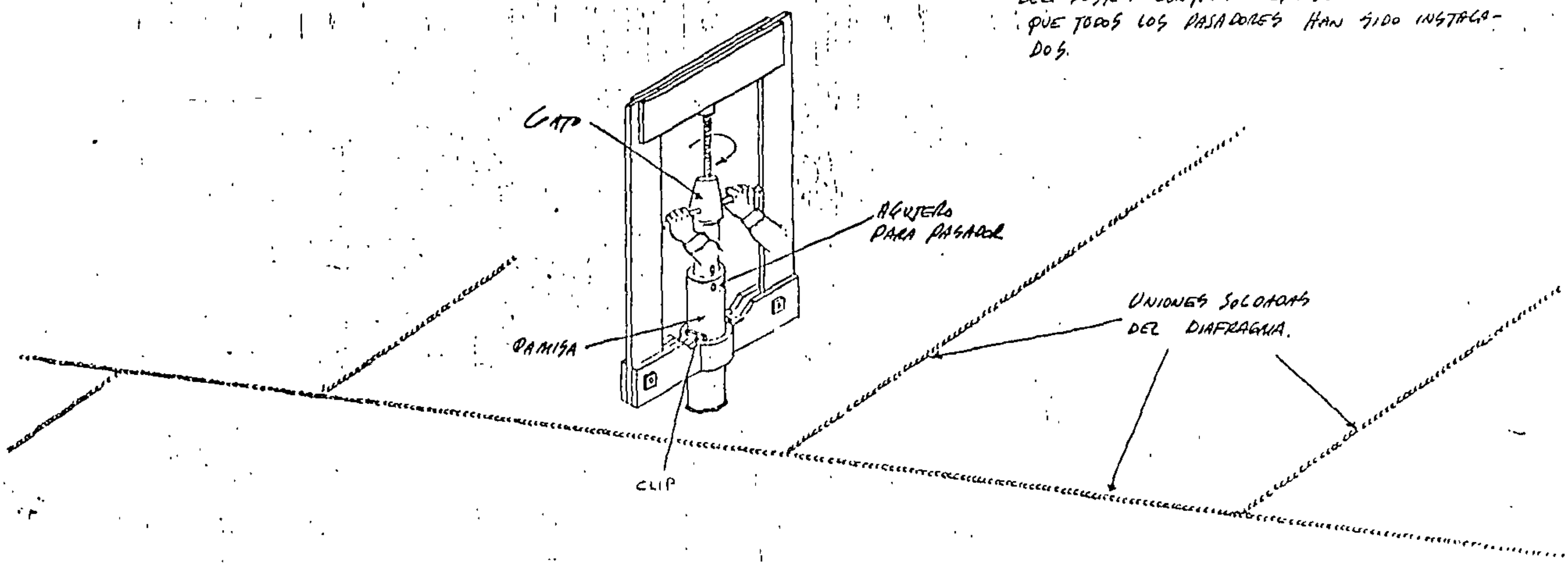


P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I J L. / APROBADO POR Ing. J H B	FECHA JUN-85	PLANO 4-005
SECCION 4.0 VENTEO DE TECHO FLOTANTE	MANUAL DE MONTAJE Nº 1		

C.2.

PARA ASEGURAR LOS POSTES A SU CAMISA CORRESPONDIENTE. ES NECESARIO ELEVAR EL DIAFRAGMA HASTA 15 MM. PARA INSERTAR EL PERNO DE SUJECCION.

USAR SIMULTANEAMENTE TRES BASTIDORES COLOCADOS EN TRES POSTES DE SOPORTE. CON GATOS PONER LOS POSTES EN POSICION E INSERTAR PASADORES ATRAVES DE LOS AGUJEROS DE LA CAMISA Y DEL POSTE. CONTINUAR ESTA SECUENCIA HASTA QUE TODOS LOS PASADORES HAN SIDO INSTALADOS.



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / AVALADO POR Ing. J. H. B.	FECHA JUN-86	PLANO 4-006
SECCION 4.0 MONTAJE DE TECHO FLOTANTE	MANUAL DE MONTAJE N° 1		

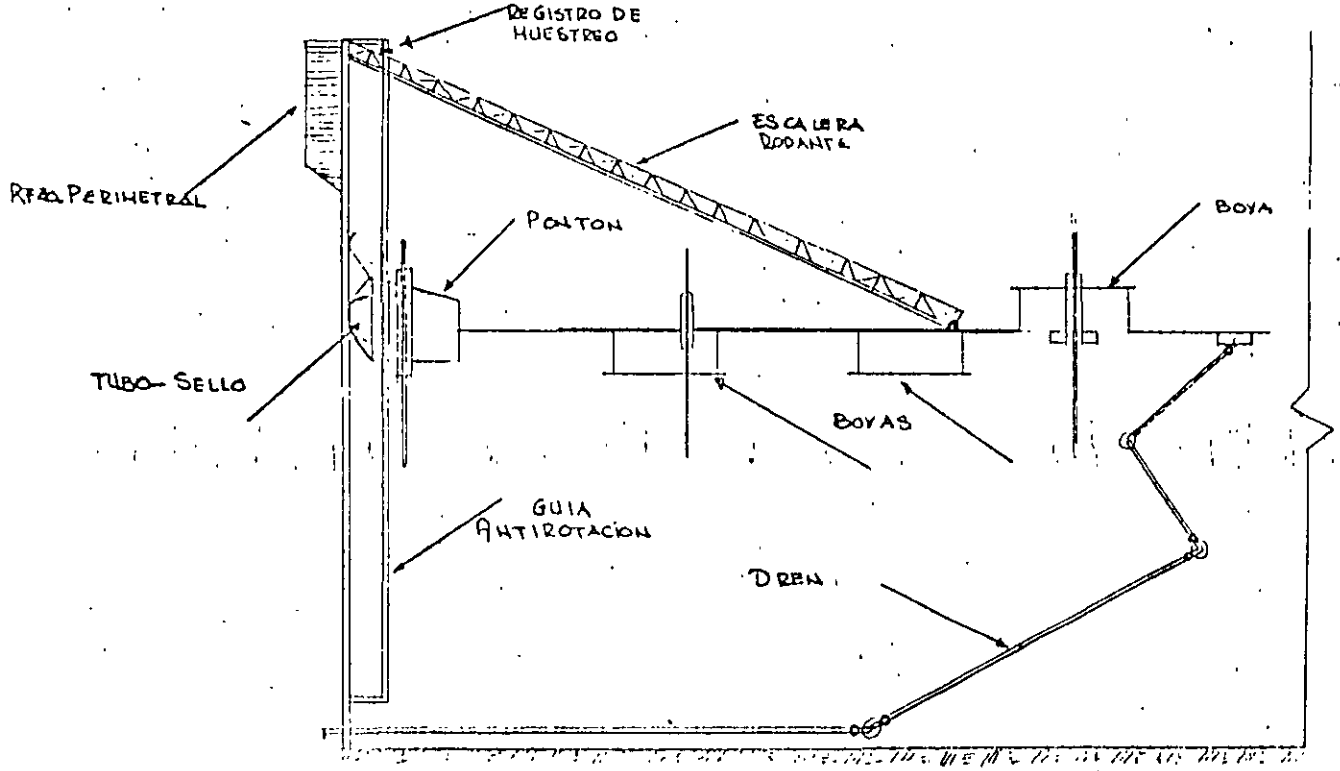
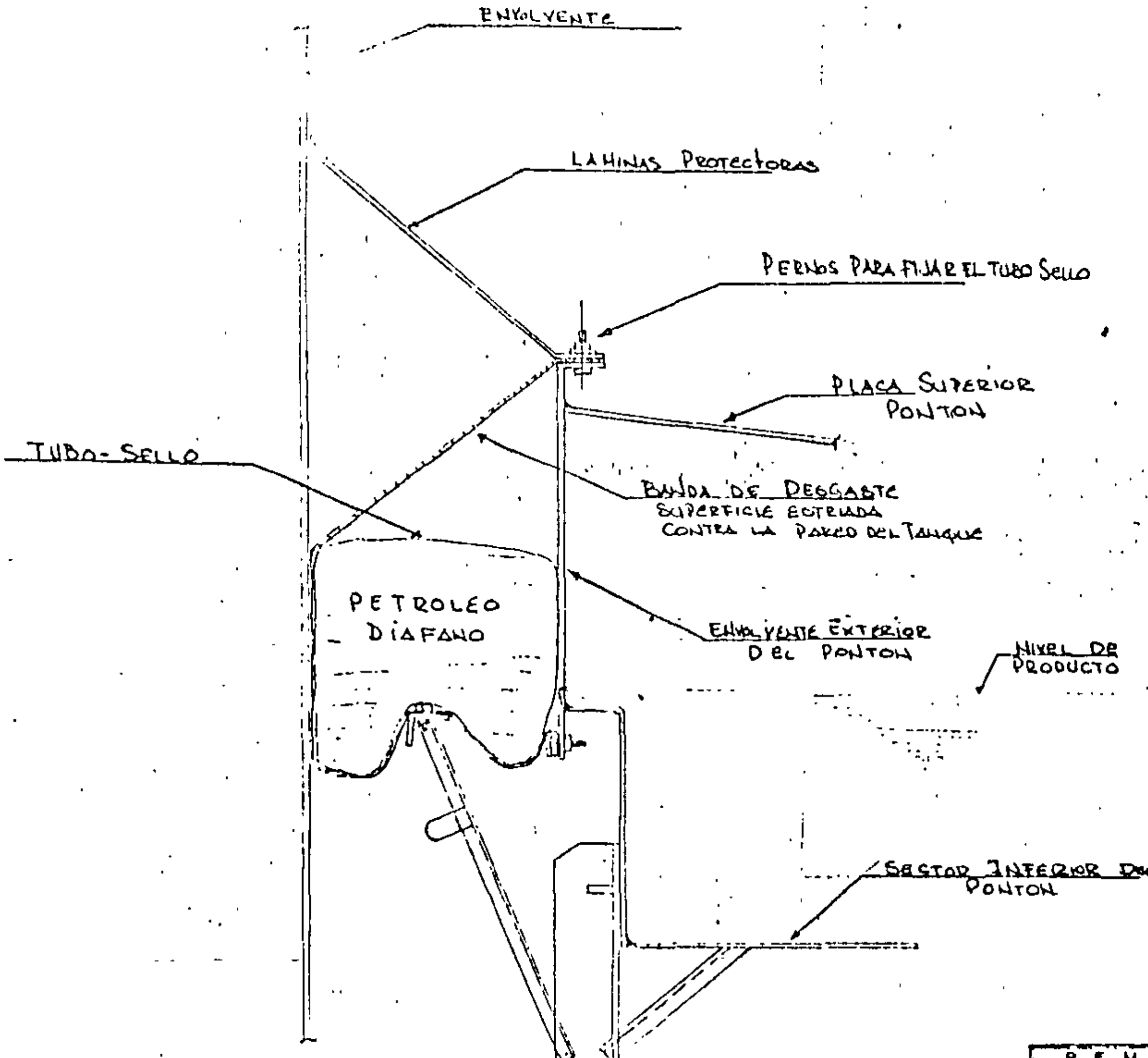


FIG. 5.1

Antes De Iniciar La Instalación Del Tubo-Sello, Deberá Estar Completamente Montado y Soldado El Fondo, La Envolvente y El techo Flotante Del Tanque. El Techo Apoyado En el Fondo Con sus soportes Definitivos y Concentricos Con la Envolvente Del Techo.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. / APROBADO POR: Ing. J. H. B.	FECHA VI-82	PLANO 5-001
SECCION 5.0 INSTALACION DEL TUBO SELLO	MANUAL DE MONTAJE Nº 1		



Revisar que la Separación Entre la Envolvente Exterior Del Ponton y la Pared del Tanque, Esté de Acuerdo con las Dimensiones Del Plano De Montaje el Cual Nos Marca 229 mm. De espacio Anular.

La Envolvente Del Pontón Deberá Estar Completamente Vertical sin Ninguna Curvatura o Comba En su Parte Superior.

FIG 5.2

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA VI-86	PLANO 5-02
SECCION 5.0 INSTALACION DEL TUBO-SELLO	MANUAL DE MONTAJE N° 1		

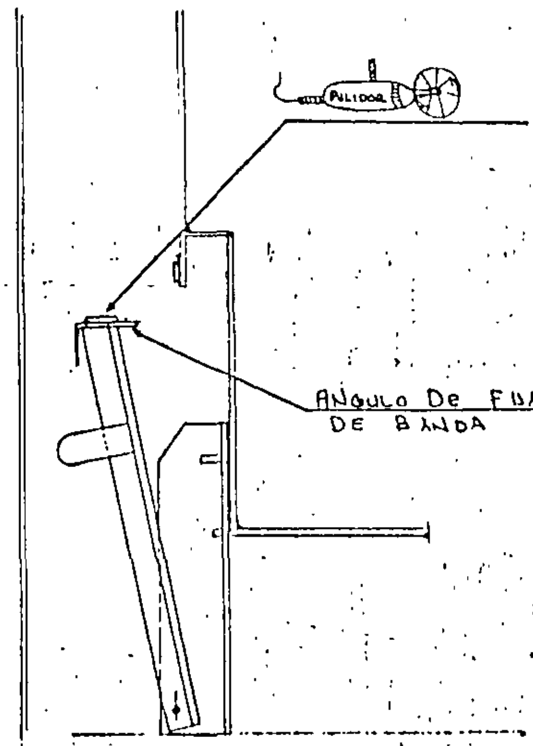


FIG. 5.4

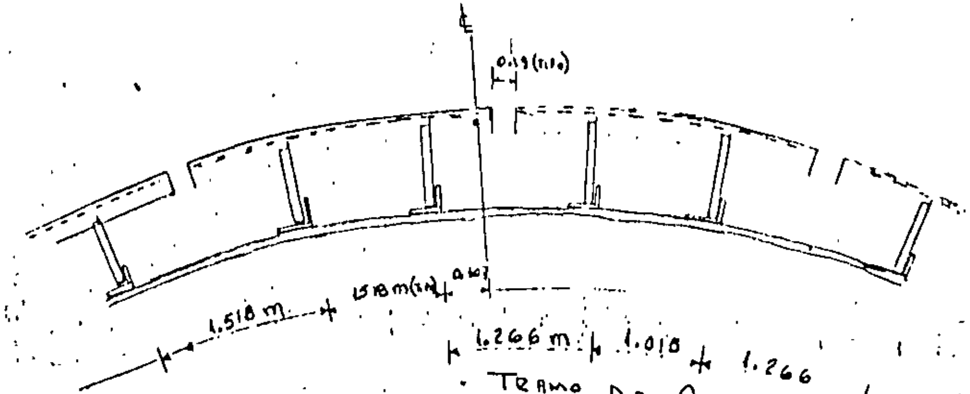
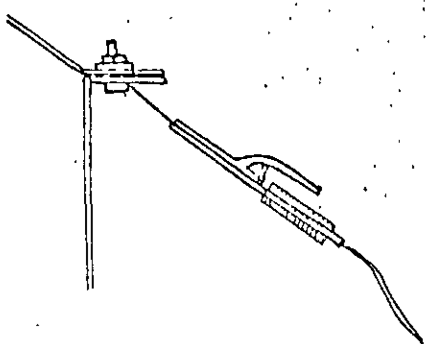
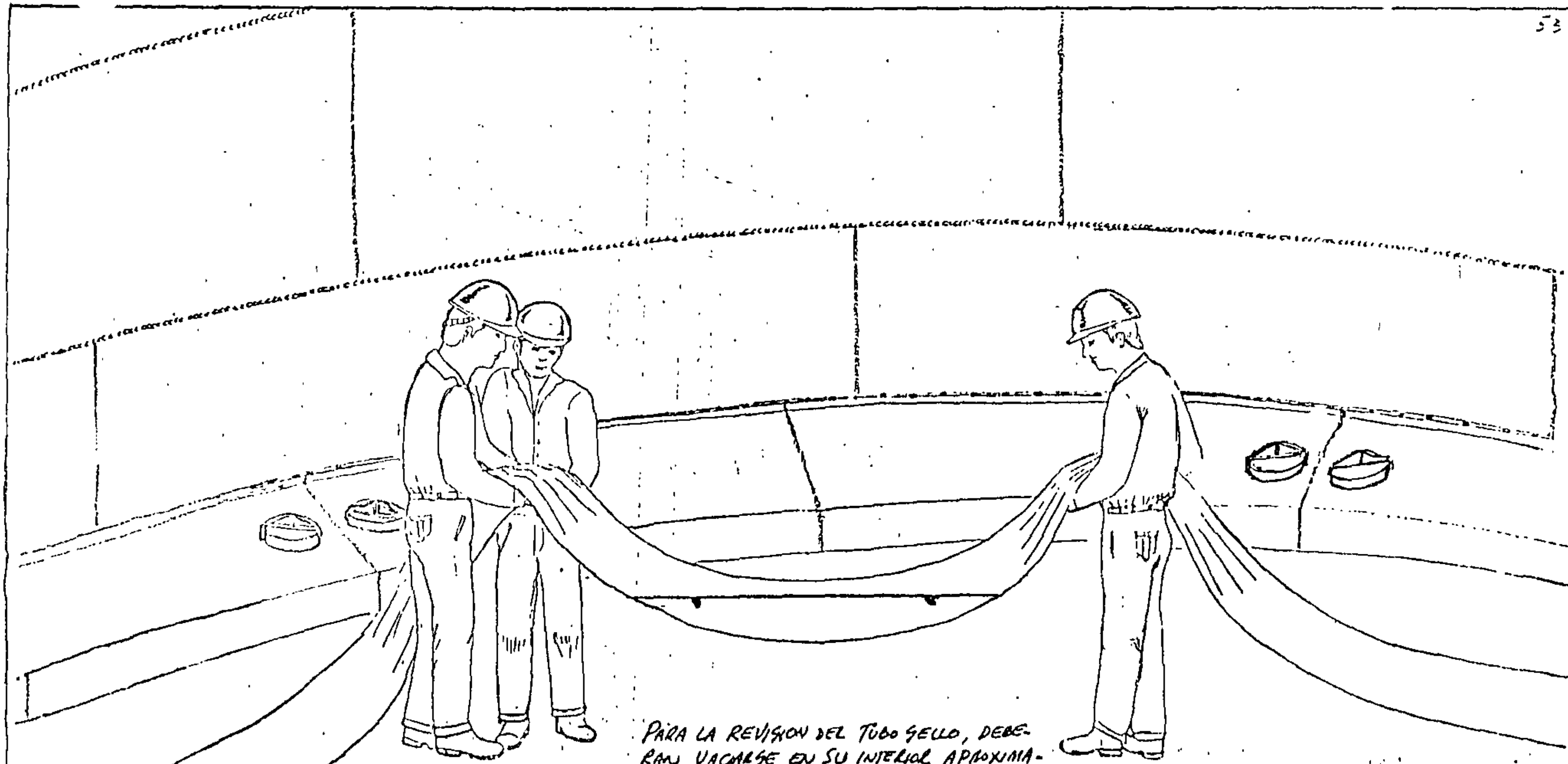


FIG. 6.5

LAS JUNTAS VERTICALES DEL ANGULO INFERIOR DE SUJECION, DEBERAN ESTAR ALINEADAS Y AL RAS, SOLDARLAS Y ESMERILARLAS A DEJARLAS ALISADAS. CUALQUIER SALIENTE EN LAS ALAS DEL ANGULO, TAMBIEN SERA REBAJADO.

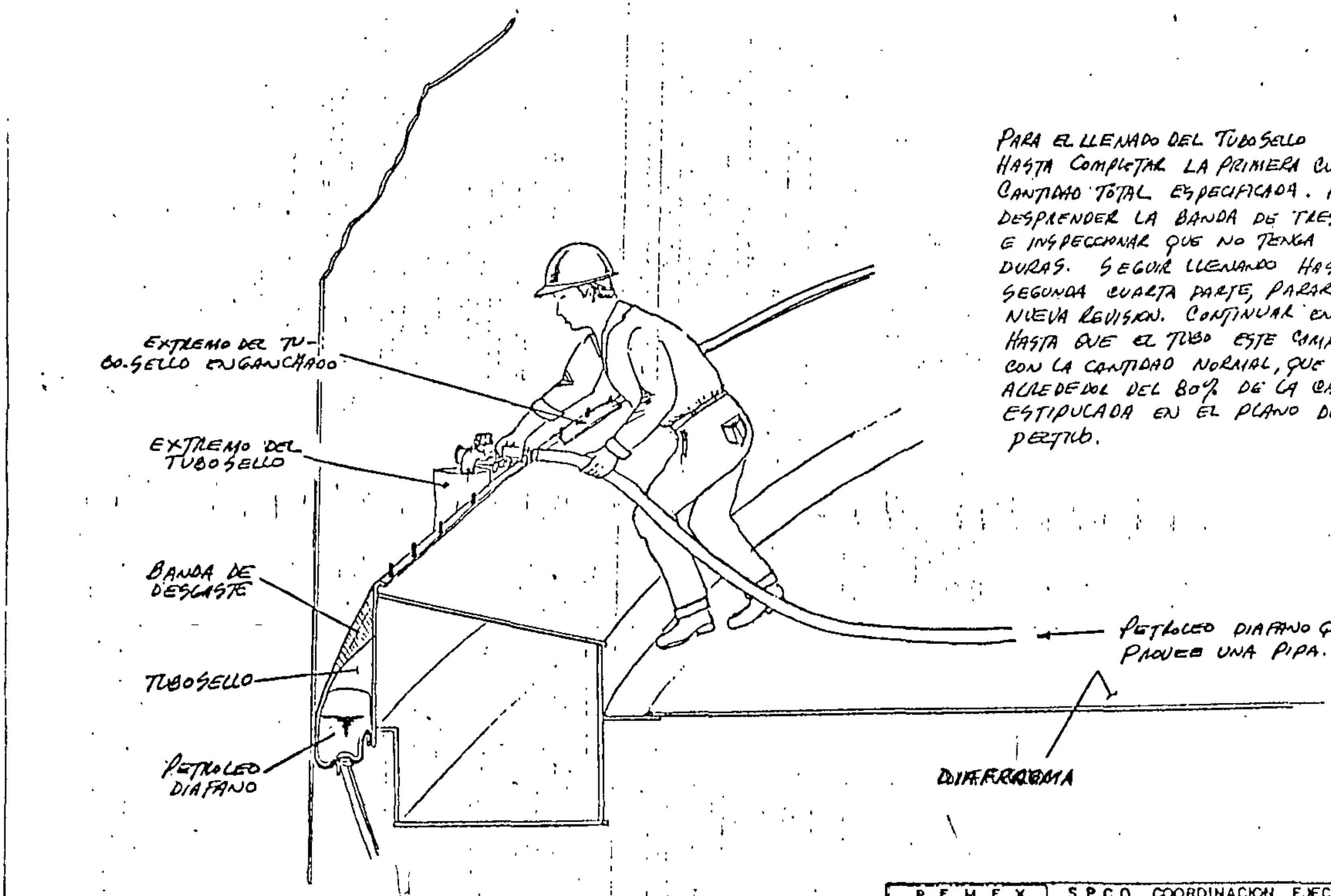


P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA VI-86	PLANO 5-002
SECCION 5.0 INSTALACION DEL TECHO FLOTANTE	MANUAL DE MONTAJE No 1		



PARA LA REVISION DEL TUBO SELLO, DEBERAN VACARSE EN SU INTERIOR APROXIMADAMENTE 40 LTS. DE PETROLEO DIAFANO, LEVANTANDOLO Y FORMANDO UN COLUMPIO PARA PODER INSPECCIONAL CORRECTAMENTE ESE TRAMO. REPETIR ESTA OPERACION RECORRIENDO LA LONGITUD TOTAL DEL TUBOSELLO.

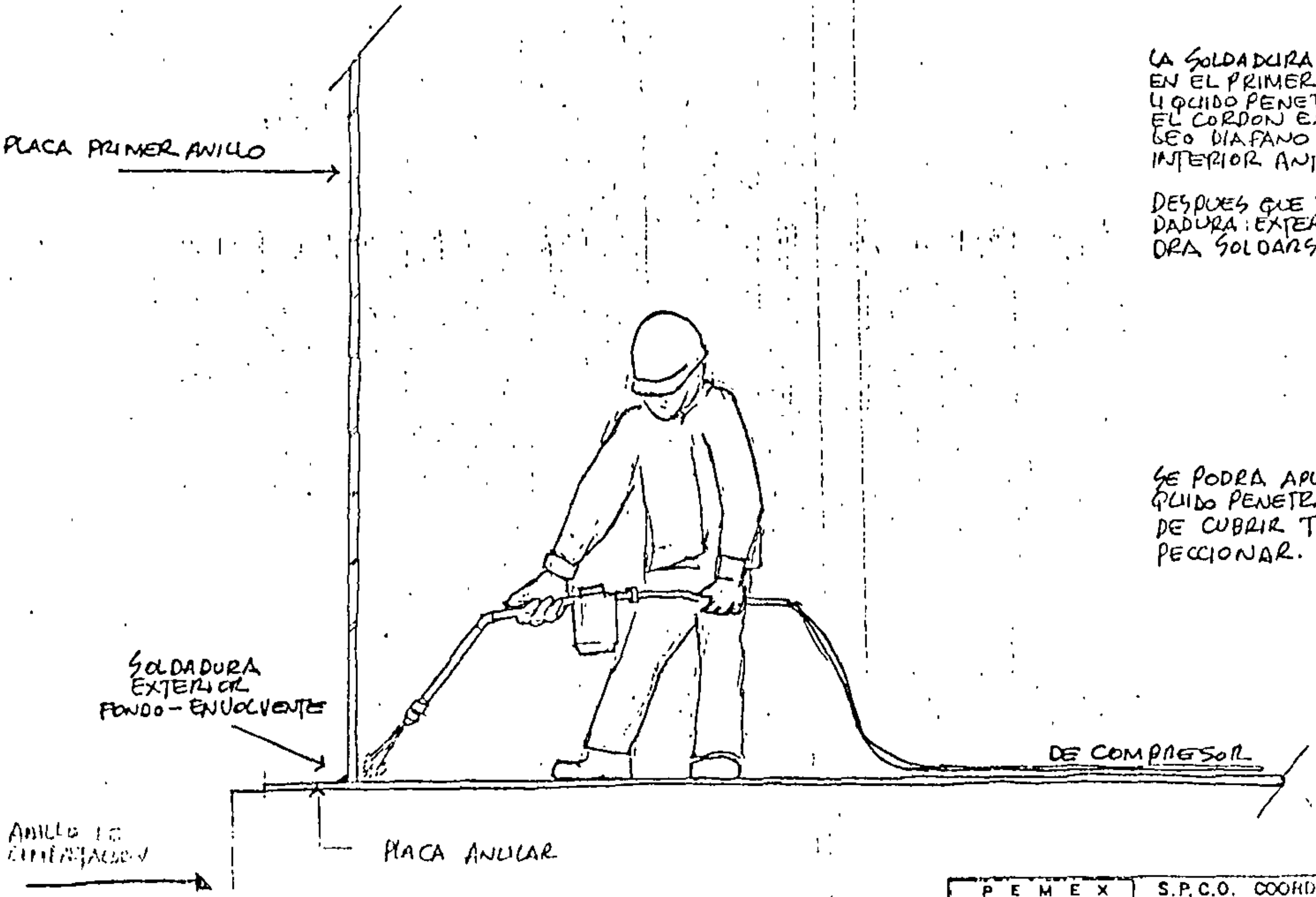
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. / APROBADO POR Ing. J. H. B.	FECHA IX-86	HOJA 5 de 009
SECCION 50 INSTALACION DEL TUBO SELLO	MANUAL DE MONTAJE Nº 1		



PARA EL LLENADO DEL TUBOSELLO SE VACIARA LIQUIDO HASTA COMPLETAR LA PRIMERA CUARTA PARTE DE LA CANTIDAD TOTAL ESPECIFICADA. PARAR EL LLENADO, DESPRENDER LA BANDA DE TRES EN TRES TORNILLOS E INSPECCIONAR QUE NO TENGA ARRUGAS NI TORCEDURAS. SEGUIR LLENANDO HASTA COMPLETAR LA SEGUNDA CUARTA PARTE, PARAR Y EFECTUAR UNA NUEVA REVISION. CONTINUAR EN LA MISMA FORMA HASTA QUE EL TUBO ESTE COMPLETAMENTE LLENO CON LA CANTIDAD NORMAL, QUE ES CASI SIEMPRE ALREDEDOR DEL 80% DE LA CANTIDAD TOTAL ESTIPULADA EN EL PLANO DE MONTAJE RESPECTIVO.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. / APROBADO POR: Ing. J. H. B.	FECHA IX-85	HOJA 5 DE 05
SECCION 5.0 INSTALACION DEL TUBO SELLO	MANUAL DE MONTAJE Nº 1		

INSPECCION DE SOLDADURA CON LIQUIDO PENETRANTE

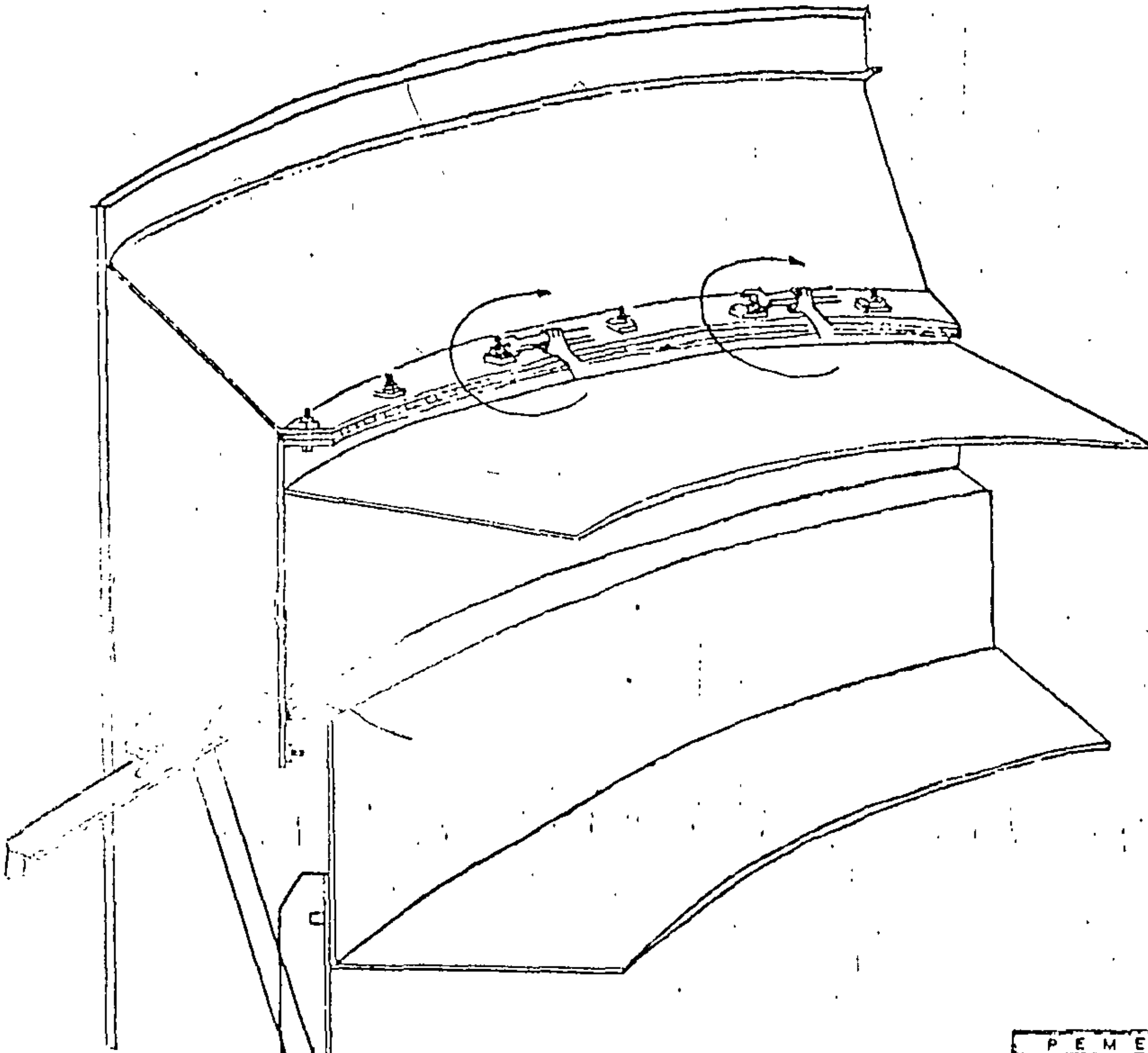


LA SOLDADURA EN EL FONDO-ENVOLVENTE EN EL PRIMER ANILLO SERA PROBADA CON LIQUIDO PENETRANTE DESPUES DE SOLDAR EL CORDON EXTERIOR. ROCIAR PETROLEO DIAFANO (KEROSENE) POR LA JUNTA INTERIOR ANTES DE SOLDARLA.

DESPUES QUE TODAS LAS FUGAS DE LA SOLDADURA EXTERNA HAN SIDO REPARADAS, PODRA SOLDARSE EL CORDON INTERIOR.

SE PODRA APLICAR MANUALMENTE EL LIQUIDO PENETRANTE TENIENDO PRECAUCION DE CUBRIR TOTALMENTE EL AREA A INSPECCIONAR.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I.J.L. / APROBADO POR Ing. J.H.B.	FECHA IX-86	5-006
SECCION 5.0 INSTALACION DEL TUBO-SELLO	MANUAL DE MONTAJE No 1		

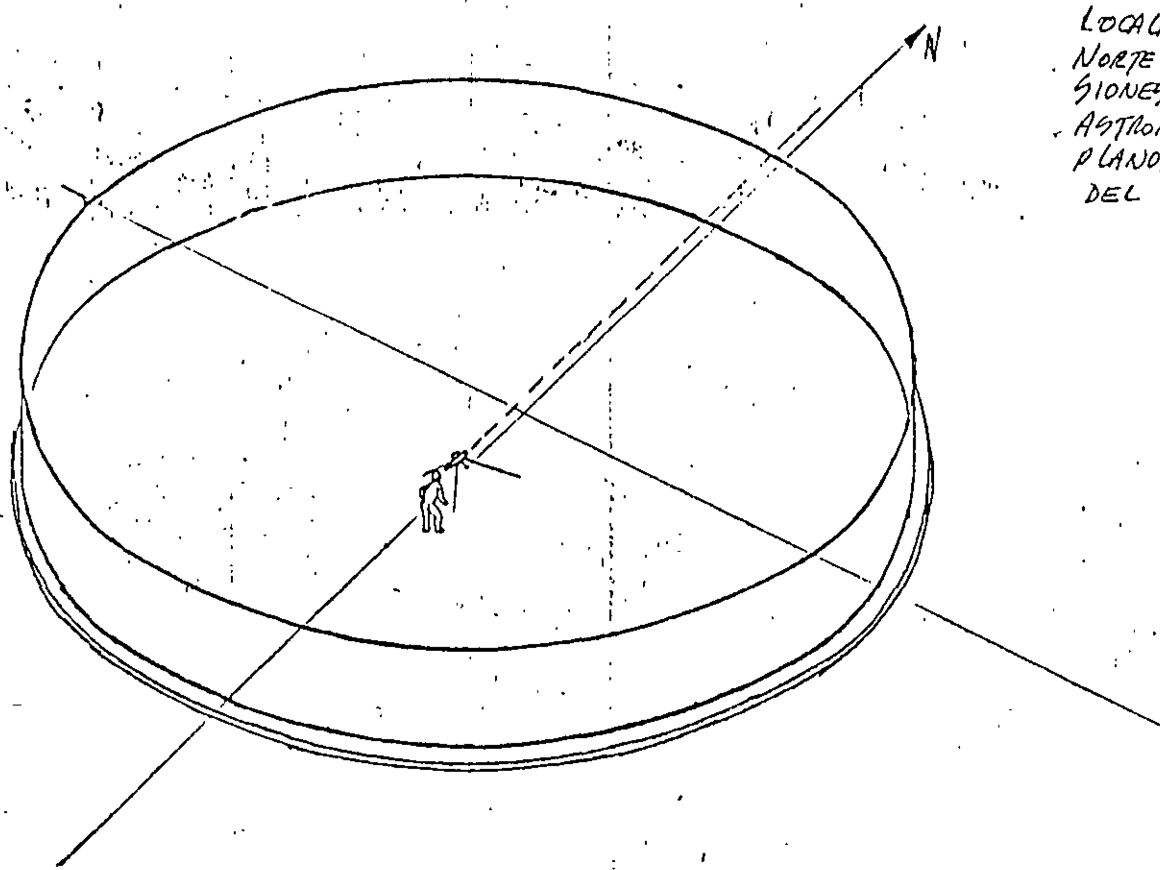


Cada Dos Tornillos Uno Si y El Otro No, Localizados En El Sector Superior Del Ponton, Estan En Linea Directamente Con Cada Agujero Del Angulo Inferior De Sujecion. Es Suficiente Verificar Mas o Menos Cada Diez Pernos Con Sus Correspondientes Agujeros Del Angulo, Que Esten En Linea y Comprobar Igualmente Que Sus Separaciones Sean Las Mismas.

Salpicaduras De Soldaduras Rebabas y Cualquier Otro Saliente Cortante Que Haya En El Espacio Donde Se Alojara El Sello, Deberan Ser Removidos.

FIG. 5.3

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. J. J. L. / APROBADO POR Ing. J. H. B.	FECHA VI-82	5-007
SECCION 5.0 INSTALACION DEL TUBO-SELLO	MANUAL DE MONTAJE Nº 1		

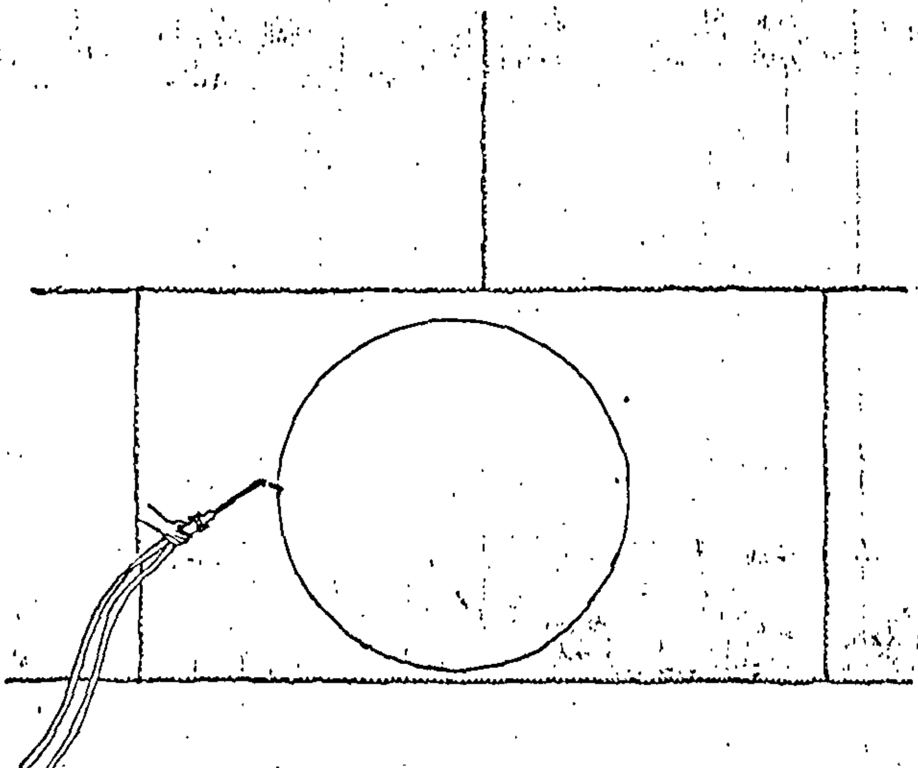


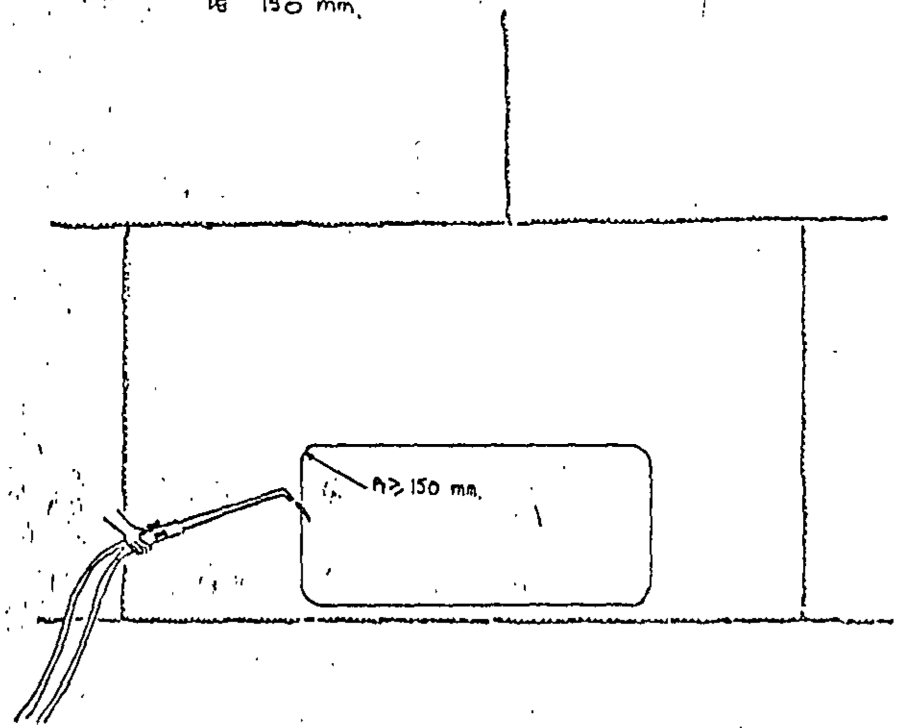
LOCALIZAR ACCESORIOS A PARTIR DEL NORTE CONSTRUCTIVO EL CUAL EN OCASIONES NO COINCIDE CON EL NORTE ASTRONOMICO DE ACUERDO CON LOS PLANOS DE LOCALIZACION DE ACCESORIOS DEL PROYECTO.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. F. J. L. APROBADO POR: Ing. J. H. B.	FECHA VIII-84	HOJA 1 de 5
SECCION 6.0 INSTALACION DE ACCESORIOS	MANUAL DE MONTAJE N° 1		

LOS CORTES EN LA ENVOLVENTE PARA ENTRADA DE BOQUILLAS DEBEN HACERSE CON EXACTITUD

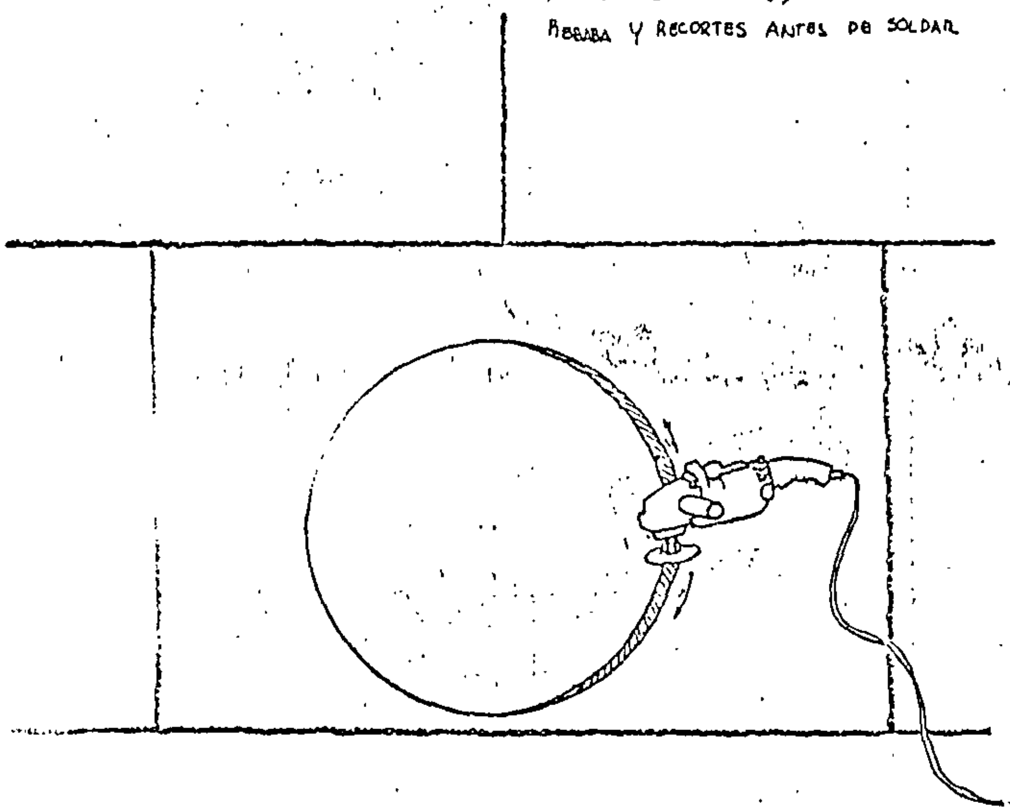
SIEMPRE QUE SEA POSIBLE NO DEBEN DISEÑARSE ENTRADAS RECTANGULARES, CUANDO SEA NECESARIO, REDONDEAR ESQUINAS CON UN RADIO NO MENOR DE 150 mm.





P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR [illegible] APROBADO POR [illegible]	FECHA VIII-86	HOJA 2 DE 5
SECCION 6.0 INSTALACION DE ACCESORIOS	MANUAL DE MONTAJE Nº 1		

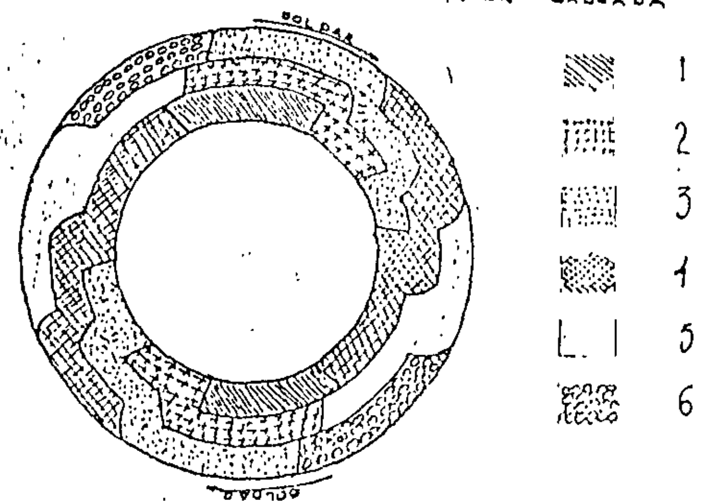
ELIMINAR BORDES O CANTOS ASPEROS Y ESQUINAS CON FILOS, REMOVER ESCORIA REBABA Y RECORTES ANTES DE SOLDAR



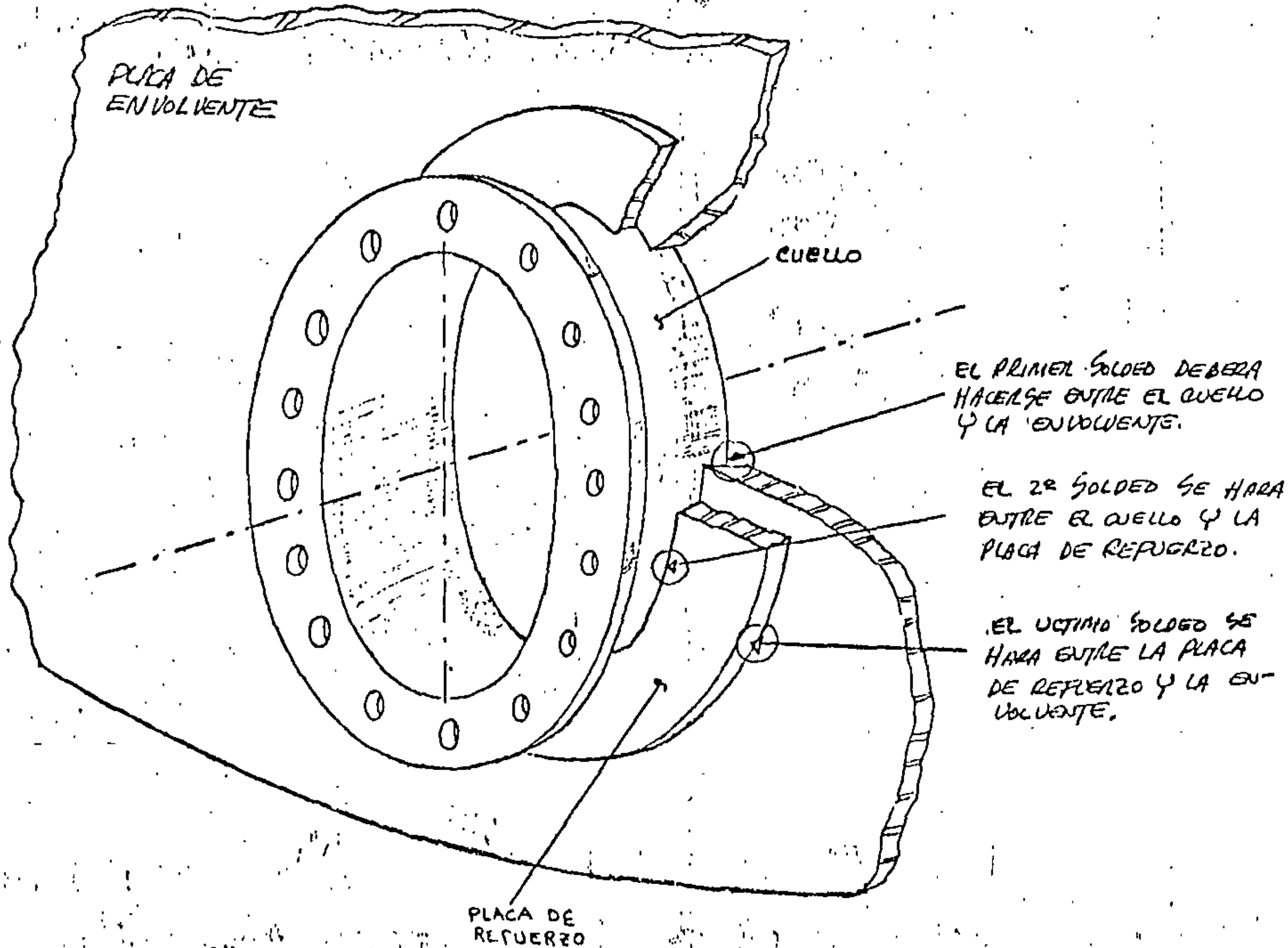
EFECTUAR SOLDADURA POR EL METODO DE CASCADA CON EL FIN DE MANTENER SIEMPRE CALIENTE LA BOQUILLA HASTA TERMINAR EL SOLDEO, SIGUIENDO EL PROCEDIMIENTO QUE SE INDICA.

NO MARTILLAR EN LA PRIMERA Y LA ULTIMA CAPA, PERO SI ES PERMITIDO EN LAS CAPAS INTERMEDIAS CON EL FIN DE EVITAR DEFORMACIONES.

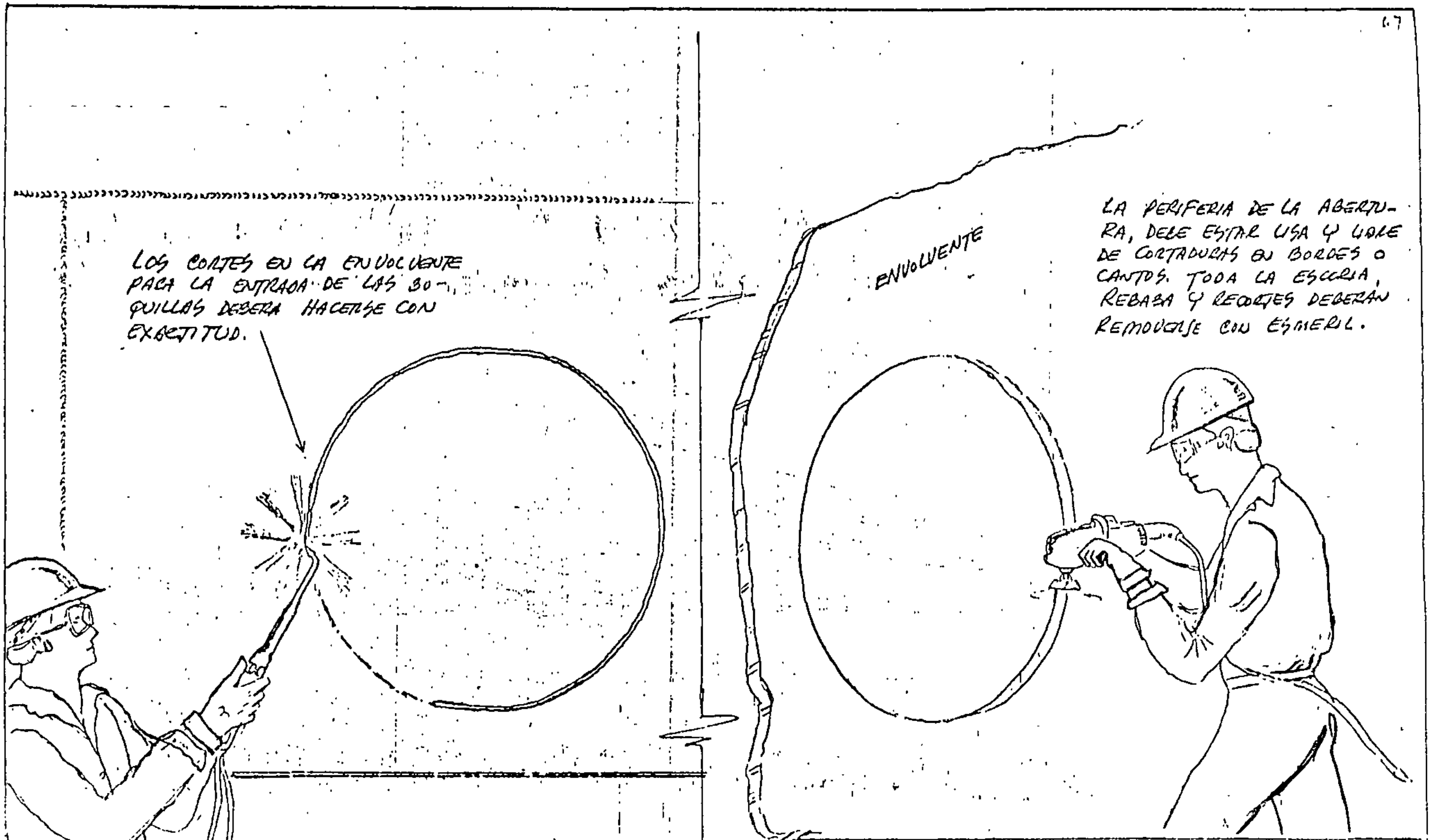
SECUENCIA DE SOLDADURA EN "CASCADA"



P E M E X — S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION			
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA VIII-86	HOJA 3 de 5
SECCION 6.0 INSTRUMENTACION Y ACCESORIOS	MANUAL DE MONTAJE No 1		

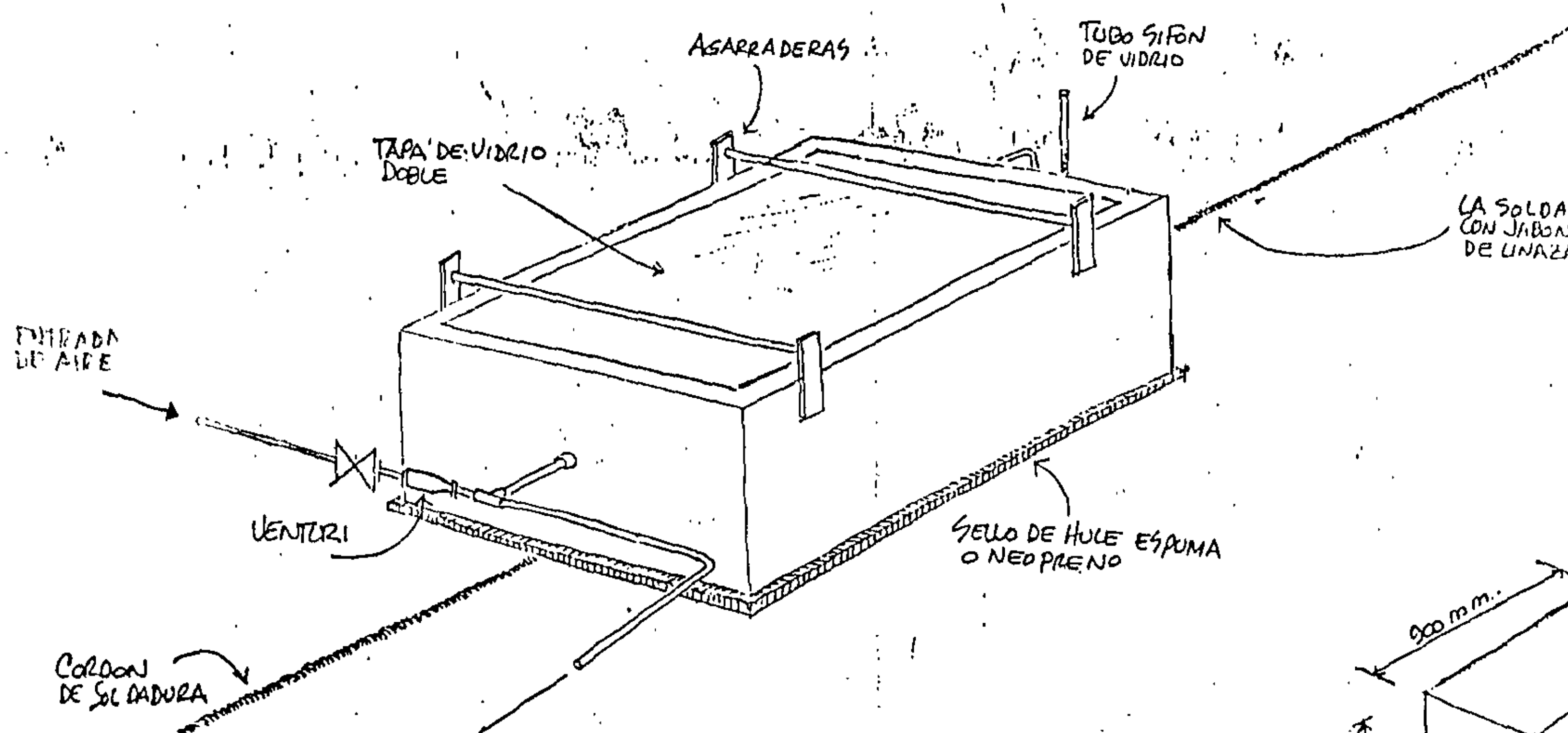


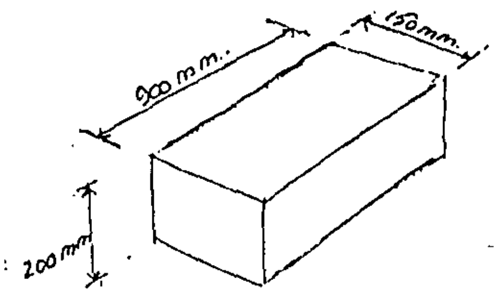
P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I. J. L. APROBADO POR: Ing. J. H. B.	FECHA 6/11-86	HOJA 4 de 5
SECCION 6.0 ACCESORIOS	MANUAL DE MONTAJE Nº 1		



P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing I. J. L. / AUTORIZADO POR Ing J. H. B.	FECHA VIII-86	HOJA 5 de 5
SECCION 6.0 ACCESORIOS	MANUAL DE MONTAJE N° 1		

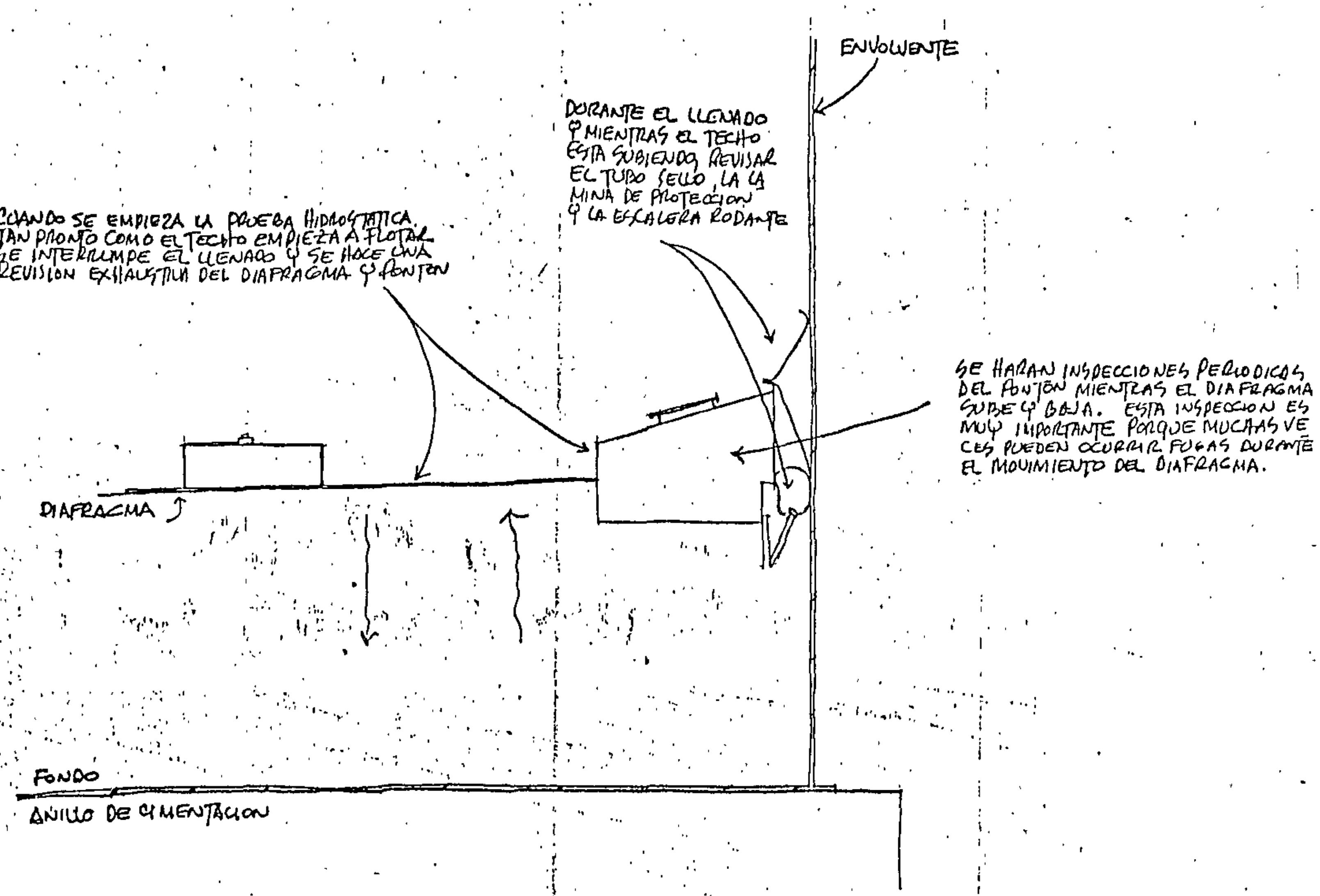
CAJA METALICA PARA PRUEBAS DE VACIO :





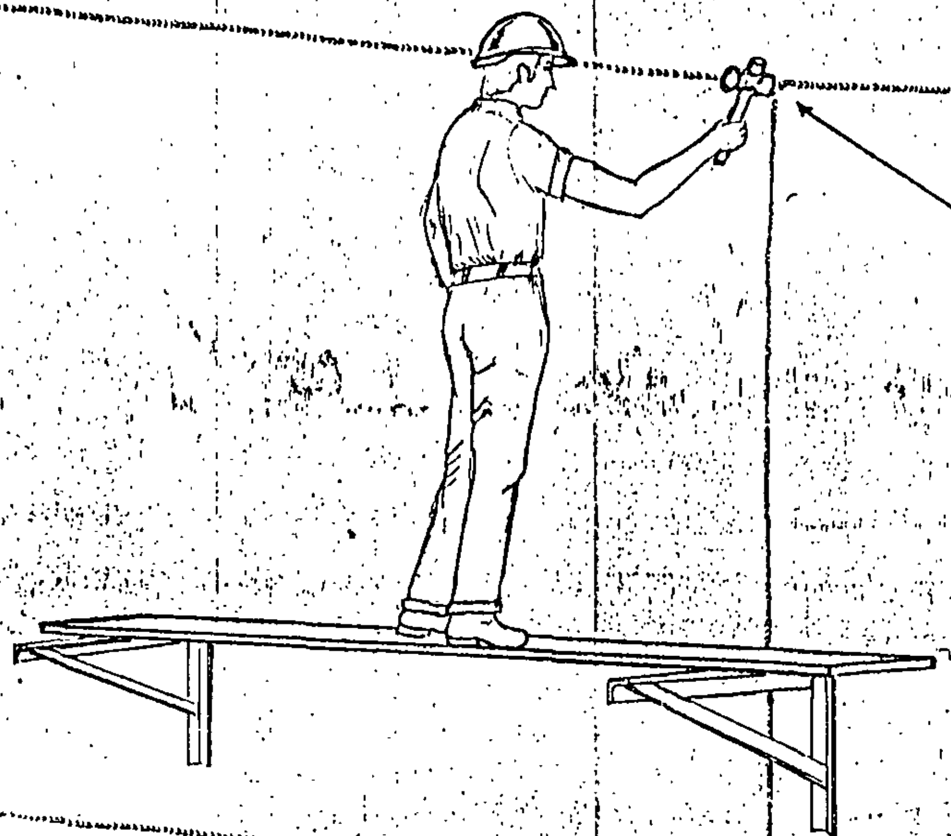
UN PROCEDIMIENTO EFICAZ PARA INSPECCIONAR LOS CORDONES DE SOLDADURA DE FONDOS Y APROBADOS POR EL API, ES MEDIANTE LA CAJA METALICA PARA PRUEBAS DE VACIO.

P E M E X	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. M. B.	FECHA IX-86	PLANO 1 DE 3
SECCION 7.0 PRUEBAS, INSPECCION	MANUAL DE MONTAJE Nº 1		



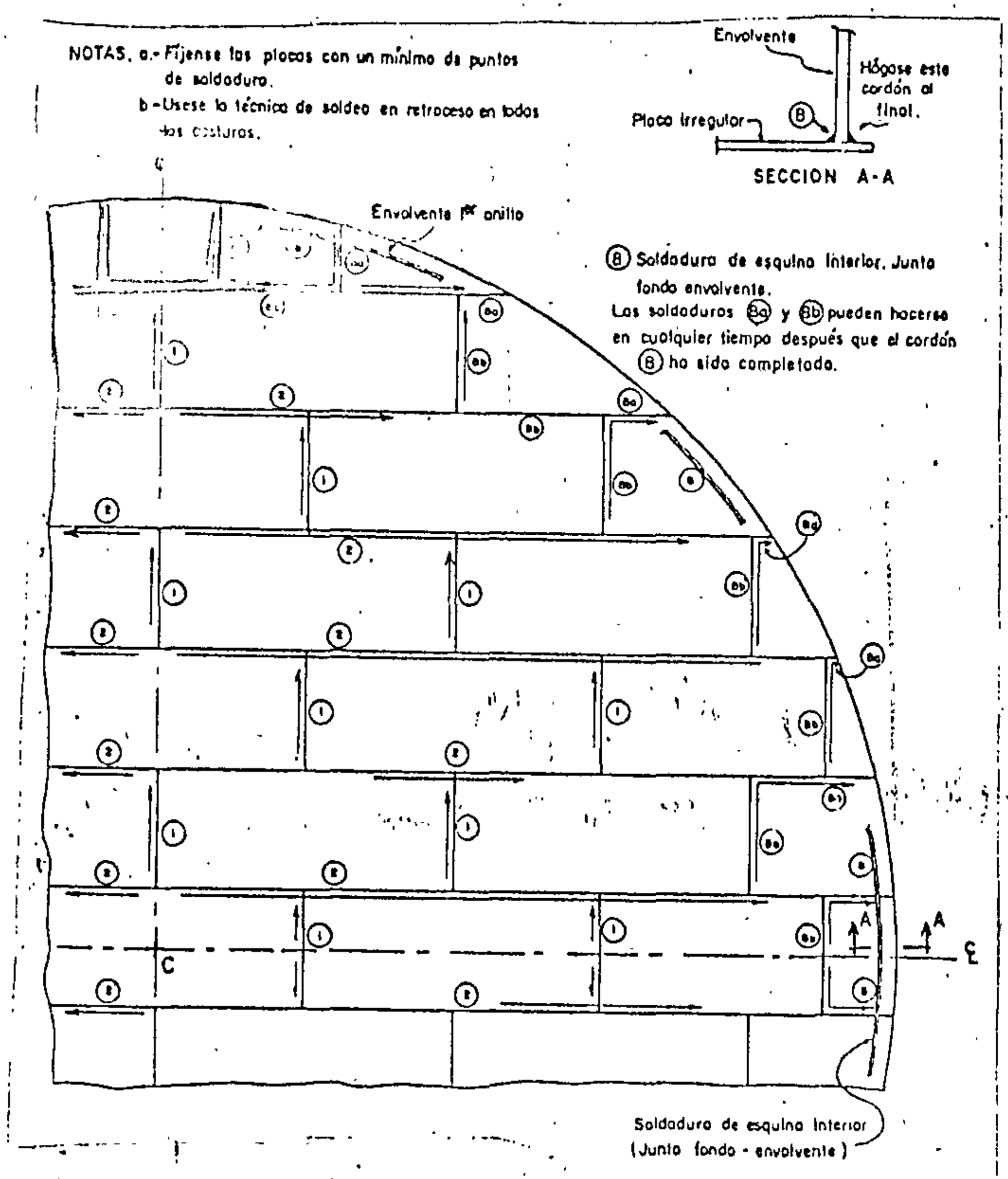
PEMEX	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR Ing. I. J. L. APROBADO POR Ing. J. H. B.	FECHA IX-86	PLANO 2 de 3
SECCION 7.0 PRUEBAS, INSP. FINAL	MANUAL DE MONTAJE Nº 1		

CUANDO SE TIENE EL TANQUE LLENO DE AGUA SE DEBE HACER UNA REVISION OCULAR DE LAS SOLDADURAS POR SI SE DESCUBRA ALGUNA FUGA.



ESTA PERMITIDO GOLPEAR CON UN MARTILLO DE BOLA LAS SOLDADURAS ESPECIALMENTE LOS CRUCES AL HACER LA REVISION.

PEMEX	S.P.C.O. COORDINACION EJECUTIVA DE CONSTRUCCION		
TANQUES CILINDRICOS VERTICALES TECHO FLOTANTE	HECHO POR: Ing. I.J.L. APROBADO POR: Ing. J.H.B.	FECHA IX-86	HOJA 3 DE 3
SECCION 7.0 PRUEBAS INSP. FINAL	MANUAL DE MONTAJE Nº 1		



ARREGLO DE TENDIDO N° 3

EL ARREGLO MOSTRADO EN ESTA FIGURA ES TIPO PLATAFORMA, CON TODAS LAS PLACAS RECTANGULARES E IRREGULARES EN UN SOLO SENTIDO Y SIN PLACAS ANULARES. ESTE ARREGLO SE USA EN TANQUES DE MEDIANA CAPACIDAD (55,000 A 500 BLS).

LA TECNICA DE SOLDEO SE DESCRIBA EN EL MANUAL DE MONTAJE N° 1, SECCION 2.1.3.1 QUE ES LA UTILIZADA POR C.B.I.

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FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MÓDULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA

EQUIPO DE APOYO PARA EL MONTAJE DE ESTRUCTURAS METÁLICAS

ESPOSITOR: ING. VICTOR J. SÁEZ DE OCARIZ ALBISÚA
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OCTUBRE DE 1998

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PART TWO

Notes

(i) Where safe working load (SWL) tables are given, these are intended only as guidelines for use in equipment selection exercises as the permitted lifting capabilities of machines vary depending upon the regulations operating in different countries. Typically in this book SWL is quoted as 75% of the tipping load, but up to 85% is allowable under some National Standards.

(ii) Cranes designed for grabbing or magnet use, the calculated load is increased by 25%, i.e. safe working load is 80% of that for cranes.

(iii) The use of terms such as winch and hoist, jib and boom, derricking and luffing are freely interchanged to have the same meaning.

Chapter 12 SIMPLE LIFTING MECHANISMS

INTRODUCTION

The choice of lifting equipment is vast, ranging from simple ropes and pulleys, to large cranes and the trend is towards ever more sophisticated lifting devices. For example over 3000 tonnes capacity crawler cranes are currently in the design stage. However, even the large crane relies upon the principles of hoisting tackle, i.e. lifting rope, pulley blocks and a winch. This basic arrangement has been used successfully to design temporary works-lifting mechanisms for many tasks, including the erection of bridges, high-rise structures and industrial buildings. It is only during the past century that the modern crane has extended the application of such devices.

PRINCIPLES OF HOISTING TACKLE (ROPES AND PULLEYS)

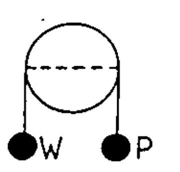


Figure 12.1 Single line pulley block

Figure 12.1 shows a single line fixed pulley block, clearly P and W are only equal if there is no friction. When the block is free to rotate on an axis, the friction is a small component of the total forces and P and W can be assumed equal. Such a system, called 'single part' or 'fall', is commonly used for reeving on cranes.

The load-carrying capacity of the hoist tackle may be increased by using two, or more pulleys as in fig. 12.2. In the arrangement shown, if n is the number of ropes leading from the lower to the upper blocks, then neglecting friction, each rope supports W/n. Thus $P = W/n$. It is obvious, however, that considerably more rope must be wound in to raise W, compared with the

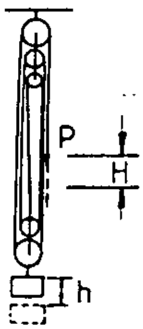


Figure 12.2 Blocks and tackle

single line system. The velocity ratio of the system, defined as the ratio of the distance moved by P to the distance moved by W is calculated as follows:

$$\begin{aligned} \text{Work done by } P &= \text{work done by } W \\ PH &= Wh \end{aligned}$$

$$\text{therefore} \quad \frac{H}{h} = \frac{W}{P} = \frac{W}{W/n} = n$$

In practice the pulley blocks at the top and bottom are mounted on a single axis.

The reeving loads are governed by BS 1757:1964, which covers all the possible reeving arrangements from single up to eight and above falls.

Figure 12.3 shows the methods of reeving for two, four, six and eight falls. The rope follows the numbers on the pulleys beginning with the lead line over the top block.

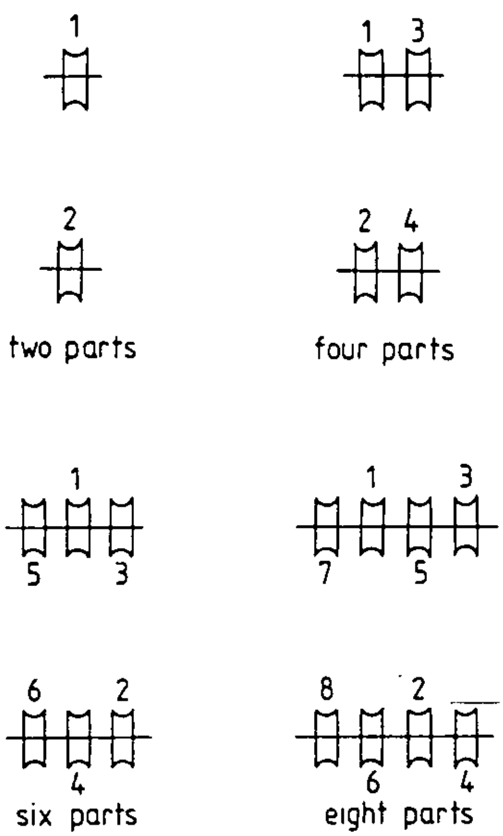


Figure 12.3 Reeving methods

ROPES

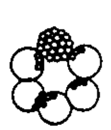


Figure 12.4 Wire-rope configuration

Cranes use wire ropes which comprise strands made up of individual wires, of which there are several configurations as typified by the example shown in fig. 12.4. The load carrying capacity of the rope depends upon its diameter and the number of wires and is manufactured with a safety factor of approximately five.

The three common lays are ordinary lay, Lang's lay and non-rotating lay.

Figure 12.5 Ordinary right-hand lay

Figure 12.6 Ordinary left-hand lay

Figure 12.7 Right-hand Lang's lay

Figure 12.8 Left-hand Lang's lay

ORDINARY LAY

In the right-hand lay method the wire spirals to the left and the strands to the right (fig. 12.5). In the left-hand lay the arrangement is *vice versa* (fig. 12.6). These types of lay are useful as slings, but tend to wear quickly if operated as hoist ropes, since only the crown wires are in contact with the pulley.

LANG'S LAY

Lang's lay has both the wires and strands spiralling in the same direction (figs 12.7 and 12.8), but obviously both ends must be secured to prevent twisting. This lay has better wearing properties.

NON-ROTATING LAY

For most hoisting work with single part reeving the rope must avoid twisting. This is achieved by using a double rope construction, e.g. the inner rope can be in right-handed Lang's lay and the outer strands in left-handed ordinary lay. This method produces a non-rotating rope.

The size of the rope may be designed to suit the load requirements but for a medium size crane, e.g. 30 tonnes lifting capacity, an 18–20 mm diameter rope with up to six part reeving is usual.

WINCHES

All cranes and most lifting tasks with a block and tackle require a winch and rope drum to provide the lifting force. The winch may be powered by compressed air, electric motor, hydraulic motor or a diesel engine.

For portability around the construction site, the electric or compressed air winch is favoured. A typical arrangement of an electric winch is shown in fig. 12.9. The hoist speed of most winches may be varied to accommodate the line load. For example, with four-part reeving the described electric winch of 100 hp (74.6 kW) can handle 4000 kg at speeds up to 100 m/min, whereas at the full 20 000 kg load, the maximum operating speed is about 20 m/min.

Within these working ranges, both the lowering and raising speeds may be finely controlled, particularly on the modern independently powered electric or hydraulic winches. For the traditional diesel powered all-mechanical transmission winches, this is achieved by gear selection in conjunction with a brake and clutch arrangement. The hoisting speed may be further adjusted by altering the engine speed with the throttle.

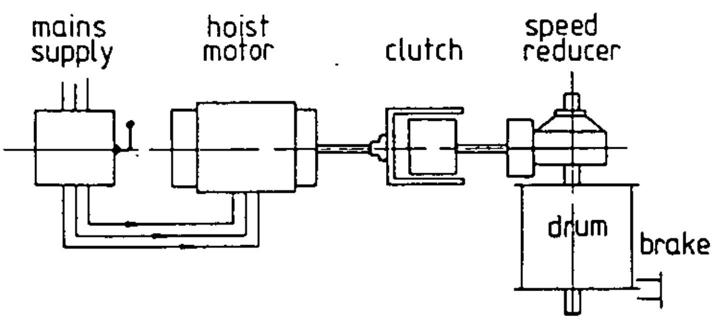


Figure 12.9 Typical arrangement of an electric winch

Chapter 13 CRANES – SHEAR LEGS AND DERRICKS

INTRODUCTION

The applications of the simple block and tackle are somewhat restricted by the limitation of reach. Thus cranes of various forms have been designed to combine both horizontal and vertical movement, so providing a three-dimensional capability.

The earliest devices for use in civil engineering work were shear legs and later the derrick. Even today shear legs are widely used as a site-made temporary works crane, designed for special tasks peculiar to the particular duties called for on the project.

SHEAR LEGS

Where very heavy lifts are needed, then shear legs offer a very simple and inexpensive method. The equipment comprises a winch, and block and tackle suspended from a pin-jointed frame fabricated from steel tubing or RSJs, etc.,

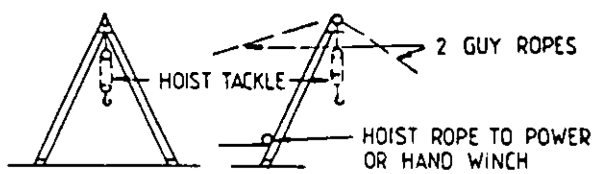


Figure 13.1 Shear legs

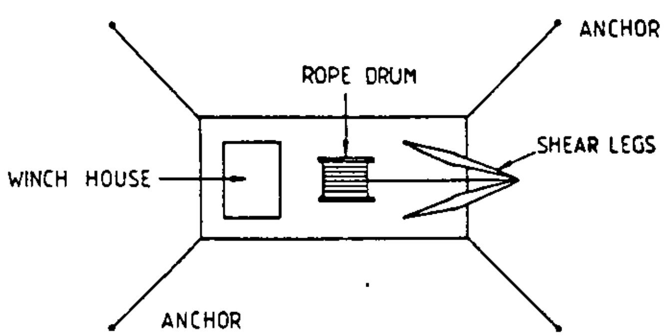


Figure 13.2 Shear legs mounted on a pontoon

stabilised by guy ropes. Loads may only be raised and lowered, horizontal movement is not available. However, for river work, shear legs mounted upon a barge or pontoon can offer three-dimensional movement. Several bridges have been erected using this technique, but great skill in manoeuvring the vessel is required, and generally movement is only obtained by winching from anchorages located at convenient positions.

DESIGN PRINCIPLES

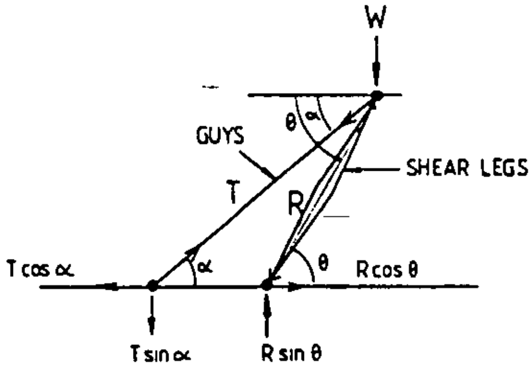


Figure 13.3 Forces acting on shear legs

Resolving horizontally

$$T \cos \alpha = R \cos \theta \tag{1}$$

Resolving vertically

$$T \sin \alpha + W = R \sin \theta \tag{2}$$

Thus T or R is calculated by solving equations (1) and (2). It is usual to apply a factor of safety of at least three to the forces in the members when selecting the actual material requirements.

GUYED DERRICK

The guyed derrick consists of a single boom and mast. The mast stands vertically and is guyed to anchorages in a similar fashion to shear legs. The arrangement allows both luffing (changing of radius) and slewing (turning),

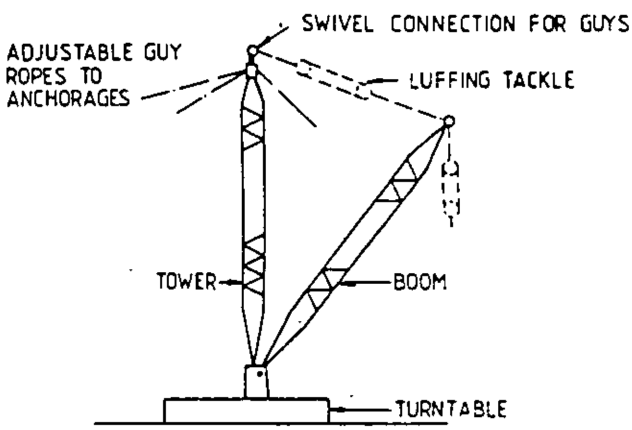


Figure 13.4 Guyed derrick

but lifting is usually only attempted under a guy rope. It is possible to arrange for full 360° slewing, which makes for a very versatile crane and was at one time widely used where lifting facilities were required over a long period, and could justify the setting-up costs, e.g. the erection of steel-framed structures.

DERRICK

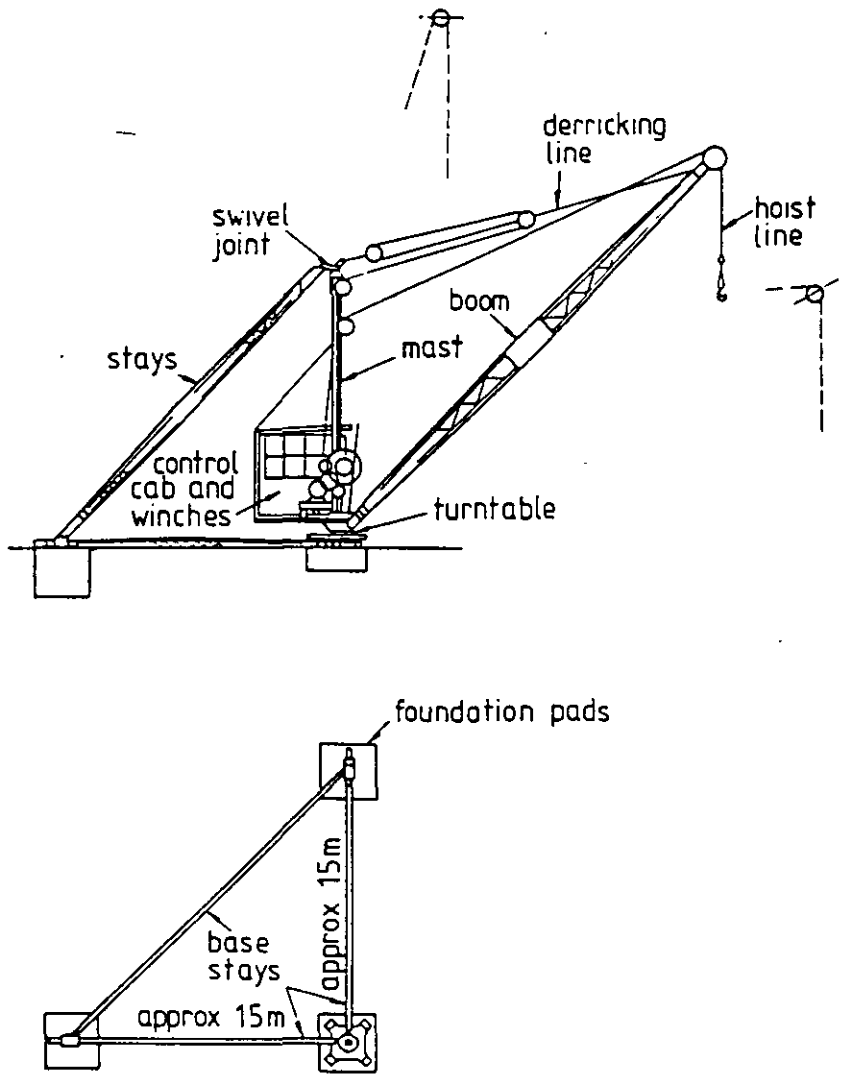


Figure 13.5 Derrick

The Scotch or stiff-leg derrick is more commonly chosen in preference to either the shear legs or guyed derrick, and is used for heavy lifting over long and high reaches. The derrick is extensively used for steelwork erection, especially in heavy plant construction, such as power stations and process plants, although with the continued development of the tower crane it is gradually losing its advantages.

CONSTRUCTION

The derrick consists of a vertical mast, usually made of steel plate, supported by two sloping fixed legs. The whole arrangement is seated upon a triangular frame of lattice construction, with the centre mast free to rotate on bearings at its top and bottom supports. The boom, attached to the base of the mast may be rotated through 270°, between the mast stays, and is capable of hoisting, slewing and luffing. The stays are relatively short compared to the length of the boom, and heavy ballasting is required at the base plates of the mast and stays. As a general rule the weight of kentledge at each ballasting point should be about four times the maximum lifting capacity of the crane. It is therefore essential that the unit is firmly supported on well prepared foundations. A common method of providing suitable foundations is to mount the derrick on bogies at the base apex points. The bogies themselves are supported on rails and sleepers, to provide the extra dimension of mobility.

The working height of the derrick may be increased by means of lattice towers, called gabbards, placed under the base apex points.

DESIGN PRINCIPLES

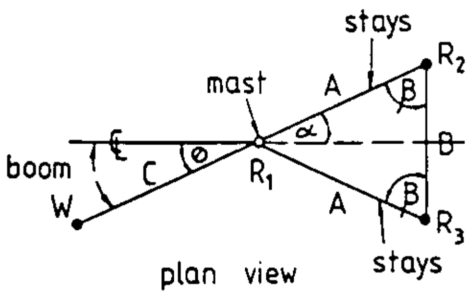


Figure 13.6 Forces acting on a derrick

(a) Taking moments about a line through R_2R_3

$$W \times (C \cos \theta + A \cos \alpha) = R_1 \times A \cos \alpha$$

$$R_1 = W \frac{(C \cos \theta + A \cos \alpha)}{A \cos \alpha} \qquad (3)$$

Therefore when θ increases to 90°, R_1 is equal to W.

(b) Taking moments about a line through R_1R_2

$$R_3 \times B \sin \beta = W \times C \sin (\alpha \mp \theta)$$

$$R_3 = W \times \frac{C \sin (\alpha \mp \theta)}{B \sin \beta} \qquad (4)$$

(c) Taking moments about a line through R_1R_3

$$R_2 \times B \sin \beta = W \times C \sin (\alpha \mp \theta)$$

$$R_2 = W \times \frac{C \sin (\alpha \mp \theta)}{B \sin \beta} \qquad (5)$$

It can be seen that as θ changes the reactions also change.

METHOD OF ERECTION

Complete erection, including commissioning, takes about 40 hours using three men and crane assistance. Dismantling can be achieved in about half the erection time. The following procedure is recommended:

(a) prepare the foundation bases and position the base frame;
(b) place the mast sole plate on the frame;
(c) erect the crane mast and temporarily guy to anchors;
(d) erect the stays in turn and secure with holding bolts;
(e) load on ballast;
(f) connect boom to mast;
(g) fit gears, winches, ropes, pulleys, etc.;
(h) raise the boom.

OPERATING THE DERRICK

The derrick is provided with two rope drums, one for derricking (i.e. luffing) and one for hoisting. Both are driven through gearing by an electric or hydraulic motor, steam or diesel engine. The more common form is electric, when separate motors are provided for slewing, hoisting and luffing. The hoisting facility is usually available with a gear change, the fast speed for light loads, and the slow speed for heavy loads. Some types of derrick have a third drum for opening a clamshell bucket when used as a grab.

DERRICK CRANE CHARACTERISTICS

The derrick is designed as a pin-jointed lattice frame structure to lift a certain load, called the design capacity, and is typically available with maximum load capacities up to 200 tonnes. Table 13.1 shows the corresponding maximum and minimum radius of operation for the more common sizes. It can be seen in fig. 13.7 that between the minimum possible radius (x) and maximum permitted radius (y) the load is limited to the design capacity load. For loads less than the design capacity the permitted operating radius (z) may be slightly increased. The corresponding values are shown in table 13.2. The working ranges for various lengths of boom are given in fig. 13.8.

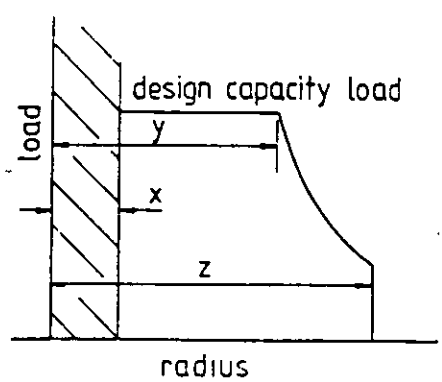


Figure 13.7
Load–radius diagram for the derrick

Figure 13.8 Boom height–radius diagram for the derrick

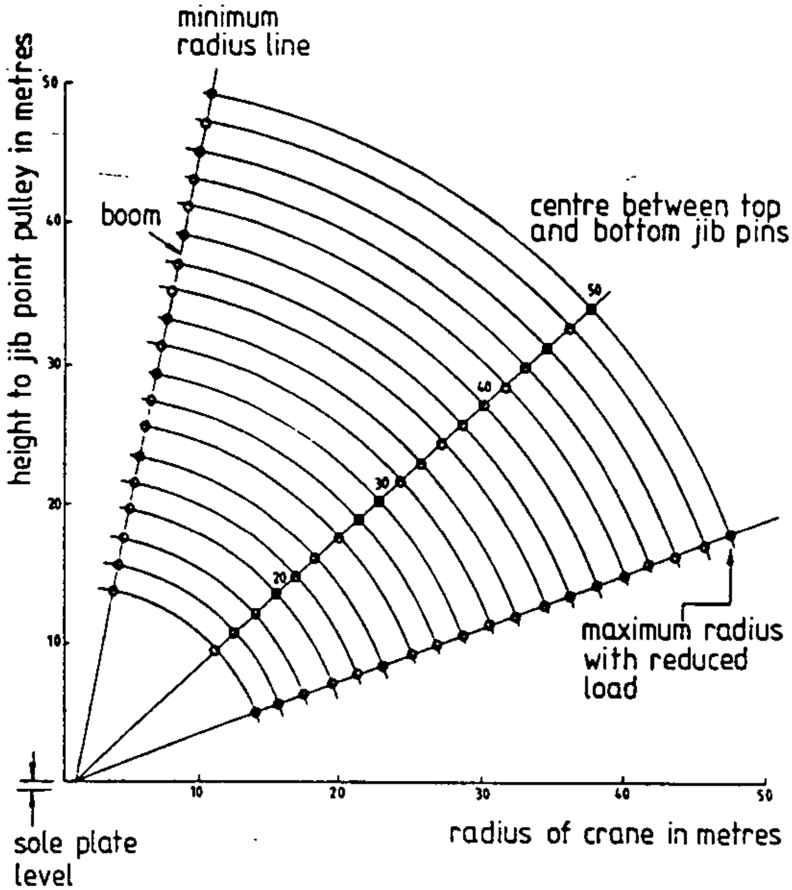


Table 13.1 Maximum operating radius at design capacity of derrick

Boom length (m)	*Max. (y) radius (m)*	*Design capacity (tonnes)*											
		3	*5*	*7*	*10*	*15*	*20*	*25*	*30*	*35*	*40*	*55*	
36	27	8	8.5	8.75	9	9	9.25	9.25	10	11	11	11	Min. operating
46	36	9	9	9.5	9.5	10	10.5	11	11	11	11	12	radius (*x*) (m)

Table 13.2 Permissible load at maximum operating radius of derrick (*z*)

Boom length (m)	*Radius (z)(m)*	*Design capacity (tonnes)*											
		3	*5*	*7*	*10*	*15*	*20*	*25*	*30*	*35*	*40*	*55*	
36	35	2.5	2.5	3	4.25	5	5.75	7.5	7.5	8.75	10	14	Load (tonnes)
46	42.5	2.5	2.5	3	4.25	5	5.75	7.5	7.5	8.75	10	14	at radius (*z*)(m)

DERRICK CHARACTERISTICS

	Up to 7 tonnes capacity	*10 tonnes capacity and over*
Hoisting speed – lifting design capacity (m/min)	30–35	10–15
Hoisting speed – lifting light load (m/min)	70	20–30
Derricking (luffing) speed (m/min)	30	12–15
Slewing speed (rev/min)	1	0.3
Hoist motor (kW)	50	50
Slewing motor (kW)	10	30
Derricking motor (kW)	40	50
Travelling speed (m/min)	40	10
Travelling motor (kW)	20	60

Note: All are electric motors.

EXAMPLE OF DERRICK SELECTION

A derrick crane is used to construct three concrete monoliths 20 m × 10 m on plan, for the widening of the docks entrance shown in fig. 13.9. The monoliths are constructed in 1.5 m lifts and a lift of reinforcing steel protrudes above the top of the shuttering. A maximum of four lifts of concrete is allowed above ground level as shown. The crane is to be used (i) for all materials handling including a 764 litre (1 yd^3) concrete skip, (ii) for excavating by grabbing action inside the monoliths with a 1000 litre (33 ft^3), 15° CECE rating, capacity grab, (iii) for lifting a complete cell of formwork weighing 3 tonnes.

Problem

Select a suitable size derrick crane.

Solution

(a) *Choosing the correct position*
Figure 13.9 shows that given the smallest available boom of 36 m and allowing two clear $1\frac{1}{2}$ m lifts between the top of the reinforcement and the underside of the boom, the derrick can reach the four sides of the monolith when set up at its minimum operating radius of 8.5 m (position A on the section diagram), or alternatively at the crane's maximum radius (position C). This latter position however, is beyond the maximum permissible operating radius for the design capacity of the derrick. Clearly, therefore, the derrick can provide clearance at the maximum load permitted radius of 27 m (position B) and thus may be established in any position between points A and B. The top corners of each monolith, however, are outside the reach of the boom at all placings. Therefore the derrick should be rail mounted to cover all three monoliths.

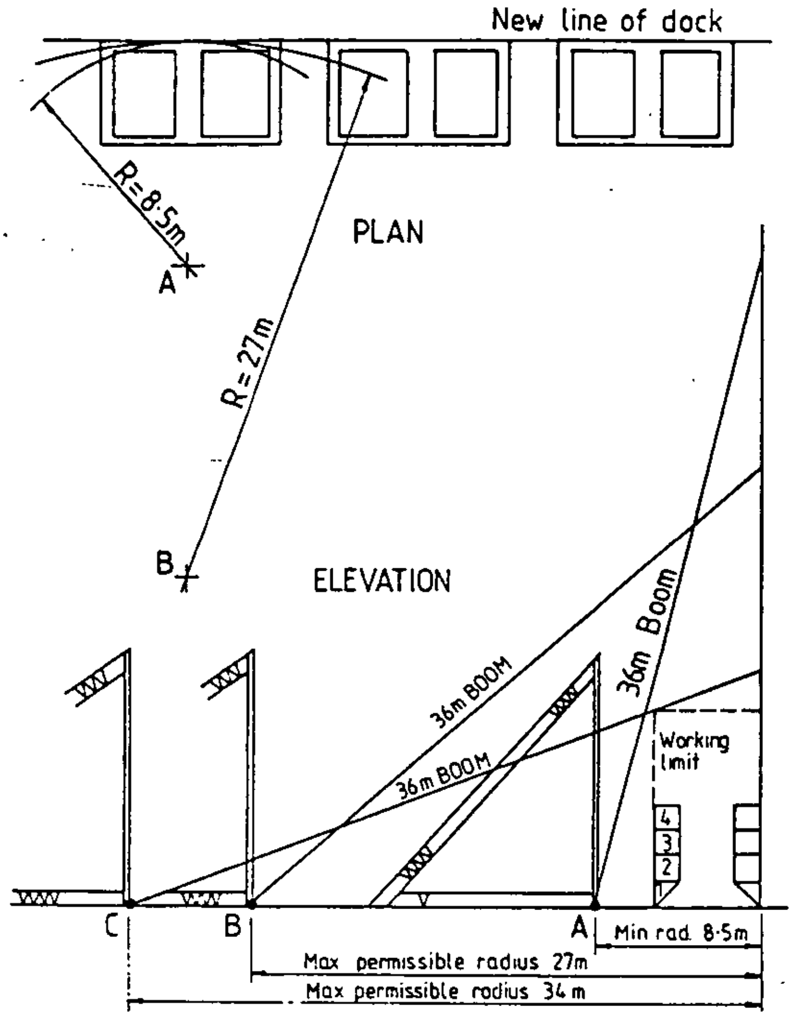


Figure 13.9 Example layout of derrick crane

(b) *Choosing the appropriate crane capacity*

Possible loads are:

(i) Shuttering – 3 tonnes.

(ii) Concrete skip + concrete

1 yd^3 skip	=	500 kg
1 yd^3 concrete	=	1800 kg
		2300 kg

(iii) Grab + contents

1000 litre grab	=	1350 kg (see table 14.4)
moist earth	=	1875 kg (heaped) (see table 14.2)
		3225 kg

(iv) Hook block, etc. = 775 kg.

Max. possible load = 3225 + 775 = 4000 kg.

Include an extra 25% for surcharge, thus max. load = 4000 × 1.25 = 5000 kg.

A 5 tonnes capacity derrick is needed.

(c) *Ballast*

The ballast required on each foundation is approximately 4 × derrick capacity, i.e. 20 tonnes.

Chapter 14 CRAWLER-MOUNTED CRANE

INTRODUCTION

On many construction sites a crane is needed to lift small to medium loads, such as concrete skips, reinforcement, formwork, etc. Flexibility of movement around the site is at a premium, thus where the work is spread over a wide area beyond the reach of tower cranes and derricks, a crane capable of operating on unprepared surfaces is demanded. Rubber-tyred mobile cranes are excellent for lifting on level firm surfaces, but on many sites the ground conditions are so bad that these models would become bogged down and unable to work easily and efficiently. In conditions such as these the crawler-mounted crane is the most advantageous model to use. This is because the weight of the crane is spread over a large bearing area under wide and long tracks.

The crawler crane has the further advantage that conversion from crane to grabbing crane or dragline for excavation purposes is readily achieved.

CRANE CONSTRUCTION

The crane is built in three sections; the base frame, superstructure and boom, the whole unit being powered by a diesel engine.

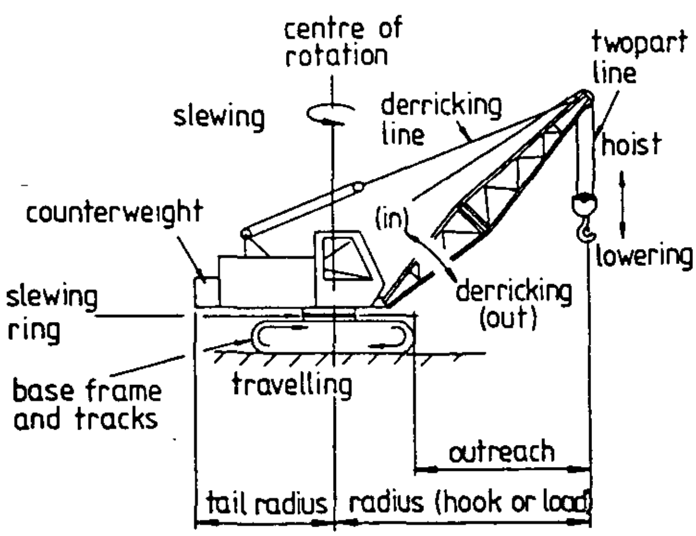


Figure 14.1 Crawler-mounted strut-boom crane

THE BASE FRAME

The base frame is made from a welded steel channel to which the two machine axles are attached, and supports the weight of the engine, gearing and winches, controls, cab, boom and counterweight.

THE SUPERSTRUCTURE

The superstructure consists of a revolving frame sitting on a large turntable mounted on the base frame to give 360° slewing. The engine, gears, winches and counterweight are all mounted on this part of the unit.

The machine relies on a mechanical or hydraulic transmission system and two rope drums with independent brakes and clutches facilitate gravity lowering of the load. The front drum near the boom is the hoist winch and serves to raise and lower the crane hook and load, while the rear drum is used for luffing the boom.

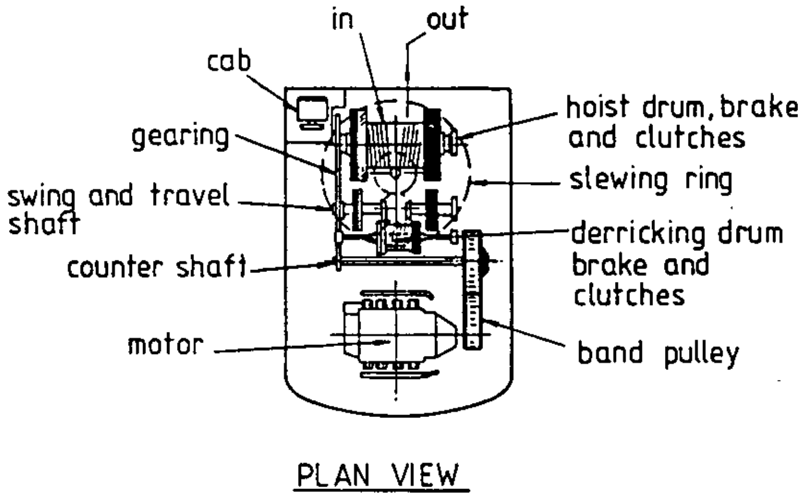


Figure 14.2 Gearing and winching layout (mechanical-type transmission)

THE BOOM

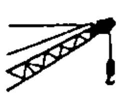


Figure 14.3 Simple head sheave

The basic boom is assembled in two sections each constructed from four high tensile rectangular hollow sections braced with round tube lacing members. The sections are pin-connected together and the top section incorporates a head sheave (fig. 14.3) for light loads of up to 10 tonnes or a hammerhead boom point (fig 14.4) for heavy lifts. Intermediate sections may be inserted to extend the length of the boom.

Figure 14.4 Hammerhead boom point

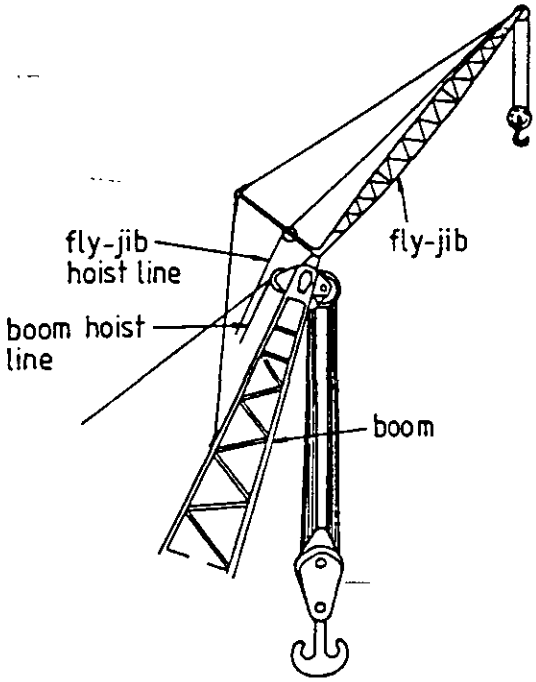
HOISTING TACKLE

The hoist rope between the head sheave and hook block may be arranged with one, two, three, four or more falls to accommodate the load being raised as described in chapter 12. A single part tackle naturally permits fast hoisting, but the weight of the load must be kept within the permissible tensile strength of the cable.

FLY-JIB

The straight boom suitably extended is ideal for general lifting duties but

Figure 14.5 Fly-jib



because of the inclined angle, obstructions often restrict load positioning. To overcome this problem a fly-jib may be attached to the boom point as shown in fig. 14.5.

The fly-jib is of similar construction to the main boom and is available in various lengths depending on the duties and capacity of the crane mounted in line with the main boom to act as a simple extension or more customarily at 30° offset to provide increased operating radius. If the crane is supplied with a third drum it is feasible to use this as the hoist winch with the fly-jib, leaving the main tackle on the boom for heavy lifts. Fly-jibs are designed for load lifting purposes only and are not suitable for grabbing crane or dragline operations.

WORKING LOADS

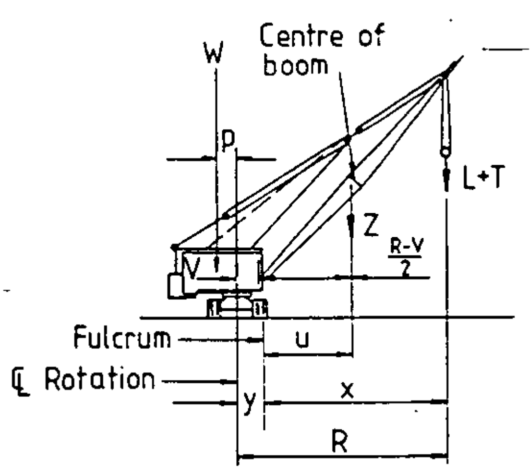


Figure 14.6 Forces acting on the strut-boom crane

It is preferable to operate the crane on a flat well-prepared surface. The tipping load may be determined by using the following formula.

$$L = \frac{W(p + y) - Zu}{x} - T$$

where

L = tipping load of the crane;
W = weight of the machine without the boom (but including counter-weights);
Z = weight of boom;
T = weight of head sheave;
R = radius to load from centre of slewing ring;
y = fulcrum distance;
p = centre of gravity of machine without boom to centre line of slewing ring;
v = connection point of boom to centre line of slewing ring;
x = $R - y$;

$$u = \frac{(R - v)}{2} + v - y.$$

Therefore safe working load (P) = L − margin for safety.

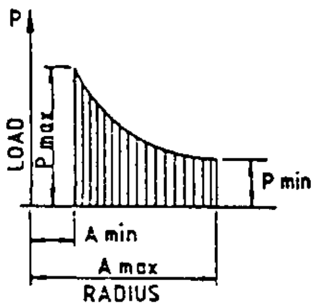


Figure 14.7
ad–radius diagram for
..e strut-boom crane

The shape of the load–radius diagram is shown in fig. 14.7. The weight of the hook block, together with any slings, etc., should be included when selecting a crane of suitable lifting capacity. If a fly-jib is attached but not in use then the safe working load should be reduced in accordance with the manufacturer's recommendations. Approximate load reductions are:

Fly-jib length (m)	*Load reduction (kg)*
6	850
9	1000
12	1200
18	1500
24	2100
30	2600
36	3000

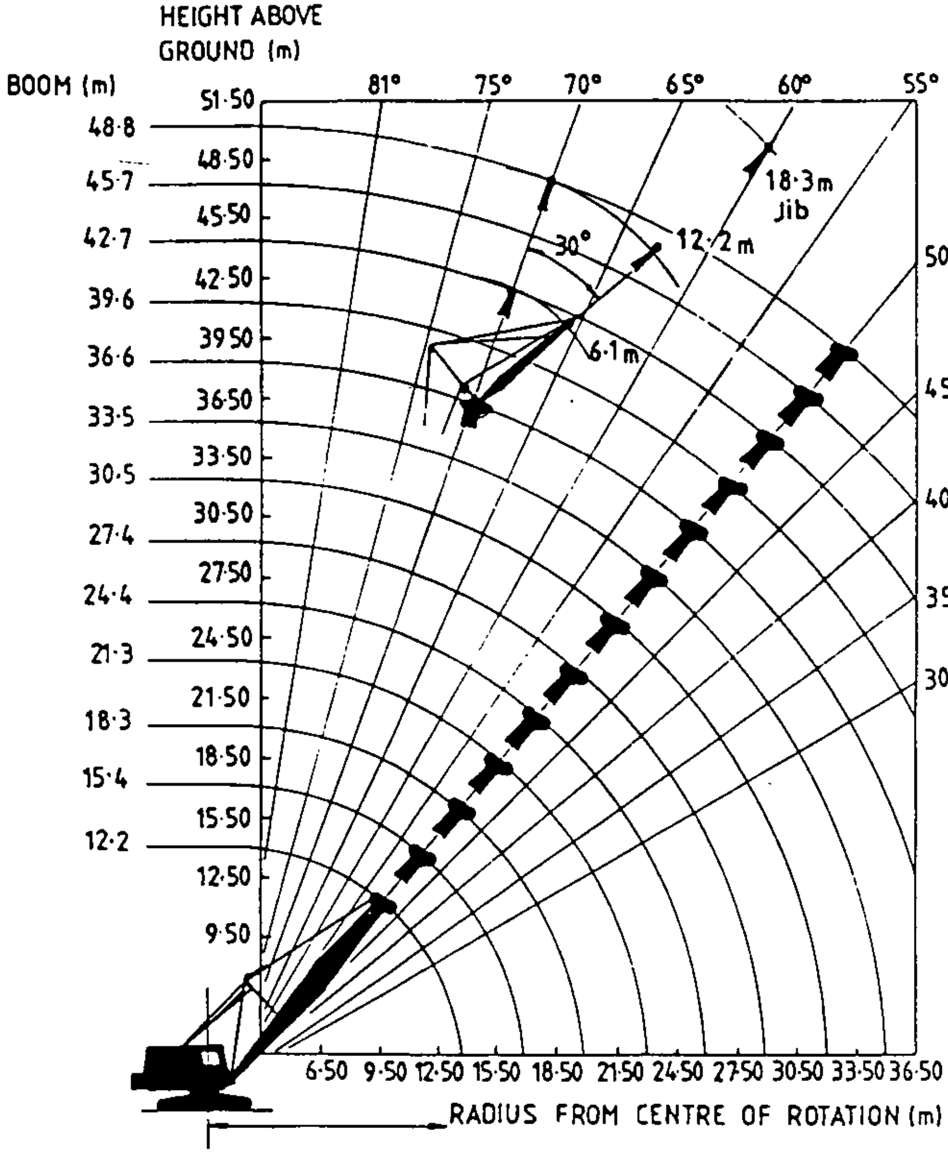
Crane Radius Diagram

The precise range of boom lengths and the corresponding reaches and radii vary with individual manufacturers, but the dimensions shown in fig. 14.8 are typical of the makes of machine that are available.

CRANE CAPACITIES

The lifting ability at a given radius varies slightly with the particular crane manufacturer, with capacity being designated in terms of the maximum load

Figure 14.8a Boom height–radius diagram for strut-boom crawler crane: (i) Crane supported on tracks only: (ii) Crane on tracks and ringer system



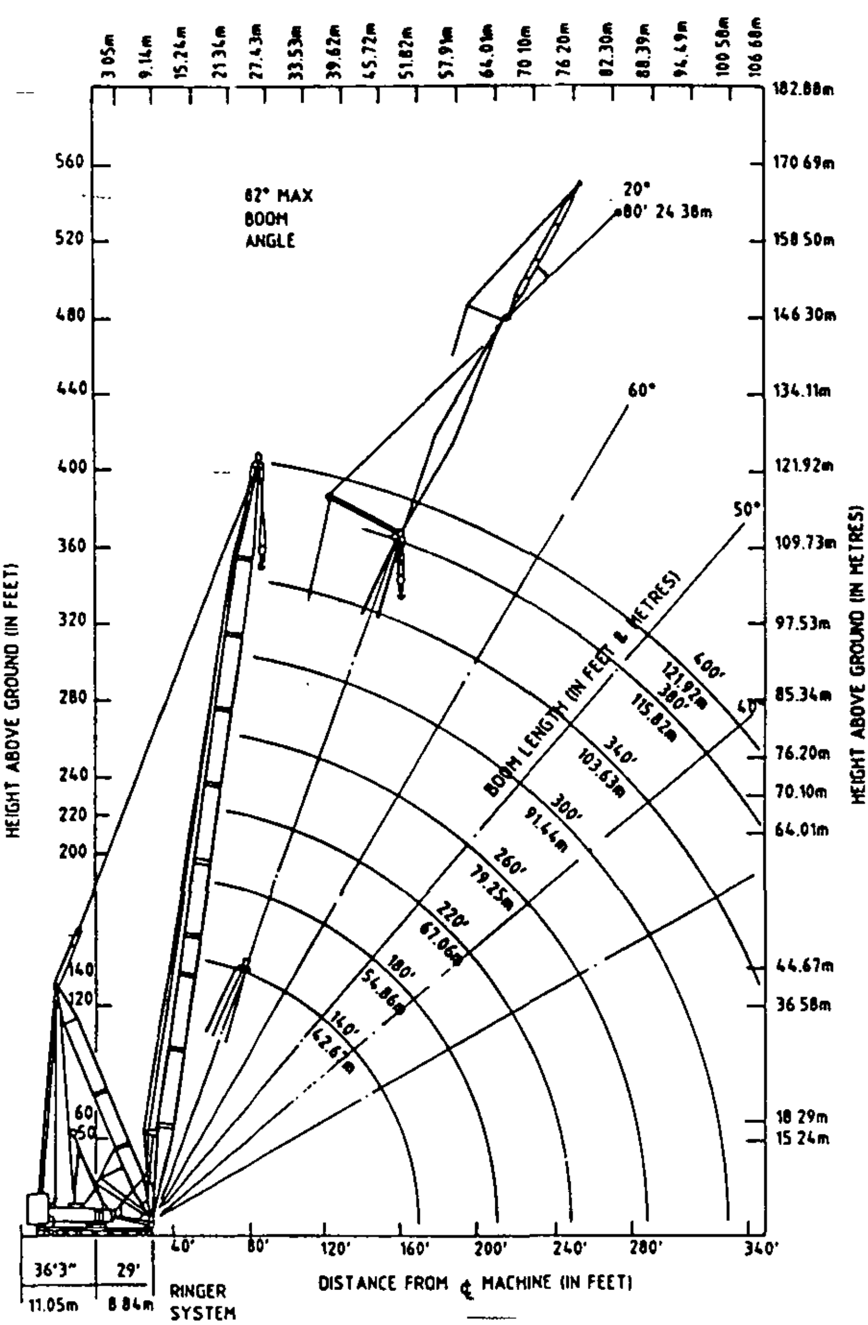
(i)

at the minimum operating radius. The most popular sizes are in the range 15 to 120 tonnes but cranes with capacities of 3000 tonnes or so are available. These larger units often have a ringer configuration (fig. 14.9), which as a rough rule-of-thumb doubles the SWL compared to the conventional tracked mounted version.

TRANSPORT AND ASSEMBLY

The crane is not intended to be self-transporting and will travel at little more than walking speed, and must be moved from depot to site on a low-loader truck. This choice is therefore not a practical option when lifting capacity is required for only a short period of hours or even a few days as the time taken to load, transport, unload and prepare for work may require a full day or more.

Figure 14.8b



(ii)

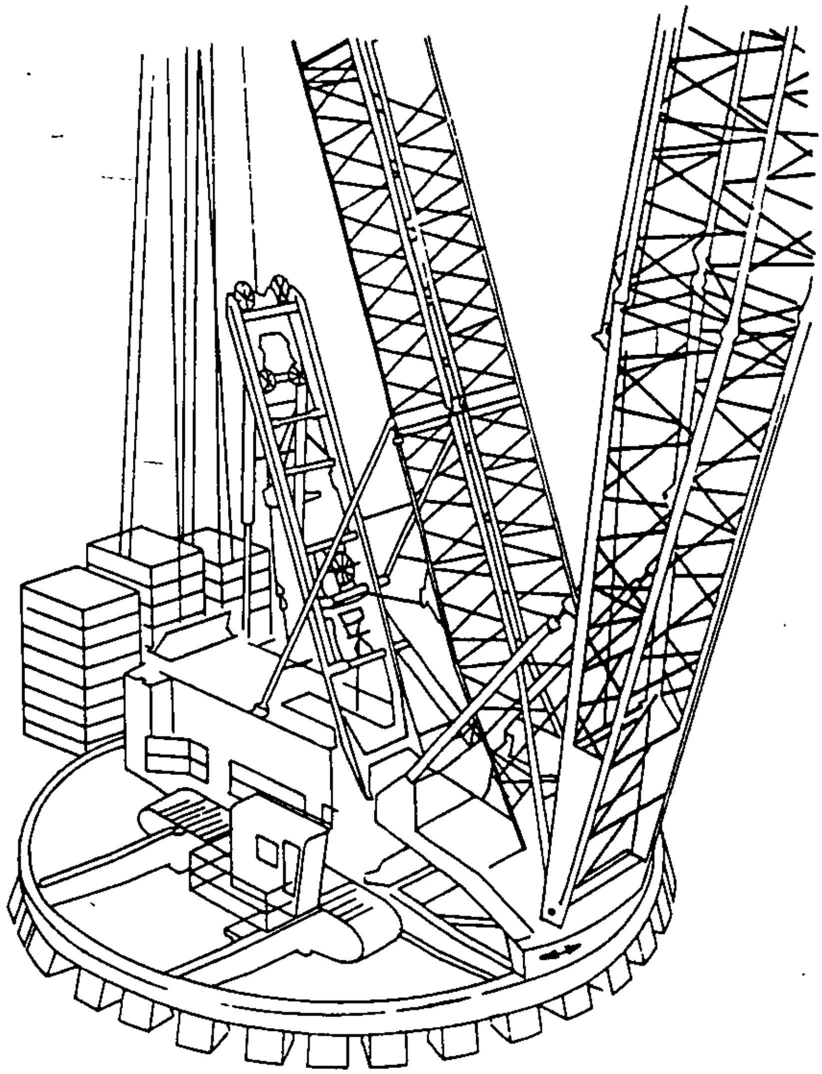
CONVERSION OF STRUT-BOOM CRANE TO GRABBING CRANE OR DRAGLINE (SEE CHAPTER 2)

Conversion to a grabbing crane or dragline can be readily obtained by changing the size of the hoist rope and hoist drum. The derricking drum is retained and an additional drum (i.e. third drum) included to control the bucket. Larger and wider tracks are often fitted to provide greater stability.

WORKING RANGES OF THE DRAGLINE AND GRAB CRANE

The dragline, like the crane, is usually designated with safe working loads

Figure 14.9 Ringer configuration



based on a fixed percentage of the tipping load, depending on the country of operation, with the grabbing crane (and magnet cranes) normally being 80% of crane values.

The boom size may be varied to suit the required operating radius and lifting height as shown in fig. 14.10. As a rule-of-thumb the dragline is capable of digging to a depth below its tracks of roughly from one-third to half the length of the boom. Furthermore, the throw of the bucket beyond the radius of the boom may be of similar length, depending upon the skill of the operator.

The safe working loads of cranes shown in table 14.1 can be used to calculate the dragline working range, but should be reduced by 20% for grab cranes. Also, in practice, the working load may be further restricted in order to maintain an adequate factor of safety on the roping system, which will be in single fall reeving. The weight of the bucket must be included in the weight of the load to be handled, as recommended in tables 14.2, 14.3, 14.4 and fig. 14.11.

Figure 14.10 Working ranges of the dragline nd grabbing crane

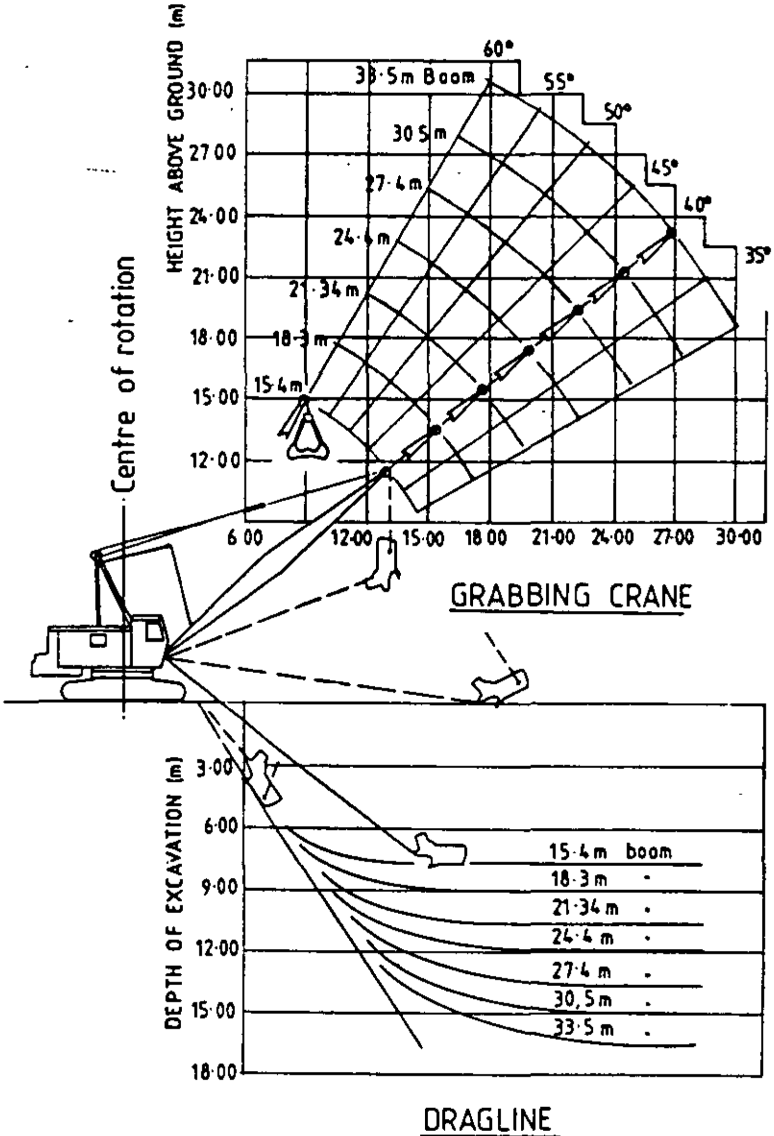


Table 14.1 Guidelines to safe working loads of crawler-mounted strut-boom cranes.

(a) Main Boom Rating: 15 Tonnes Crane				
	Boom (m)			
Radius (m)	*10.7*	*15.2*	*21.3*	*27.4*
3.0	15.00			
4.5	8.6			
6.0	5.6	5.5	5.4	
7.5	4.2	4.1	3.9	3.7
9.0	3.2	3.1	3.0	2.9
10.5	2.7	2.5	2.5	2.4
12.0		2.1	2.0	1.9
15.0		1.5	1.4	1.4
18.0			1.0	0.9
21.0				0.8
24.0				0.7

(b) Main Boom Rating: 20 Tonnes Crane

	Boom (m)			
Radius (m)	*10.7*	*15.2*	*21.3*	*30.5*
3.0	20.3			
4.5	10.6			
6.0	6.6	6.4	6.4	
7.5	4.9	4.7	4.6	4.5
9.0	3.9	3.7	3.6	3.4
10.5	3.3	3.0	2.9	2.8
12.0		2.5	2.9	2.2
15.0		1.8	1.7	1.5
18.0			1.2	1.1
21.0				0.8
24.0				0.5

(c) Main Boom Rating: 35 Tonnes Crane

	Boom (m)					
Radius (m)	*12.2*	*15.2*	*18.3*	*21.3*	*24.4*	*27.4*
3.5	35.0					
6.0	17.0	16.9	16.8	16.7	16.2	
8.0	11.4	11.2	11.1	11.1	10.9	10.8
10.0	8.4	8.2	8.1	8.1	7.9	7.8
12.0	6.5	6.4	6.2	6.2	6.0	5.9
14.0	5.4	5.1	5.0	4.9	4.7	4.6
16.0				4.0	3.8	3.7
18.0				3.3	3.1	3.0
20.0				2.7	2.6	2.5
22.0					2.1	2.0
24.0						1.7
26.0						1.4

(d) Main Boom Rating: 67 Tonnes Crane

	Boom (m)					
Radius (m)	*15.2*	*21.3*	*27.4*	*33.5*	*39.6*	*45.7*
3.5	67.0					
6.0	33.0	31.9	30.0			
8.0	21.5	20.9	20.3	19.4	17.5	
10.0	15.5	15.1	14.6	14.2	13.9	12.3
12.0	12.2	11.8	11.5	11.2	11.1	10.4
16.0	8.1	7.5	7.3	6.9	6.7	6.3
20.0		5.3	4.8	4.6	4.3	4.1
24.0			3.8	3.4	3.1	3.0
28.0				2.7	2.2	2.1
32.0					1.5	1.4
36.0					0.9	0.8

(e) Main Boom Rating: 91 Tonnes Crane

	Boom (m)			
Radius (m)	*18.3*	*27.4*	*36.6*	*45.7*
4.5	91.0			
6.0	82.8	63.8		
8.0	42.9	42.1	41.1	
10.0	32.3	31.5	30.6	30.0
12.0	25.4	24.6	23.7	23.1
16.0	17.1	16.3	15.4	14.7
20.0		11.7	10.8	10.1
24.0		8.8	7.8	7.2
28.0			5.8	5.2
32.0			4.3	3.7
36.0				2.6
42.0				1.3

(f) Main Boom Rating: 110 Tonnes Crane

	Boom (m)					
Radius (m)	*18.3*	*27.4*	*36.6*	*45.7*	*57.9*	*71.6*
4.5	110.0					
6.0	77.6	22.0				
8.0	59.9	59.3	51.6			
10.0	46.8	46.2	45.7	35.2		
12.0	37.1	37.5	37.1	35.1	21.4	
16.0		25.4	24.9	24.7	20.1	12.8
20.0		18.9	18.5	18.3	17.6	12.3
24.0		14.9	14.4	14.2	13.6	11.8
28.0			11.7	11.5	10.9	10.2
32.0			9.7	9.5	8.8	8.2
36.0				8.0	7.4	6.7
42.0				6.3	5.7	4.8
48.0					4.4	3.7
52.0					3.8	3.1
58.0						2.2

(g) Main Boom Rating: 217 Tonnes Crane

	Boom (m)					
Radius (m)	*18.0*	*27.0*	*36.0*	*45.0*	*57.0*	*63.0*
4.5	217.8					
6.0	180.0					
8.0	122.5	122.0				
10.0	87.0	86.6	86.3			
12.0	67.1	66.7	66.3	65.7		
16.0	45.4	44.9	44.5	44.0	43.0	42.8
20.0		33.3	32.9	32.4	31.6	31.2
24.0		26.0	25.7	25.1	24.3	23.9
28.0			20.7	20.2	19.4	19.0
32.0			17.1	16.6	15.8	15.4
42.0					10.9	9.5
48.0					7.6	7.2
56.0						5.0

(h) Main Boom Rating: 270 Tonnes Crane

	Boom (m)							
Radius (m)	*21.34*	*27.43*	*36.58*	*45.72*	*51.82*	*57.91*	*64.01*	*73.15*
5.6	270.0							
7.0	212.0							
9.0	148.4	148.2						
12.0	94.2	94.0	93.5	93.2				
16.0	62.3	62.00	61.4	61.0	60.5	60.2	59.9	59.4
20.0	45.7	45.4	44.8	44.5	44.1	43.8	43.4	42.9
30.0			25.4	25.0	24.6	24.3	24.0	23.5
40.0				16.1	15.6	15.2	14.9	14.4
45.0					10.7	10.3	9.8	9.2
50.0							6.8	6.1
55.0								5.0

Crawler crane.

(i) Main Boom Rating: 550 Tonnes Crane

	Boom (m)												
Radius (m)	*22.86*	*30.48*	*38.1*	*45.72*	*53.34*	*60.96*	*58.58*	*76.2*	*83.82*	*91.44*	*99.06*	*106.68*	*114.3*
5.4	544												
7.0	442	440											
8.8	362	359	357	342									
12.2	229	228	228	225	230	230	227	224					
21.3	100	99	99	97	102	102	98	98	98	98	97	96	94
30.5		60	60	57	60	61	60	59	58	58	57	56	54
39.6				38	43	42	41	40	41	39	38	37	35
48.8					32	31	30	29	28	27	26	25	24
57.9						23	22	21	20	19	18	17	17
67.1							17	16	15	15	14	13	11
77.7									11	10	9	8	6

Crawler crane.

(j) Main Boom Rating: 680 Tonnes Crane with 18 m dia. ringer

	Boom (m)						
Radius (m)	*42.7*	*54.9*	*67.1*	*79.2*	*91.4*	*103.6*	*121.9*
21.3	680	676	503	391			
30.5	370	381	379	353	273	217	149
39.6	232	261	258	256	247	197	134
48.8			193	190	188	176	118
57.9			147	149	147	144	102
67.1			107	118	115	116	88
77.7				85	92	92	57
82.3					81	85	40
86.9					70	75	—
91.4					61	55	—

Crawler crane.

(k) Main Boom Rating: 910 Tonnes Crane

Radius (m)	*Boom (m)* 47.2	62.5	77.7	92.9	108.2	123.4	138.6	153.9
13.7	907							
21.3	500	495	492	476	415			
30.5	295	290	287	284	281	252	224	161
39.6	205	200	196	193	190	185	170	128
48.8	153	148	144	141	138	133	129	99
57.9		115	111	108	105	98	96	75
67.1			88	85	82	77	73	53
77.7			71	65	62	57	53	43
82.3				59	56	50	46	41
86.9				52	49	44	40	37
91.4				48	44	35	35	32
93.5					38	32	29	25
103.6					33	27	26	20

Crawler crane.

Table 14.2 Approximate densities of soils

Material	*Density, lb/yd³ (kg/m³)*
Earth – moist	2500 (1490)
Sand – dry	2700 (1600)
Sand – wet	3300 (1960)
Gravel	2900 (1720)
Loose stone	2700 (1600)
Clay – wet	3000 (1780)
Coal	1350 (800)

Table 14.3 Dragline bucket data

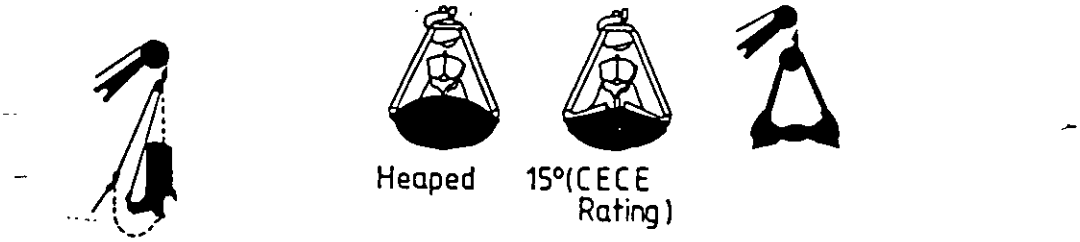
Capacity	(yd^3)	5	4½	4—	3½	3	2½	2	1¾	1½	1¼
	(m^3)	3.82	3.44	3.06	2.670	2.29	1.91	1.53	1.35	1.15	0.96
Weight	(lb)	9200	7700	7000	6400	5700	4700	4250	3300	2900	2300
empty	(kg)	4175	3495	3175	2905	2585	2130	1925	1495	1315	1040
Length	(m)	6.8	6.2	6.1	5.9	5.8	5.5	5.5	4.8	4.7	4.4

Table 14.4 Grabbing crane – medium-weight grabs

Capacity*	(ft^3)	100/80	90/72	80/64	71/57	63/51	50/40	44/35
	(m^3)	2.75/2.25	2.5/2.0	2.25/1.75	2.0/1.6	1.75/1.50	1.4/1.1	1.25/1.00
Weight	(lb)	6550	5200	5100	3900	3850	3050	2950
empty	(kg)	2975	2400	2350	1800	1750	1375	1350
Length	(m)	4.4	4.2	4.0	3.8	3.8	3.5	3.4

* Capacities given are heaped with 15° (CECE rating).

Figure 14.11 Dragline and grabbing buckets



GROUND PRESSURE UNDER THE TRACKS

Pressure is given in N/mm^2.

	Crane capacity *30 tonnes*	*80 tonnes*
Crawlers, standard tracks	0.06	0.113
Short crawlers, wide tracks	0.06	0.098
Long crawlers, standard tracks	0.055	0.076
Long crawlers, wide tracks	0.047	0.067

The pressure from a human foot is 0.02 N/mm^2.

LIFT CRANE DATA

Hoisting speed (single fall line)	approx. 40–50 m/min
Derricking (max. to min. radius)	approx. 50–100 s
Slewing speed	approx. 2 rev/min
Travelling speed	2½–3 km/h
Max. gradients when travelling: loaded	1 in 16
no load	1 in 5

EXAMPLE OF CRAWLER CRANE SELECTION

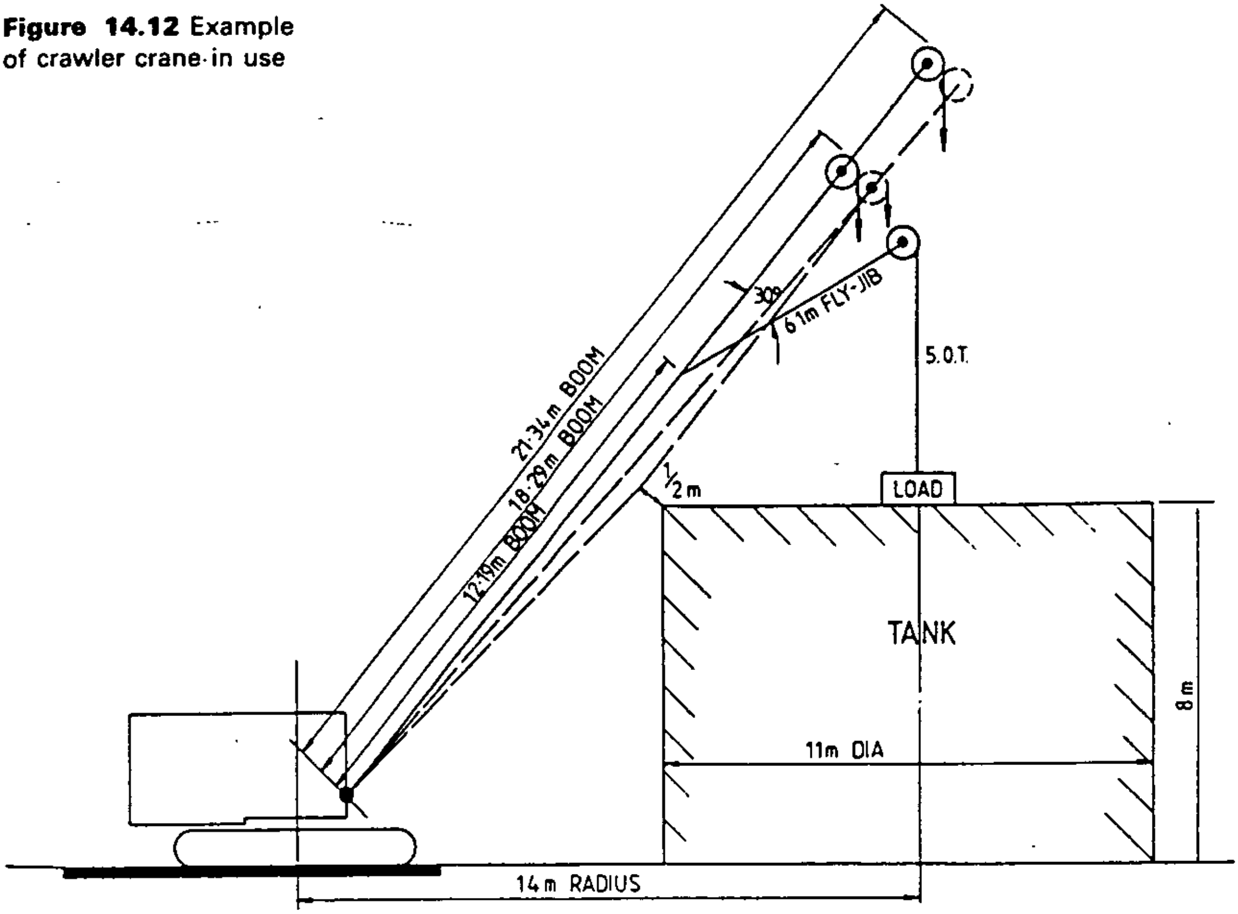
A crawler crane is used to lift a load of 5 tonnes (including hook block and tackle) from a temporary road (which acts as a flat and well-founded surface), to the centre of a reinforced concrete tank, as shown in fig. 14.12. The crane must provide ½ m gap between the underside of the boom and the edge of the tank. Select an appropriate crane to perform the task.

Solution

(a) The crane is operated from the temporary road and with the gap between boom and tank maintained at the minimum distance of ½ m a crane with a 21.34 m long boom is required. The alternative smallest next size of 18.29 m will not provide the necessary reach of 14 m radius from the centre line of rotation of the machine to the lifting hook.

Reference to table 14.1 reveals that the smallest size of crane capable of lifting 5 tonnes at 14 m radius is the 35 tonnes capacity crane However, this may only be achieved with an 18.29 m boom, which is too short. The next size of crane would therefore be required.

Figure 14.12 Example of crawler crane in use



A cheaper alternative is to attach a 6.1 m fly-jib to a 12.19 m boom on the 35 tonnes capacity crane, (the lifting capacity of the crane is equivalent to the 18.29 m boom). The sketch shows that in this configuration the desired operating radius of 14 m is achieved with some loss of operating height (which is not critical in this case). Thus a load of 5 tonnes may be lifted. It will be noticed that there is sufficient clear height between boom and tank to raise the load clear of the tank rim when slewing the load.

(b) The crane may be used for lifting with the main boom and sheave, with the fly-jib attached and out of action. The safe working load of the crane therefore must be reduced from the values shown in table 14.1 by 850 kg. Thus the maximum 5 tonnes load at an operating radius of 14 m with a 12.19 m boom is reduced to 4.15 tonnes.

(c) The crane may also be re-roped and operated as a dragline or grab crane. (Note – Only from the main boom, not the fly-jib.)

(i) *Grab crane* – the lift crane safe working load is reduced by 20% for grabbing or clamshell purposes.
Small bucket – 1250 m^3 heaped capacity bucket plus dry sand = 1350 kg self-weight + (1600 kg/m^3 × 1.25 m^3) = 3350 kg.
From table 14.1 for a 35 tonnes capacity crane, the permissible operating radius is about 14 m with a 24.40 m boom, i.e. permissible load is 4.7 × 0.8 = 3.76 tonnes, which is greater than actual load of 3.35 tonnes. (Note – from fig. 14.10 jib angle is at the upper limit thus ruling out the longer 27.4 m boom.)

Large bucket – 2750 m^3 heaped capacity bucket plus dry sand = 2975 kg self-weight + (2.75 m^3 × 1600 kg/m^3) = 7375 kg. From table 14.1 for a 35 tonnes capacity crane the maximum operating radius is about 9 m with a 18.2 m boom, i.e. interpolating permissible load 9.5 × 0.8 = 7.6 tonnes, which is greater than ac. load.

Note – from fig. 14.10 jib angle is near the limit.

(ii) *Dragline* – the lift crane safe working loads are directly applicable for dragline selection.

Thus using dragline data from table 14.3 a 1¼ yd^3 (0.96 m^3) capacity bucket + dry sand weight = 1040 + (0.96 × 1600) = 2576 kg. From table 14.1 for a 35 tonnes capacity crane, the permissible operating radius with a boom of suitable length, say 21.34 m, is 2.7 tonnes at 20 m. Assuming the bucket is cast one-third of the distance of the boom length beyond the boom, the boom could be set at an operating radius between 12 and 20 m, to keep within the permitted working range given in fig. 14.10.

A final check should be included to ascertain if the rope strength is sufficient to carry the working load.

Chapter 15 SELF-PROPELLED CRANE ON RUBBER-TYRED WHEELS

INTRODUCTION

Since the end of the second world war considerable growth in the use of cranes on wheels has taken place. The reasons for this are many but undoubtedly an important factor has been the demand for specialist one-off lifting facilities, effectively facilitated by the greater mobility afforded to cranes with the development of the diesel engine, efficient gear boxes and more recently by the introduction of telescopic booms – all aided by the crane rental market, which has allowed higher utilisation of specialist equipment than could otherwise have been achieved by individual construction companies owning their own plant.

Self-propelled cranes divide roughly into two classifications: the strut-boom type, but with the crawler tracks replaced by rubber-tyred wheels, and the mobile telescopic-boom vehicles. Whereas the mobile version is capable of travelling at 30–40 km/h on public highways, the 'converted crawler' crane achieves perhaps 8–10 km/h. The market trend seems to be moving away from the latter and may in the near future cease to be manufactured.

STRUT-BOOM CRANE

The crane is built in three sections similar to the crawler crane and comprises the base frame, superstructure and boom, the whole unit being powered by a diesel engine.

The base frame consists of a welded steel chassis and power is transferred to the wheels via a king-post gear passing through the turntable to a differential gearbox on the drive axle. The other axle is used for steering. The chassis usually has two-wheel drive, but for use on bad ground some machines are available with four-wheel drive.

The boom and winching arrangements are similar to the crawler crane, with all-mechanical transmission on the older versions and hydraulic motors on newer models. The machine may also be used in the grabbing mode, with suitable rearrangement of the drum sizes and ropes.

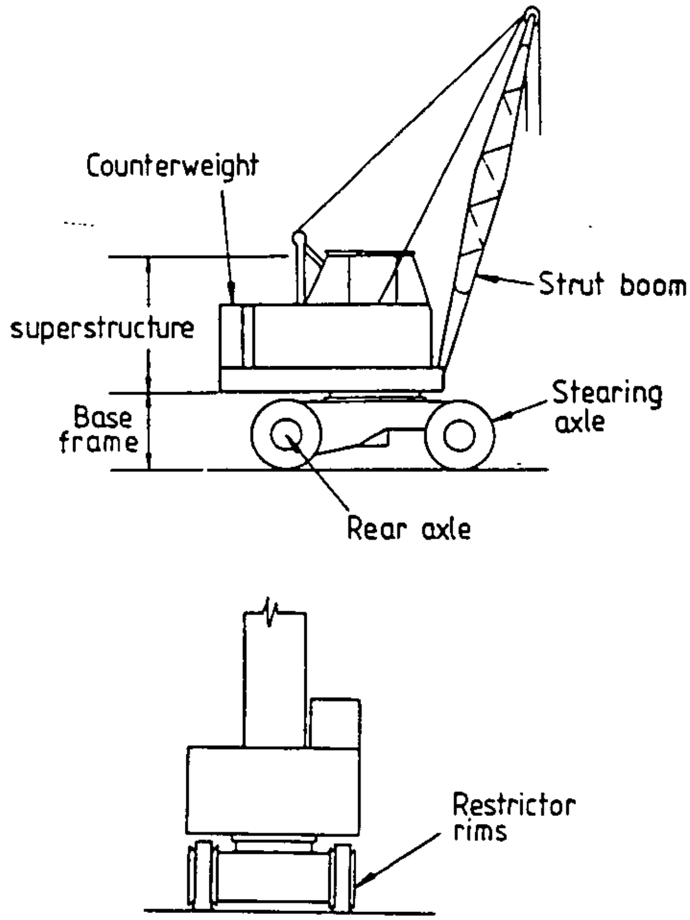


Figure 15.1 Self-propelled strut-boom crane

USES AND OPERATION

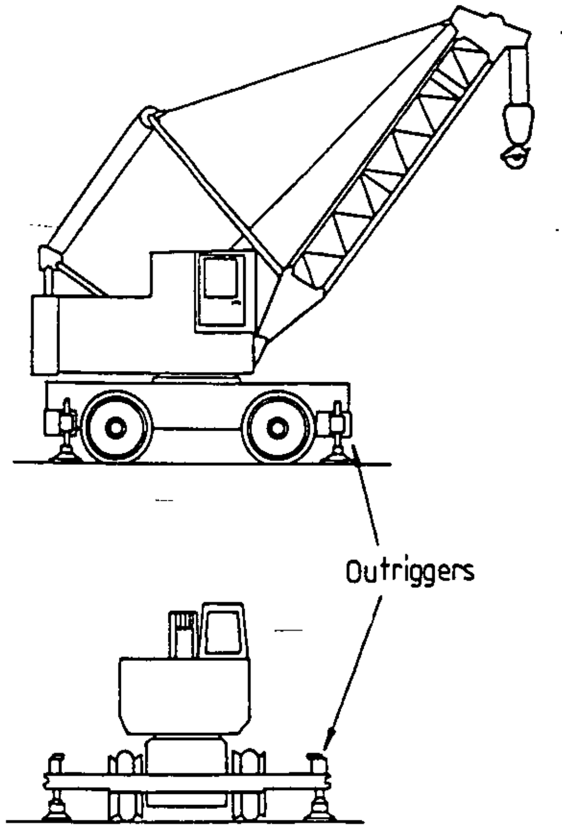
The crane is operated on fairly hard ground such as temporary hardcore roads in stockyards, in scrapyards (where it is usually fitted with a magnet attachment), and for lifting and transporting relatively light loads over short distances of a few tens of metres.

The load is transferred to the ground directly through the large rubber tyres, but some cranes' wheels are made with restrictor rims. There are two large-diameter heavy gauge steel discs mounted one each side of the rubber-tyred wheels. In effect they act as tyre stabilisers by restricting the flexing of the tyre and so dampen down the tendency of the machine to bounce when lifting and transporting. The restrictors also have a lipped rim which considerably increases the bearing area under load, particularly when operating from hard surfaces such as concrete.

OUTRIGGERS

To increase the operating range of the crane, outriggers are incorporated into the base frame on the larger versions, as shown in fig. 15.2. They are housed in heavy steel compartments and are extended and retracted by a winding mechanism. The effective width of the supporting base is thereby extended and any differential level of the ground can be taken out by adjustment of the pads attached to the extremities. The whole unit, including wheels, is

Figure 15.2
Outriggers for improved stability



raised clear of the ground when lifting, all the load being transferred through to the outriggers. This arrangement is called the *blocked* position. It is essential that the ground is well prepared and a solid foundation is available particularly wherever heavy lifts are undertaken. The crane is of course stationary in this configuration and is only used in this way for maximum lifting.

CRANE CAPACITY

The crane size is defined by the maximum load at the minimum operating radius. Lifting capacities *free* on the wheels, i.e. not *blocked*, range from small cranes of 5 tonnes up to 15 tonnes. Typical capacity–boom–radius data are shown in table 15.1 for free-on-wheels and blocked-on-outriggers arrangements. It is apparent from these data that the load–radius diagram for wheeled machines is similar in shape to that of crawler cranes (fig. 14.7), but suitable only for light lifting duties compared to the range of crawler cranes.

CRANE CHARACTERISTICS

Hoisting speed (single fall line)	40–60 m/min
Derricking (max. to min. radius)	20–40 s
Slewing speed	3 rev/min
Travelling speed (unladen)	10 km/h
Max. gradients: max. load	1 in 16
no load	1 in 8

Crane capacity (free-on-wheels) (tonnes)	5	10	15
Engine power, kW (hp)	30 (40)	37 (50)	52 (70)
Machine self-weight (tonnes)	14	18	28

CANTILEVER-BOOM CRANE

COMPARISON OF STRUT- AND CANTILEVER-BOOMS

A strut-boom offers the advantage of high lifting capacity when placing heavy loads at a wide radius, but being pin-jointed to the crane fairly close to the ground restricts the ability to lift and travel with wide loads when height restrictions are in force, for example, stockyard work where access into buildings is required. A cantilever-boom, however, is pivoted at a much higher position on the superstructure and so provides greater clearance, thereby facilitating the handling of bulky loads as shown in fig. 15.4.

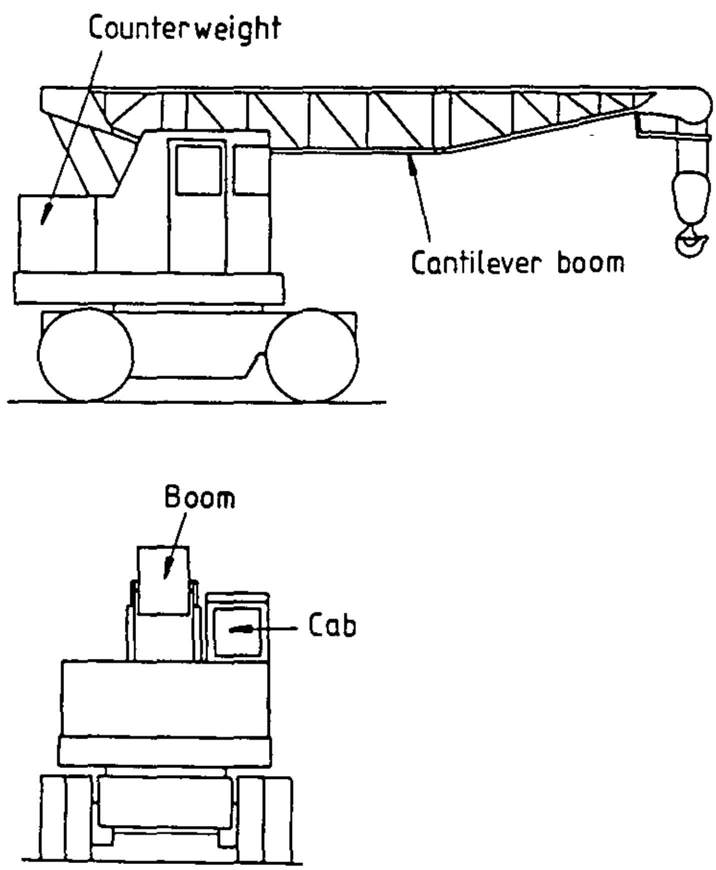


Figure 15.3 Self-propelled cantilever-boom crane

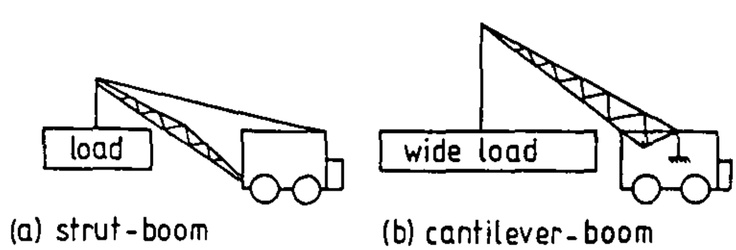


Figure 15.4 Comparison between strut- and cantilever-boom cranes

DESIGN PRINCIPLES OF THE CANTILEVER-BOOM

The forces in a strut-boom are compressive, the tension component of the load being transferred to the superstructure through the derricking ropes. In contrast a cantilever-boom has a wide base tapering to the sheave support and is pin-jointed to the superstructure at the underside its lower end and restrained by the derricking rope running over a sheave along the top side as shown in fig. 15.5. The top member lattice frame is thus in tension and the bottom member in compression.

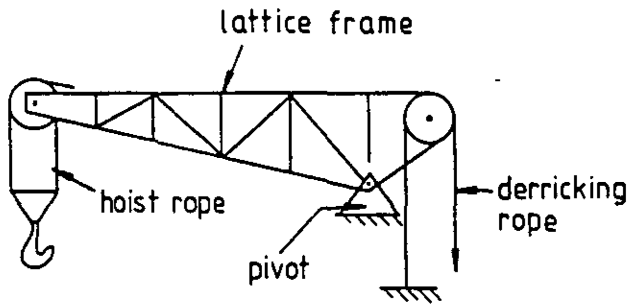


Figure 15.5 Cantilever-boom principles

CRANE CHARACTERISTICS

The choice of cantilever crane is limited within the range 5–15 tonnes free-on-wheels capacity.

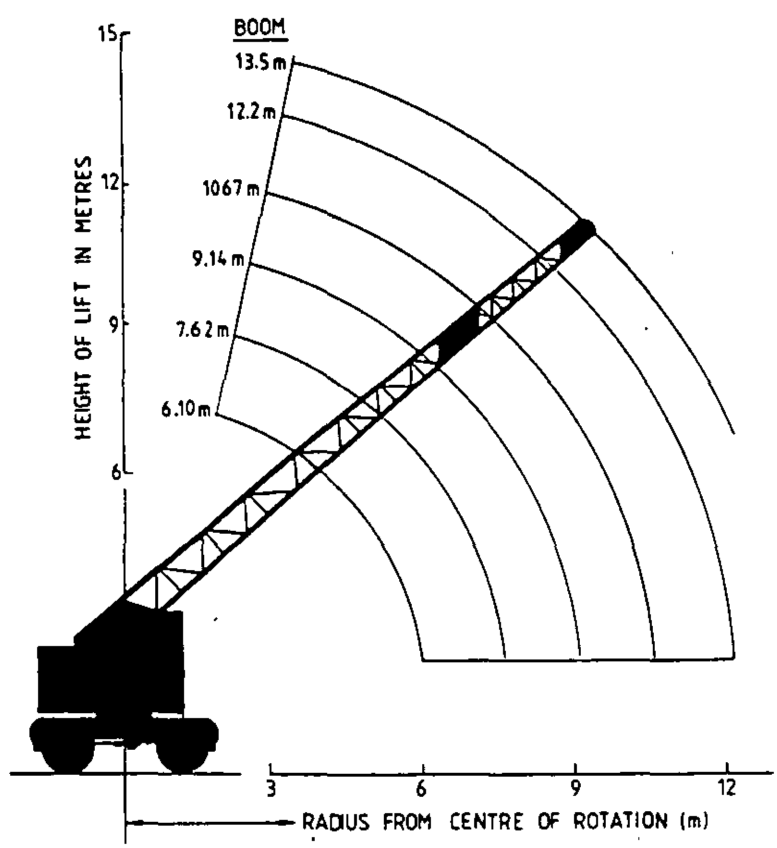


Figure 15.6 Boom radius–height diagram for the cantilever-boom crane

Six Tonnes Capacity Crane (example only)

Max. safe working load	free-on-wheels	6 tonnes as 3.0 m radius on 9.14 m jib
Max. radius and safe working load		850 kg at 13 m radius on 15.2 m jib
On outriggers		Not available
Max. hoisting speed (single fall line)		40 m/min
Slewing speed		3 rev/min
Derricking (max. to min. radius)		25 s
Travelling speed (unladen)		7 km/h
Weight of crane		16 tonnes
Engine horse power		48 kW (66 hp)

Eleven Tonnes Capacity Crane

The lifting characteristics are given in table 15.1. The weight of the crane is about 20 tonnes and is designed only for lifting *free*-on-wheels.

Table 15.1 Guidelines to safe working loads of strut-boom cranes on rubber-tyred wheels

(a) 7 Tonnes Crane FREE ONLY

	Boom (m)				
Radius (m)	*7.6 FREE*	*10.7 FREE*	*13.7 FREE*	*16.8 FREE*	*19.8 FREE*
3.0	6.0				
3.5	5.3	5.0			
4.0	4.2	4.0	3.5		
5.0	3.2	2.8	2.7	2.3	
6.0	2.5	2.1	2.1	1.8	1.7
7.0	1.7	1.6	1.6	1.5	1.3
8.0	1.4	1.4	1.4	1.2	1.1
9.0		1.1	1.1	1.0	0.9
10.0		1.0	1.0	0.9	0.7
11.5			0.9	0.8	0.6
13.0			0.7	0.6	0.5
14.5				0.4	0.3

(b) 11 Tonnes Crane FREE ONLY

	Boom (m)		
Radius (m)	*6.1 FREE*	*9.1 FREE*	*12.2 FREE*
3.0	11.0	11.0	
3.5	9.2	9.0	
4.0	7.5	7.4	6.2
5.0	5.5	5.4	5.2
6.0	4.3	4.2	4.1
7.0		3.4	3.3
8.0		2.9	2.8
9.0		2.5	2.4
10.0			2.0
11.0			1.6

Cantilever crane.

(c) 12.5 Tonnes Crane

	Boom (m)				
Radius (m)	*9.1 BLKD*	*12.2 BLKD*	*15.2 BLKD*	*18.3 BLKD*	*21.3 BLKD*
3.0	12.5				
3.5	10.6	10.2			
4.0	9.4	8.9	8.5		
5.0	7.4	6.8	6.6	6.0	
6.0	5.9	5.0	4.7	4.5	4.3
7.0	4.3	4.0	3.8	3.6	3.5

(d) 12.5 Tonnes Crane

	Boom (m)			
Radius (m)	*9.1 FREE*	*12.2 FREE*	*15.2 FREE*	*18.3 FREE*
3.0	7.0			
3.5	6.3	5.8		
4.0	5.4	5.2	5.1	
5.0	4.4	4.0	3.9	3.8
6.0	3.3	3.1	3.0	2.9
7.0	2.7	2.6	2.5	2.3

Table 15.1 Guidelines to safe working loads of strut-boom cranes on rubber-tyred wheels (contd)

(c) 12.5 Tonnes Crane

	Boom (m)				
Radius (m)	*9.1 BLKD*	*12.2 BLKD*	*15.2 BLKD*	*18.3 BLKD*	*21.3 BLKD*
8.0	3.5	3.3	3.1	3.0	2.9
9.0	3.0	2.9	2.7	2.6	2.5
10.0		2.5	2.4	2.3	2.1
11.5			1.9	1.8	1.7
13.0			1.7	1.6	1.4
14.5					1.1

(d) 12.5 Tonnes Crane

	Boom (m)			
Radius (m)	*9.1 FREE*	*12.2 FREE*	*15.2 FREE*	*18.3 FREE*
8.0	2.2	2.1	2.0	1.9
9.0	1.8	1.7	1.7	1.6
10.0		1.5	1.4	1.3
11.5			1.2	1.1
13.0			1.0	0.9
14.5				0.7

(e) 15 Tonnes Crane

	Boom (m)						
Radius (m)	*9.1 BLKD*	*12.2 BLKD*	*15.2 BLKD*	*18.3 BLKD*	*21.3 BLKD*	*24.4 BLKD*	*27.4 BLKD*
3.0	15.0						
3.5	13.2	12.6					
4.0	12.0	11.6	11.6				
5.0	9.6	9.0	9.0	8.5	8.3		
6.0	7.3	7.2	7.2	7.1	7.1	7.2	
7.0	6.0	6.0	6.0	5.9	5.9	5.9	
8.0	4.9	4.9	4.9	4.8	4.8	4.8	
9.0	4.1	4.0	4.0	3.9	3.9	3.8	3.6
10.0		3.5	3.5	3.4	3.4	3.3	3.2
11.5		2.9	2.9	2.8	2.8	2.7	2.6
13.0			2.6	2.5	2.5	2.4	2.3
14.5				2.0	2.0	1.9	1.8
16.0				1.6	1.6	1.6	1.5
17.5					1.4	1.3	1.2
19.0						1.1	1.1
20.5							0.9

(f) 15 Tonnes Crane

	(Boom (m)					
Radius (m)	*9.1 FREE*	*12.2 FREE*	*15.2 FREE*	*18.3 FREE*	*21.3 FREE*	*24.4 FREE*
3.0	7.0					
3.5	6.3	6.1				
4.0	5.6	5.4	5.3			
5.0	4.4	4.3	4.2	4.1		
6.0	3.5	3.4	3.3	3.2	3.0	
7.0	2.8	2.7	2.6	2.5	2.4	2.3
8.0	2.3	2.2	2.1	2.0	1.9	1.8
9.0	2.0	1.9	1.8	1.7	1.6	1.5
10.0		1.7	1.6	1.5	1.4	1.3
11.5		1.4	1.3	1.2	1.1	1.0
13.0			1.1	1.0	0.9	0.8
14.5			0.9	0.8	0.7	0.6
16.0						0.4

(g) 32 Tonnes Crane

	Boom (m)						
Radius (m)	7.6 BLKD	12.2 BLKD	15.2 BLKD	19.8 BLKD	22.9 BLKD	27.4 BLKD	30.5 BLKD
3.0	32.0						
3.5	31.0						
4.0	28.0	26.0					
5.0	23.0	22.0	21.0	18.0			
6.0	19.0	18.0	18.0	17.0			
7.0	15.0	15.0	15.0	14.0	13.5		
8.0		12.0	11.5	11.3	11.2	10.0	8.8
10.0		8.0	8.0	7.8	7.8	7.7	7.4
12.0			5.8	5.8	5.8	5.8	5.7
14.0			4.8	4.7	4.6	4.5	4.5
16.0				3.9	3.8	3.7	3.7
18.0				3.3	3.2	3.2	3.1
20.0					2.8	2.7	2.7
24.0						2.0	1.9
28.0							1.5

(h) 32 Tonnes Crane FREE ONLY

	Boom (m)						
Radius (m)	7.6 FREE	12.2 FREE	15.2 FREE	19.8 FREE	22.9 FREE	27.4 FREE	30.5 FREE
3.0	15.5						
3.5	13.8						
4.0	12.3	12.1					
5.0	9.8	9.7	9.7	9.7			
6.0	7.9	7.9	7.8	7.7			
7.0	6.5	6.4	6.3	6.3	6.2	5.5	
8.0		5.3	5.3	5.2	5.2	5.1	5.0
10.0		4.0	3.9	3.9	3.8	3.7	3.7
12.0		3.2	3.1	3.0	3.0	2.9	2.8
14.0			2.5	2.4	2.4	2.3	2.2
16.0				2.0	1.9	1.8	1.8
18.0				1.7	1.6	1.5	1.4
20.0					1.3	1.2	1.2
24.0						0.9	0.8

Chapter 16 MOBILE TELESCOPIC-BOOM CRANE

Since the early 1960s telescopic booms have been steadily increasing their share of the crane market at the expense of the self-propelled strut-boom model which has hitherto been very economical and efficient for recurrent usage on specific tasks, e.g. stockyard work. The mobile telescopic version is suitable for similar tasks but is more versatile and is capable of travelling at 20–30 km/h quickly moving from site to site in a particular area. The telescoping action is very flexible and various one-off lifts can be easily accommodated.

Unfortunately, the crane is expensive and as yet has not entirely replaced the self-propelled type for the duties required on the small-to-medium size site. In the long term, however, it must be considered as a serious competitor, particularly the specific versions developed to cope with rough terrain work, when the duties now fulfilled by the crawler crane, such as placing concrete, etc., may also be threatened.

CRANE CONSTRUCTION

CHASSIS

The chassis consists of two welded steel rectangular hollow side beams connected at each end by similar hollow boxes. A diesel engine is mounted at the rear of the vehicle and drives a rigidly mounted rear axle through a torque converter and power-shift gear box (see chapter 2, Fixed-position Excavating Machines, for details).

On some vehicles the rear as well as the front wheels have independent steering as shown in fig. 16.1. Steering on the latest models is controlled by hydraulic cylinders attached between the chassis and wheel hubs.

Two- or four-wheel drive actions are available, four-wheel drive is virtually standard on models made for rough terrain duties on construction sites. The superstructure comprises the drive cab and controls, telescopic boom, hoist and counterweight. The whole unit sits on a turntable, mounted on the chassis and thus 360° slewing is available. An independent hydraulic motor is used to induce the slewing motion. The hydraulic pressure is produced from a pump located near the engine on the base frame and thus a special rotary coupling located at the centre of the slewing ring is required to deliver hydraulic fluid to both the hoist and slewing motors.

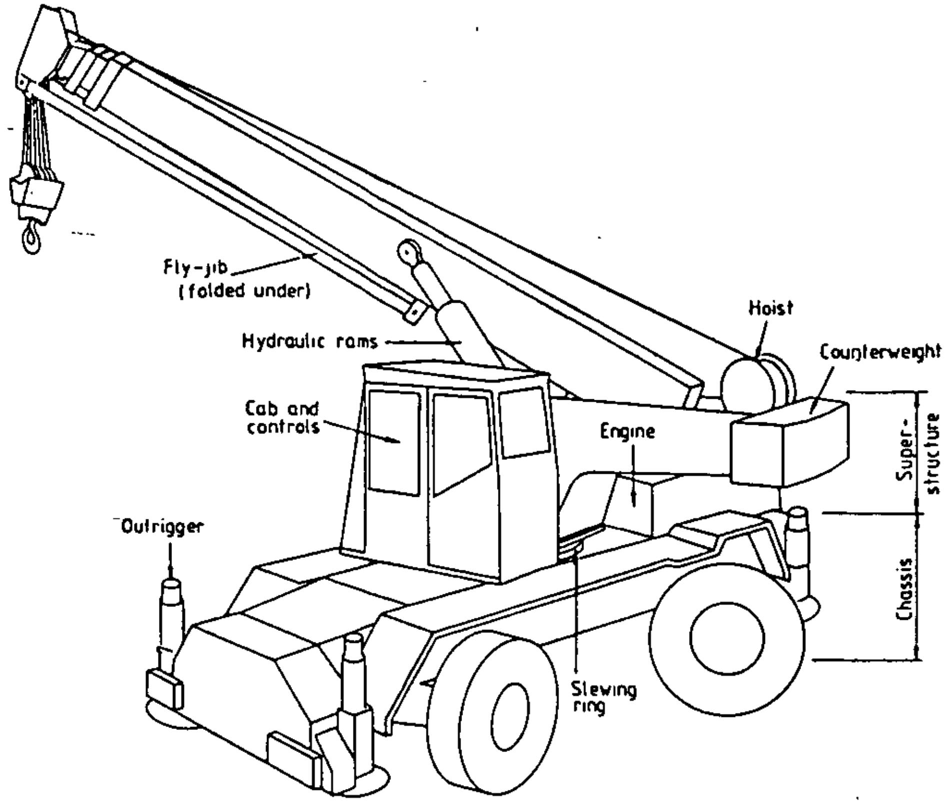


Figure 16.1 Self-propelled telescopic-boom crane

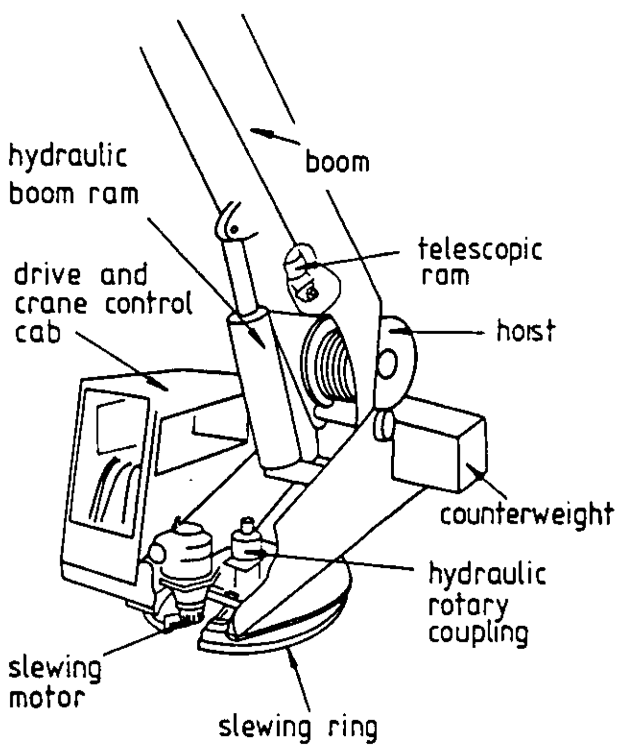


Figure 16.2 Arrangement of superstructure

THE BOOM

The boom is designed on the cantilever principle shown in fig. 16.3. Whereas the self-weight of a strut-type boom is carried by the derricking ropes, the cantilever version is designed to support its own weight in addition to the

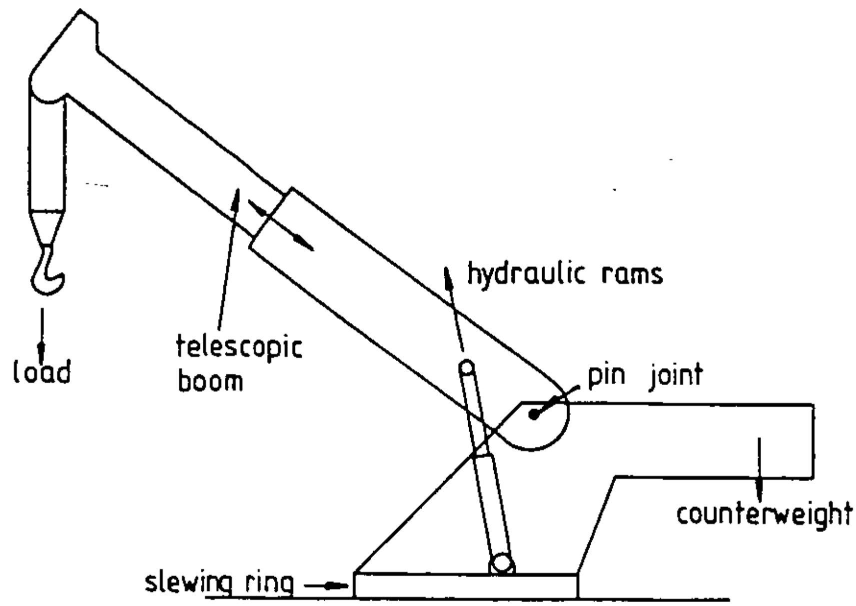


Figure 16.3 Principles of the cantilever–telescopic boom

load, the overturning moment being balanced by the counterweight with a suitable factor of safety. The boom is pin-jointed at its base to the superstructure and derricking is controlled by one or two hydraulic rams.

The boom itself consists of three or four sections, one sliding within the other. The telescoping action is provided by a hydraulic ram and a multiplying chain arrangement, as shown in fig. 16.4.

This configuration causes each section to be telescoped simultaneously and so ensures that the boom is maintained in a tapered shape to match the bending moment caused by the load.

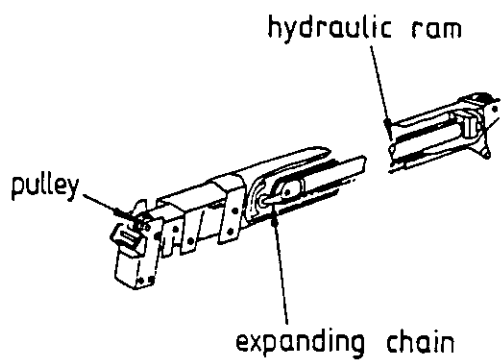


Figure 16.4 Section through a telescopic boom

Boom Design

The restrictions on the design of the telescopic boom are those imposed by the self-weight and weight of the hydraulic telescoping rams which reduce the payload compared to the strut-jib. However, developments in metal strengths are gradually permitting the use of larger booms, and so increasing the operating range of the crane. Most manufacturers adopt a trapezoidal boom cross section made from high tensile alloy steel plate and sometimes the final jib section is of the lattice construction to reduce the weight of the boom.

HOIST OPERATION

The hoist is operated by a hydraulic motor driven from the main hydraulic pump. The hoisting and lowering speed may be varied for precision control of the load.

STEERING

Operation of the crane is carried out from a single cab mounted on the superstructure, The steering may be arranged to suit the duties required as follows:

(a) As a general purpose mobile unit the crane is either two-wheel drive from the rear axle (fig. 16.5a) or alternatively four-wheel drive for improved traction (fig. 16.5b) and is steered by the front wheels. Maximum travelling speed is about 50 km/h.
(b) To improve manoeuvrability, particularly for factory use such as stacking and warehousing duties, some versions of the crane have four-wheel drive and four-wheel steer (fig. 16.5d) which enables crab steering for diagonal movements (fig. 16.5c). Maximum travelling speed is 30–35 km/h.
(c) For heavy duty work, models equipped with the facilities described under (b) have a strengthened chassis and are provided with large diameter tyres for improved grip and flotation, and have increased ground clearance. These are called 'rough terrain' cranes and range up to about 150 tonnes maximum capacity. They are specifically designed to cope with the bad ground conditions found on construction sites. The tendency has been to try to use these cranes as genuine self-travelling cranes on the public

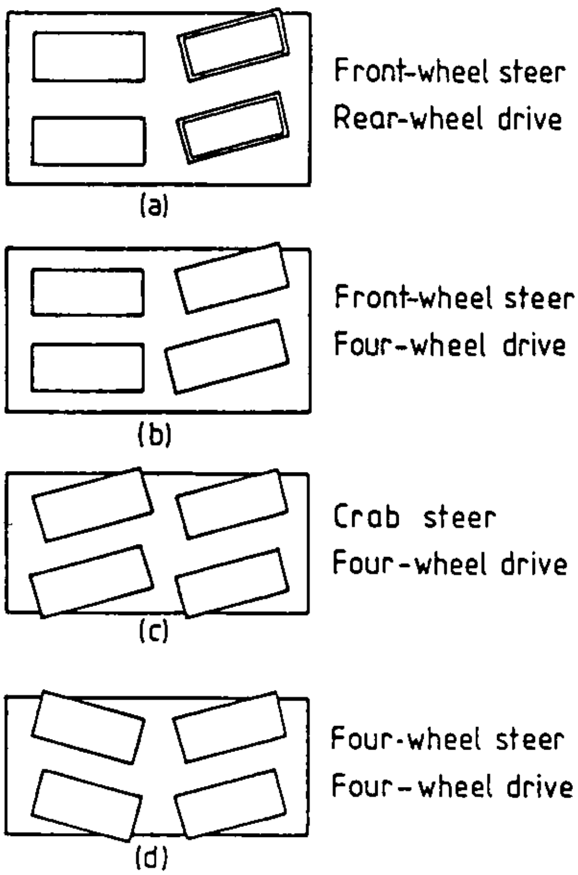


Figure 16.5 Steering alternatives

highways to increase the utilisation factor. But because they are not genuine on–off highway cranes, and due to the heavy maintenance requirements, they have not yet proved popular enough to supplant th crawler crane for difficult site work or the all-terrain model for on–of highway duties. Maximum travelling speed is less than 30 km/h.

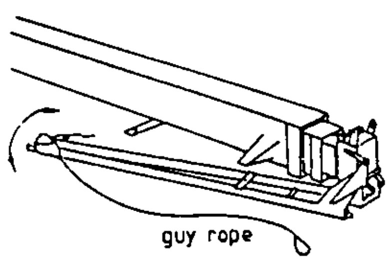
OUTRIGGERS

The operating configuration may be unblocked on the tyres, but the lifting capacity is greatly improved when used with outriggers. Those shown in fig. 16.1 can be hydraulically operated and independently controlled from the driver's cab. They can be quickly set to provide a wide and stable base.

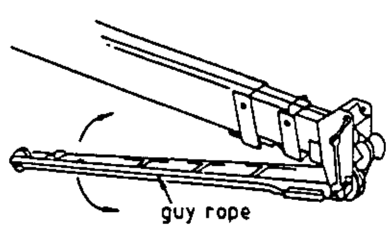
FLY-JIB

The main boom may be provided with the optional facility of a fly-jib which when not in use is stored on the side or underneath the boom. For lifting duties the jib is swung into position by means of the hoist rope and the guy ropes are attached. It may be used in line with the main boom or offset up to about 25°. A fly-jib option is usually available only on the large lifting capacity cranes of 10 tonnes and more – lengths up to about 8 m are available. On the larger rough terrain versions of the crane the fly-jib may also be telescoped (fig. 16.7) to improve the crane's reach for speedy applications.

Figure 16.6 'Quick assembly' fly-jib types



Side folding fly-jib



Under folding fly-jib

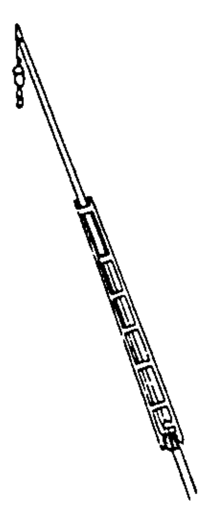
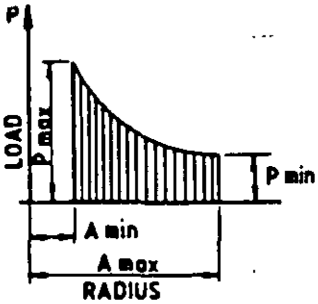


Figure 16.7 Telescopic fly-jib

CRANE CHARACTERISTICS

The smaller versions (up to 10–12 tonnes capacity) tend to be called 'mobile cranes', selected for stockyard work, whilst the larger and more robust machines are likely to be labelled 'rough terrain' cranes, being more appro-

Figure 16.8
Load–radius diagram for telescopic-boom crane



priate for site work. The shape of the load–radius diagram for these cranes is shown in fig. 16.8 and the load–radius data for individual cranes are given in table 17.1.

EXAMPLES

Four to Ten Tonnes Capacity Cranes Operating at Minimum Radius (i.e Small Cranes)

Min. lifting radius	2.5 m
Engine size	100 hp (74 kW)
Machine weight	12–15 tonnes
Max. hoisting speed (single fall line)	10 m/min on older models, 50–60 m/min on the latest models
Derricking (max. to min.)	10 s
Slewing speed	2 rev/min
Travelling speed (unladen)	30 km/h
Turning radius	6 m
Road gradient: loaded	1 in 6
no load	1 in 4
Min. boom length	4–5 m
Max. boom length	6–8 m
Overall height	approx. 2.75 m
Overall width	approx. 2.5 m
Overall length	approx. 6–7 m

The lifting capacity at maximum radius with the boom fully extended operating at an angle of 30° with the horizontal is about 1 tonnes.

Popular Rough Terrain Cranes

Safe working load at minimum radius on outriggers	15–40 tonnes
Engine size	150–200 hp (112–150 kW)
Machine weight	20–40 tonnes
Max. hoisting speed (single fall line)	up to 120 m/min
Derricking (max. to min.)	25 s
Slewing speed	3 rev/min
Travelling speed	30 km/h
Telescoping: three-part jib	20 s
four-part jib	40 s
Turning radius	10 m
Road gradient: unladen	1 in 25
Boom length: four-part	20 m
three-part	14 m
Overall height	3 m
Overall width	2.5 m
Overall length	8 m

Note: The safe working load in the *blocked* position is more than halved when lifting *free*-on-wheels only.

It may be observed by reference to tables in chapter 14 that telescopic mobile cranes as currently manufactured have a significantly lower lifting range of performance than the comparable size of strut-boom crawler model routinely used in placing concrete, handling reinforcement and bulky formwork panels. Thus when conversion to an excavating machine is considered as just one of the options available with the crawler crane, it is readily apparent that the wheeled crane is not particularly attractive for semi-permanent use on the construction site. Although this does not detract from the advantages of mobility between sites for short-term hire, and the large and more robust rough terrain of about 18 to 30 tonnes capacity suits the needs of a section of the market demand for cranage.

Chapter 17 TRUCK-MOUNTED TELESCOPIC BOOM-CRANE

While self-propelled and mobile cranes are suitable for on-site applications in stockyards, warehousing, dockyards, etc., modern construction sites often require cranes to provide medium-to-heavy lifting capacity over high and wide reaches. For example, placing precast concrete floor decks in high-rise construction. mechanical equipment in power station boiler houses, placing bridge deck beams, etc. Frequently the vehicle is only required for a short period of perhaps hours and fast travel between sites then becomes of paramount importance for economic viability. The obvious solution to the problem was development of the conventional truck or lorry to support a lifting unit. the first model being produced about fifty years ago, since then they have gradually become more efficient and reliable, with improvements made to the diesel engine and transmissions and more recently by the introduction of the telescopic boom.

The great advantage of the truck-mounted telescopic version is that travel at normal lorry speeds on the public highways is achievable and when on site takes only a few minutes to prepare for the lifting operations. Such a facility considerably reduces the hire cost, thus making the crane very competitive indeed.

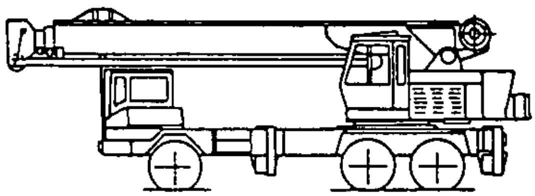


Figure 17.1 Truck-mounted telescopic boom-crane

Cranes with a telescopic boom are available from about 10 to 800 tonnes capacity. However, vehicles less than 120 tonnes or so are increasingly being labelled 'all-terrain', and only the upper end of the range are true truck-mounted cranes. The all-terrain version (fig. 17.2) generally has all-wheel drive. with more robust construction and greater stability better suited to the temporary road surfaces found on typical construction sites. The latest models also have crab steering capabilities and improved speeds for highway travel. The boom on the all-terrain machine is usually shorter than the equivalent truck-mounted version to reduce overhang in the travelling mode, the chassis being much shorter. Finally, all-terrain models of 180 tonnes capacity are

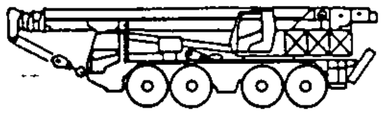


Figure 17.2 All-terrain crane

now appearing on the market and the dividing line between these and truck-mounted units will inevitably be pushed to higher capacities.

Beyond these sizes telescopic-boom technology is currently inadequate and the conventional lattice boom must be used.

CONSTRUCTION OF THE VEHICLE

CHASSIS

The basic carrier comprises a chassis constructed from two universal beams with integral outrigger boxes. The chassis supports the power units, transmission, cab, boom, counterweight and hoists.

The vehicle has the appearance of a conventional truck and is described in terms of the total number of wheels and drive wheels, for example the truck in fig. 17.1 is designated 6 × 2 wheel drive. The number of axles required depends upon the travelling weight of the truck as the legal restrictions specifying the permissible axle load vary depending upon the country, typically 8–12 tonnes.

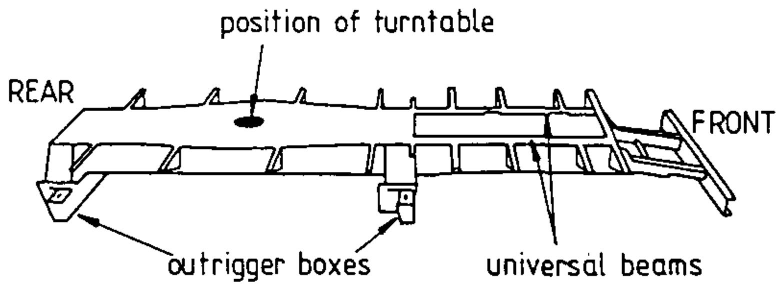
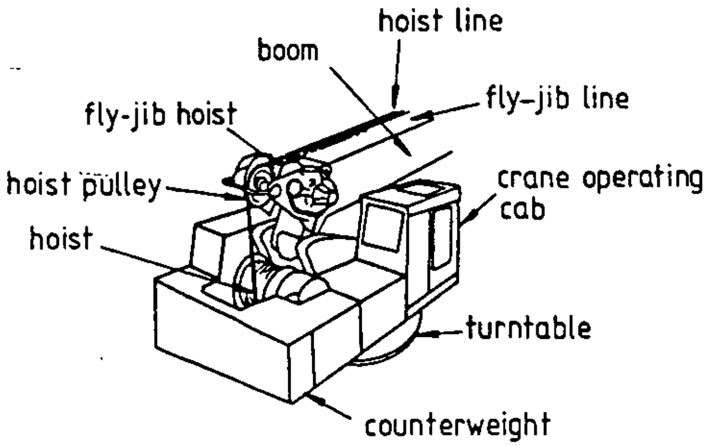


Figure 17.3 Chassis arrangement

SUPERSTRUCTURE

The upper part of the vehicle is similar in construction to mobile telescopic models, except that the counterweight is usually placed at a lower position to improve travelling stability. The whole unit is seated on a turntable to provide 360° slewing. On the smaller vehicles the crane-operating cab and drive cab are combined and a single diesel engine powers all mechanical parts, whereas larger versions have separate cabs and power sources for travelling and lifting duties. In this latter situation the hydraulic pumps and motors are all located on the superstructure and the hydraulic rotary coupling at the centre of the slewing ring is eliminated. The operating controls are also positioned on the superstructure to give the driver a clear view of the load and provide at least 180° of vision without having to look back, as is necessary when operating from the driver's cab.

Figure 17.4
Arrangement of uperstructure



CRANE CHARACTERISTICS

Max. hoisting speed (single fall line)	approx. 70–150 m/min
Slewing speed	3 rev/min
Derricking (max. to min. radius)	up to 1 min
Telescoping: in	10 m/min
out	20 m/min
Travelling speed	up to 70 km/h

TRAVELLING ON THE HIGHWAY

It is essential that the boom is fully retracted and well secured in the horizontal position, as shown in figs. 17.1 and 17.2 to avoid danger of damage to bridge decks.

CRANE CAPACITY

Mobile rough-terrain, all-terrain and truck-mounted telescopic boom cranes are designed in accordance with the following criteria:

(a) All capacities are limited to a proportion of the tipping load.
(b) With a fly-jib attached and extended but not working, the load on the main boom should be reduced, e.g. if length of fly-jib is 6–10 m, working load reduction is about 380 kg.
(c) Boom sections should be extended equally.
(d) The crane may be operated blocked on outriggers or free-on-wheels, with a corresponding reduction in capacity for the latter.
(e) The weight of the hook block, together with any slings or other lifting tackle, must be included in the working loads to arrive at a suitable size of crane.

Load-Radius-Boom Data

The data given in table 17.1 are general and should only be used for approximate crane selection purposes as the capacity of similarly designated cranes varies from make to make.

Figure 17.5 Boom radius–height diagram for truck/all-terrain cranes

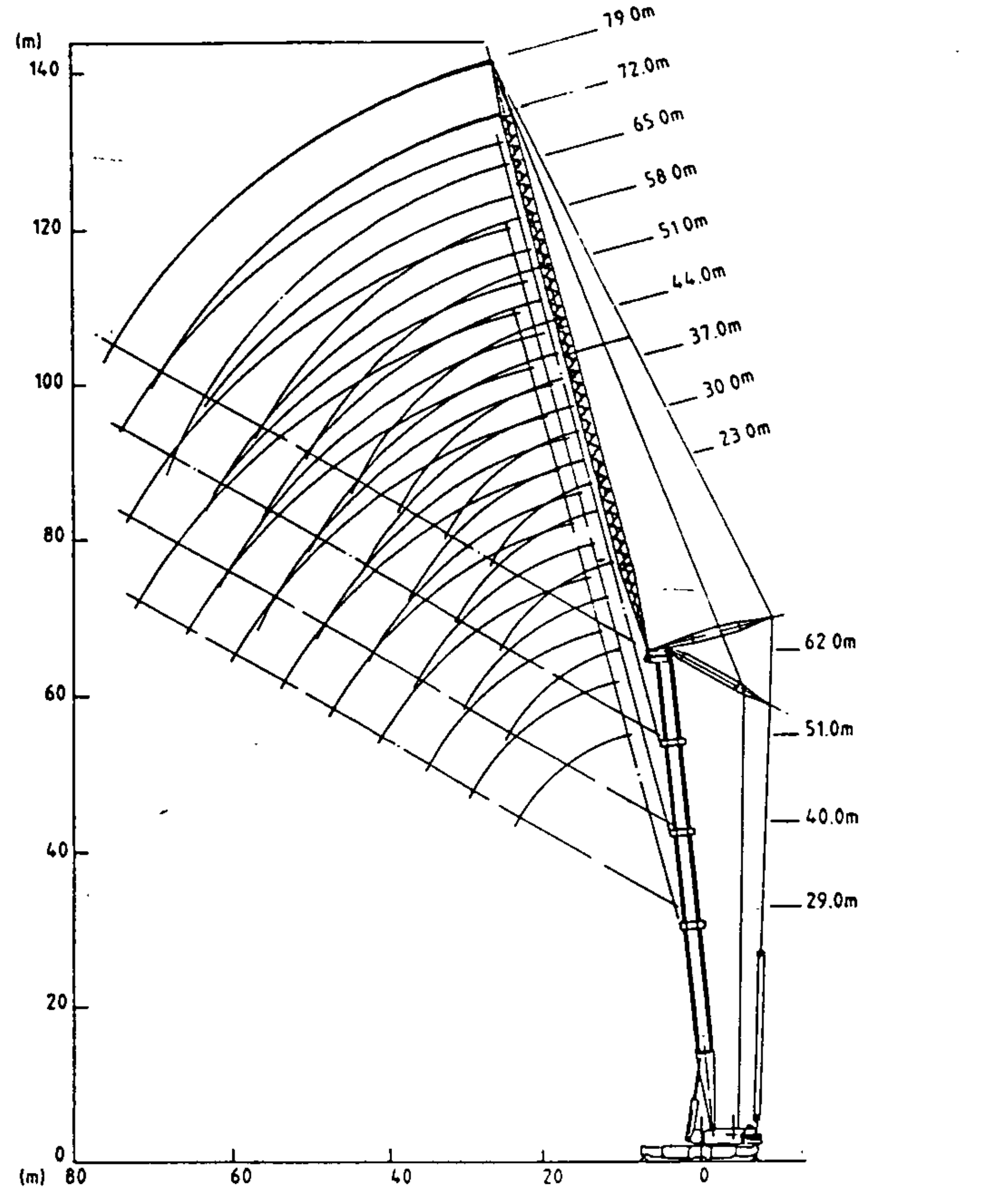


Table 17.1 Guidelines to safe working loads of mobile, rough, all-terrain and truck-mounted telescopic-jib cranes.
(Note: Boom extensions quoted are maximum permitted values at the stated radius.)

(a) 6 Tonnes Crane Mobile					*(b) 12 Tonnes Crane Mobile*				
	Boom m)					*Boom (m)*			
Radius (m)	*6.6 BLKD*	*FREE*	*8.7 BLKD*	*FREE*	*Radius (m)*	*7.92 BLKD*	*FREE*	*18.3 BLKD*	*FREE*
2.5	6.0				3.0	12.0	5.25	10.0	
2.75	5.5	5.25			3.5	10.5	4.5	8.6	
3.0	5.0	4.5	4.85		4.0	9.1	4.0	7.4	4.0
3.5	4.4	3.75	4.25	3.75	5.0	6.5	2.7	5.5	2.7
4.0	3.9	3.2	3.75	3.2	6.0	4.0	1.8	4.3	1.8

Table 17.1 Guidelines to safe working loads of mobile, rough, all-terrain and truck-mounted telescopic-jib cranes (contd)

(a) 6 Tonnes Crane Mobile

	Boom (m)			
Radius (m)	*6.6 BLKD*	*FREE*	*8.7 BLKD*	*FREE*
4.5	3.4	2.6	3.25	2.6
5.0	3.0	2.2	2.8	2.2
5.5	2.75	1.9	2.6	1.9
6.0			2.3	1.5
6.5			2.1	1.35
7.0			1.9	1.2
7.5			1.8	1.1

(b) 12 Tonnes Crane Mobile

	Boom (m)			
Radius (m)	*7.92 BLKD*	*FREE*	*18.3 BLKD*	*FREE*
7.0		1.1	3.5	1.1
8.0		0.6	3.0	0.6
9.0			2.4	0.4
10.0			1.9	
11.5			1.5	
13.0			1.1	
14.5			0.75	

(c) 18 Tonnes Crane Rough-Terrain

	Boom (m)						
Radius (m)	*7.3 BLKD*	*9.0 BLKD*	*11.0 BLKD*	*12.2 BLKD*	*14.6 BLKD*	*16.5 BLKD*	*18.3 BLKD*
3.0	18.3	15.6	14.8	14.0			
3.5	14.8	14.8	14.2	13.7	12.9		
4.0	13.4	13.4	13.1	12.9	12.0		
4.5	11.9	11.9	11.9	11.9	11.0	10.1	9.7
6.0	9.1	9.1	9.1	9.1	9.1	8.6	8.0
7.5		5.9	5.9	5.9	5.9	5.8	5.8
9.0			4.2	4.2	4.2	4.2	4.2
10.5				3.3	3.3	3.3	3.3
12.0					2.5	2.5	2.5
13.5						1.8	1.8
15.0						1.6	1.6
16.5							1.3

(d) 22 Tonnes Crane All-Terrain

	Boom (m)							
Radius (m)	*9.75 BLKD*	*FREE*	*13.4 BLKD*	*FREE*	*17.1 BLKD*	*FREE*	*24.4 BLKD*	*FREE*
3.0	22.0	7.0	19.6	6.7				
3.5	18.6	5.8	16.9	5.2				
4.0	16.0	4.7	15.1	4.3				
4.5	13.5	3.5	13.4	3.4	12.0	3.2		
6.0	9.8	2.2	9.5	2.0	8.6	1.8	6.5	1.1
7.5	6.5	1.2	6.5	1.1	6.0	0.9	5.2	0.7
9.0			4.5	0.6	4.5	0.5	4.0	0.3
12.0			2.5		2.5		2.3	
15.0					1.6		1.4	
18.0							0.9	
21.0							0.7	

(e) 30 Tonnes Crane Rough-Terrain

Radius (m)	*Boom (m)* 9.75 BLKD	13.25 BLKD	16.76 BLKD	20.37 BLKD	22.73 BLKD	30.48 BLKD
3.0	30.0	17.5	16.4			
3.5	26.0	17.5	16.4			
4.5	20.0	17.5	15.0	12.0		
6.0	16.0	14.5	13.0	10.5	8.5	
7.5	14.0	12.5	11.5	9.2	7.2	5.0
9.0		10.3	10.0	8.2	6.2	4.5
12.0			5.5	5.6	4.7	3.5
15.0				3.5	3.6	2.7
18.0				2.2	2.3	2.3
21.0					1.3	2.0
24.0						1.5
27.0						0.9
30.0						

(f) 40 Tonnes Crane All-Terrain

Radius (m)	*Boom (m)* 9.75 BLKD	11.6 BLKD	13.4 BLKD	15.2 BLKD	18.9 BLKD	23.8 BLKD	30.8* BLKD
3.5	41.0	35.0	30.0	27.0			2.5
4.5	32.4	28.8	26.4	23.9			1.7
6.0	24.0	24.0	22.8	20.8	16.8		
7.5	15.4	15.4	15.4	15.4	14.5	11.2	7.8
9.0		10.6	10.6	10.6	10.6	10.0	6.3
12.0				5.8	5.8	5.8	4.3
15.0					3.8	3.8	3.2
18.0						2.5	2.1
21.0						1.7	1.6
24.0							1.1
27.0							0.7
30.0							

(g) 60 Tonnes Crane All-Terrain

Radius (m)	*Boom (m)* 10.4 BLKD	13.4 BLKD	17.1 BLKD	20.1 BLKD	23.5 BLKD	27.1 BLKD	29.9 BLKD	36.2* BLKD
3.0	59.0	49.0						
3.5	54.0	46.0	39.0	32.0				
4.5	45.0	42.0	35.0	30.0	26.0			
6.0	36.0	36.0	30.0	24.0	22.0	21.0	16.0	7.5
7.5	25.0	26.0	25.0	20.0	18.0	16.0	13.0	7.0
9.0	18.0	18.0	18.0	17.0	15.0	14.0	11.0	6.0
12.0			10.0	10.0	10.0	9.0	7.0	5.0
15.0			6.0	6.0	6.0	6.0	6.0	4.0

Table 17.1 (cont) *(g) 60 Tonnes Crane All-Terrain*

Radius (m)	*Boom (m)* 10.4 BLKD	13.4 BLKD	17.1 BLKD	20.1 BLKD	23.5 BLKD	27.1 BLKD	29.9 BLKD	36.2* BLKD
18.0					4.0	4.0	4.0	3.5
21.0						3.0	3.0	2.8
24.0						2.0	2.0	2.2
27.0							1.2	1.8
30.0								
33.0								

* Manual extension from top section.

(h) 90 Tonnes Crane All-Terrain

Radius (m)	*Boom (m)* 13.4 BLKD	18.2 BLKD	23.2 BLKD	28.0 BLKD	32.9 BLKD	42.7* BLKD	52.4* BLKD
3.0	90.0						
3.5	80.0	53.0	48.0				
4.5	69.0	51.0	44.0				
6.0	52.0	42.0	37.0	33.0	29.0		
7.5	39.0	35.0	31.0	26.0	23.0		
9.0	28.0	28.0	27.0	22.0	19.0	19.0	
12.0		17.0	17.0	16.0	14.0	14.0	10.0
15.0		11.0	11.0	11.0	10.0	10.0	8.0
18.0		7.0	7.0	7.0	7.0	7.5	6.5
21.0			5.0	5.0	5.0	6.0	5.5
24.0				3.0	3.0	4.0	4.5
27.0					2.0	3.0	3.5
30.0						2.5	2.8
33.0						1.6	2.2
36.0						1.0	1.7
39.0						0.5	1.1
42.0							0.6

(i) 112 Tonnes Crane All-Terrain

Radius (m)	*Boom (m)* 13.9 BLKD	21.3 BLKD	28.6 BLKD	36.0 BLKD	43.0 BLKD	52.8* BLKD	62.6* BLKD
3.5	112.0	53.0					
4.5	89.0	52.0					
6.0	67.0	45.0	40.0				
7.5	53.0	40.0	34.0	31.0			
9.0	42.0	35.0	30.0	27.0	24.0		
12.0		26.0	23.0	20.0	18.0	16.0	
15.0		18.0	18.0	16.0	13.0	13.0	9.0
18.0		12.0	12.0	12.0	11.0	10.5	7.5
21.0			8.0	8.0	8.0	8.5	6.5
24.0			6.0	6.0	6.0	6.8	5.7

(i) 112 Tonnes Crane All-Terrain

	Boom (m)						
Radius (m)	13.9 BLKD	21.3 BLKD	28.6 BLKD	36.0 BLKD	43.0 BLKD	52.8* BLKD	62.6* BLKD
27.0				4.5	4.5	5.3	5.1
30.0				3.0	3.0	4.5	4.4
33.0				2.0	2.0	3.3	3.6
36.0					1.0	2.4	2.9
39.0						1.8	2.2
42.0						1.1	1.6
45.0						0.6	1.1

(j) 150 Tonnes Crane Truck-Mounted

	Boom (m)					
Radius (m)	15.24	21.34	27.43	33.53	39.62	45.72
4.0	150.0					
6.0	100.0	98.0				
8.0	73.9	73.3	72.7			
10.0	56.2	55.9	55.700	55.4	52.0	
14.0	33.9	33.7	33.3	33.1	32.9	
20.0		20.4	20.000	19.8	19.5	32.7
24.0			15.6	15.2	15.0	19.3
30.0				11.0	10.6	14.3
35.0					8.4	10.4
40.0						7.9
						6.5

(R) 350 Tonnes Crane Truck-Mounted

	Boom (m)					
Radius (m)	16.75	28.75	40.0	52.0	72*	86*
3	350.0					
4.5	251.0	150.0				
7	163.0	117.0	80.0			
9	125.0	99.0	69.0	45.0		
11	99.0	85.0	60.0	44.5	25.5	
13	79.0	74.0	52.5	41.0	25.4	
18		47.0	39.0	33.0	21.5	9.0
24		28.0	29.0	25.5	17.7	8.4
28			22.0	21.5	15.7	7.6
34			14.5	16.6	13.1	6.4
40				11.8	10.8	5.4
46				8.5	8.6	4.5
54					5.0	3.5
64						2.5

Note: This is the largest crane able to carry its telescopic boom in the road travelling configuration.
* Including fly-jib.

Table 17.1 (cont)
(l) 500 Tonnes Crane Truck-mounted

	Boom (m)						
	18.6	*31.4*	*44.2*	*57.0*	*57.0 Fly-Jib (m)*		
Radius (m)					24	48	72
3	500.0						
4	380.0						
6	288.0	190.0					
8	230.0	189.0	140.0				
10	177.0	158.0	137.5	100.0			
15	115.0	105.0	101.0	80.5			
18		82.0	87.0	71.5	45.0		
22		60.0	65.0	60.5	44.0		
25		49.0	53.0	53.0	41.5		
30			38.5	43.5	39.0	22.0	
36			26.5	31.5		22.0	
40			20.5	25.0		22.0	8.8
46				18.0		21.7	8.4
50				15.0		20.2	8.2
60							7.8
70							7.0

(m) 800 Tonnes Crane Truck-Mounted

	Boom (m)								
	18	*40*	*62*	*29 Fly-Jib (m)*			*62 Fly-Jib (m)*		
Radius (m)				*23*	*51*	*93*	*23*	*51*	*72*
4	800								
6	500								
7	420	300							
8	360	260							
9	320	235	175						
10	290	215	165						
14	200	160	130	120					
18		125	102	117			67		
22		105	82	104	65		64		
26		85	70	92	63		59	37	
30		62	60		59		55	35	
34		48	53		54	16		33	21
40			37		47	14		28	19
[illegible]			23		38	13		26	19
[illegible]			10			11			16
72						9			12
84						8			

EXAMPLE OF TRUCK-MOUNTED TELESCOPIC CRANE SELECTION

Select a truck-mounted crane as an alternative to the crawler crane for liftir– 5 tonnes to the centre of the concrete tank described in the example on p. 112.

Solution

(a) It is seen in fig. 17.6 that to place the load a boom length of at least 18.9 m is required. Table 17.1 for a 40 tonnes capacity crane (which is sufficient in the case of a crawler crane) is slightly undersize. Interpolation between the 12 m and 15 m radii given in the table indicates that at 14 m radius the crane will lift about 4.5 tonnes. The next available size crane in the tables is 60 tonnes capacity. interpolating this will lift approximately 7.3 tonnes at 14 m radius.

(b) If the load were very wide and deep, say, half the diameter of the tank and 5 m deep, then to clear the tank rim whilst slewing would require a 20.1 m boom.

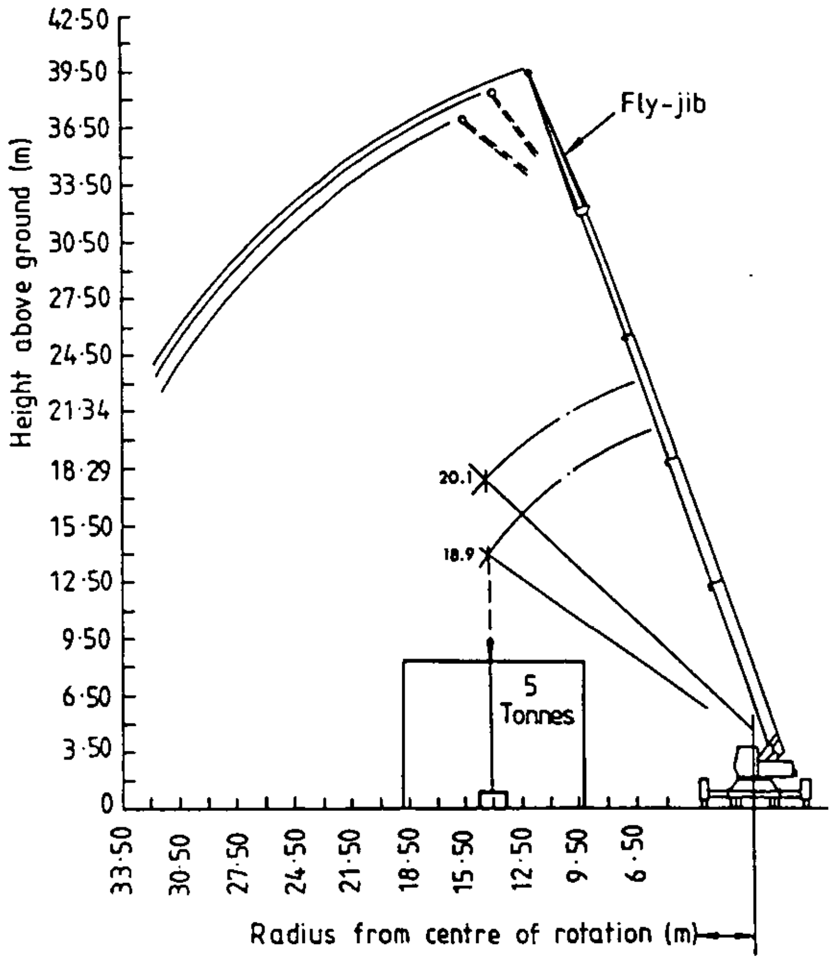


Figure 17.6 Example of truck-mounted crane in use

Chapter 18 TRUCK-MOUNTED STRUT-BOOM CRANE

Before the development of the telescoping action, the strut-boom was the predominant type of truck-mounted crane. Today, however, these cranes are limited to very specialist duties for lifts of 400 tonnes and more and are currently available with capacities up to 1200 tonnes. Designs are in hand up to 2000 tonnes.

CRANE CONSTRUCTION

The carrier consists of a stiff chassis mounted upon two or more axles depending upon the travelling weight. The chassis incorporates a diesel engine, the transmission and outriggers. The engine is used to power the vehicle for transporting only. The lifting section of the crane is a conventional strut-boom crane superstructure without the crawler tracks, which sits on a turntable mounted on the chassis. It comprises the counterweight, hoist and derricking drums, slewing mechanism, boom and operating controls, and an independent diesel engine to power the crane parts which like most other modern machines use independent hydraulic motors to provide precision hoisting and lowering control.

BOOM ASSEMBLY

This type of crane is frequently selected because of its ability to provide very specific heavy lifting capacity, economically over a short period. To do this effectively, the crane should ideally be ready and prepared for use almost immediately upon arrival at the construction site. The strut-boom unfortunately, however, cannot travel in this ready form, and must be folded and securely held. Consequently much setting up time is needed upon arrival, for example 4 hours preparation time would not be untypical. For use with the basic length boom, i.e. top and bottom sections only, the equipment packs down neatly as shown in fig. 18.1. But extensions beyond this basic length require the sections to be laid end-to-end on an area of level ground before erection. The assembly then proceeds as shown in fig. 18.2.

These very large capacity cranes often require two carriers: one for the crane (i.e. drive cab, engine, turntable, counterweights, hoist, etc.) and

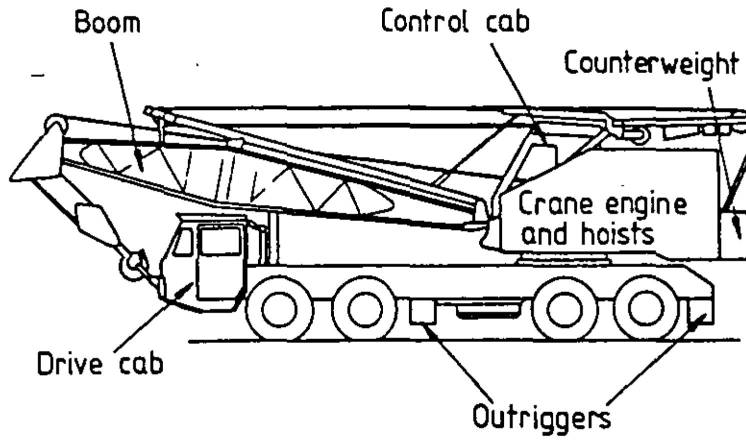


Figure 18.1 Truck-mounted strut-boom crane arranged for travelling on a highway

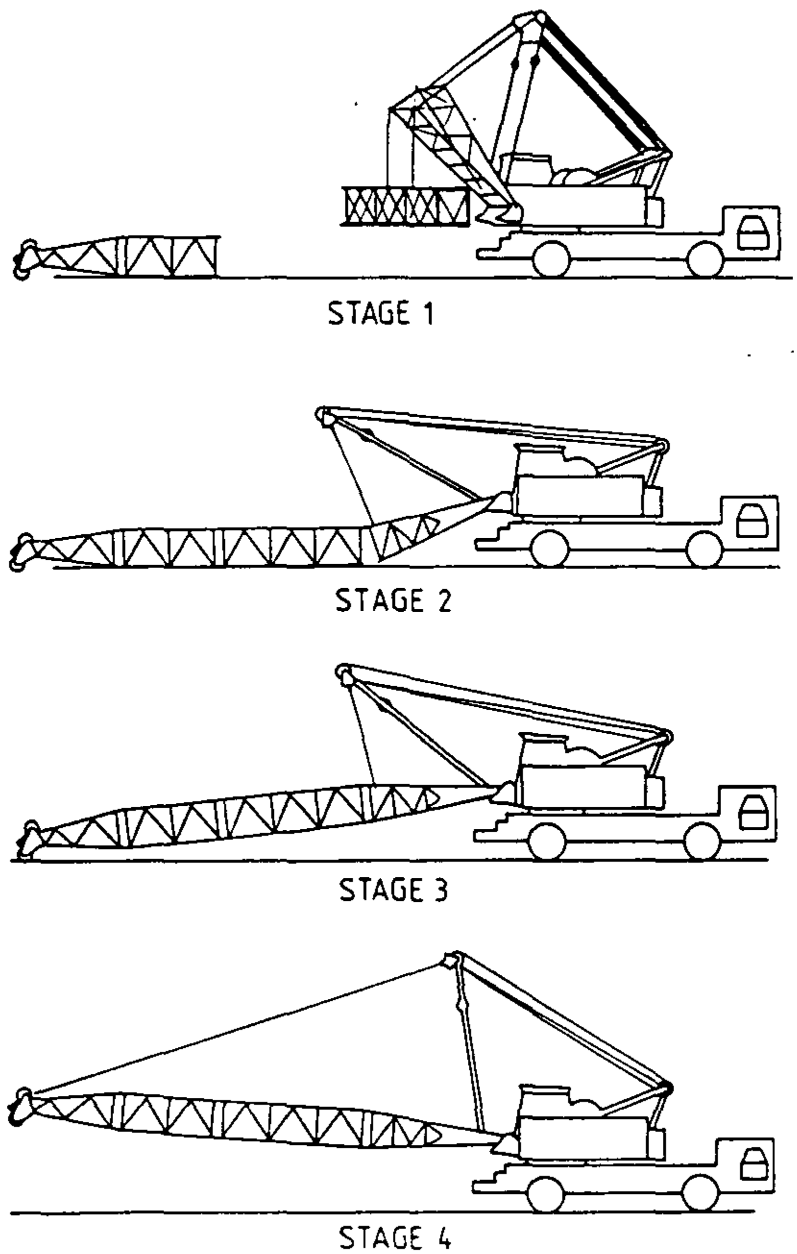


Figure 18.2 Assembly of strut-boom

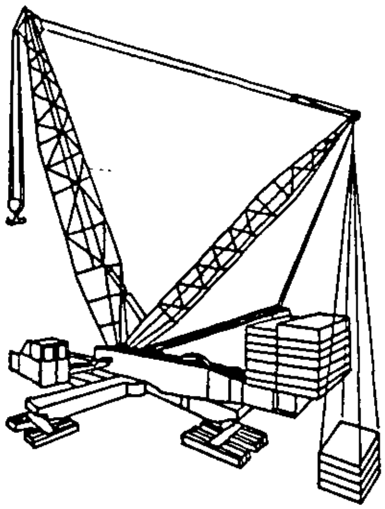


Figure 18.3 Large truck-mounted crane (chassis not shown)

one to transport the boom and fly-jib. The mechanisms for slewing, hoisting and derricking of this size crane are usually driven by electric motors powered from the main diesel engine. Extra outriggers and kentledge are also necessary (fig. 18.3) and the setting up time is much increased. The size of the load to be placed, however, is usually so specified that this can be taken into account in the construction programme.

CRANE CHARACTERISTICS

Travelling speed	up to 75 km/h
Slewing speed: up to 500 tonnes	0.5–1.0 rev/min
500–1000 tonnes	0.1–0.3 rev/min
Derricking (min. to max.)	Varies, e.g. 3 min for 300 tonnes crane with 15 m boom
Max. hoisting speed (single fall line)	60–120 m/min
Optional features	Operation as a tower crane

Load–Radius–Boom Capacity

The weight of the snatchblock, slings and other handling devices are significant and must, of course, be added to the load. The crane may be operated with a fly-jib and can be used either blocked-on-outriggers or free-on-wheels. Table 18.1 show the lifting capacities of 450 and 800 tonnes capacity cranes, respectively.

It will be noticed that these machines have very high lifting ability and are particularly suited for heavy duties, such as in offshore work, boiler installation in power stations, etc. The number of such cranes in the country are few and lifts must be planned well in advance to ensure availability of a crane of suitable capacity.

Table 18.1 Guidelines to safe working loads of truck-mounted strut-boom cranes
(a) Main Boom Rating : 450 Tonnes

	Boom (m)						
Radius (m)	*16 m*	*30 m*	*44 m*	*58 m*	*72 m*	*86 m*	*100 m*
5	450						
7	350						
9	295	292					
12	210	205	200	192			
14	176	174	170	160	125		
18		131	127	124	100	77	54
24		95	92	90	75	59	31
30		75	72	69	61	47	24
40			51	47	44	38	18
56				27	24	20	10
68					16	14	4
80						7	1

(b) Main Boom Rating : 800–1200 Tonnes

	Boom (m)							
Radius (m)	*23 m*	*35 m*	*47 m*	*59 m*	*71 m*	*83 m*	*95 m*	*113 m*
5	800							
7	650							
9	529	526						
10	478	475(1200)	470					
12	401	397(1000)	393	363				
18	268	265(645)	261	258	205	184		
22	216	214(600)	210	207	176	165	139	93
26		178(504)	174	171	152	144	126	85
32		135(410)	131	128	126	116	108	75
40			94	91	89	86	84	62
56				54	51	48	46	41
64					40	37	34	30
76						26	23	18
84							17	12
88								10

Note: Figures in brackets for 35 m boom are with alternative counterweight.

Chapter 19 TOWER CRANES

On travels far and wide the number of tower cranes in use is quite striking, with the skyline of the major cities often cluttered by new high-rise constructions. The tower crane, however, is also very suitable for low-rise work concentrated within a limited area where access by crawler or other mobile cranes is restricted.

The main advantage of the tower crane is that the jib or boom is supported at the top of a tall tower which may be set at a sufficient height to clear any obstructions. This configuration allows the crane to stand very close to, or even in, the structure under construction. In this way a relatively short boom provides more reach in comparison with other types of crane. Thus for high-rise buildings the tower crane is often the cheapest form of device.

There are two versions, the horizontal-boom and the luffing-boom, both powered electrically with 350–415 V supply.

LUFFING-BOOM

This arrangement consists of a lattice-framed vertically standing mast mounted on a sturdy turntable. The boom is hinged near to the top of the

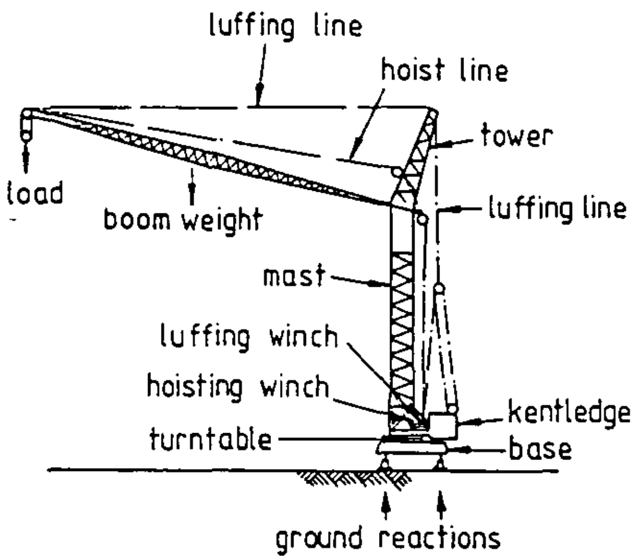


Figure 19.1 Luffing-boom tower crane (turntable at base of mast)

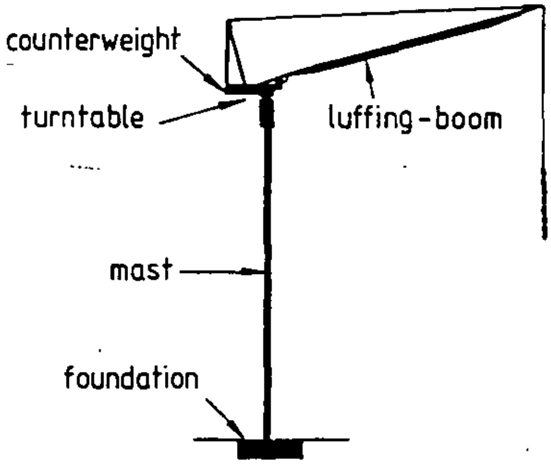


Figure 19.2 Luffing-boom tower crane, alternative kentledge at top of mast

mast and the luffing line, attached to the extremity, is passed over a pulley at the top of the tower then down to the kentledge to counterbalance the weight of the boom. The kentledge is generally located on the turntable but some manufacturers prefer to have both the turntable and kentledge at the top of the mast as shown in fig. 19.2 so that all the moving parts are above the structure under construction. The whole unit is placed on a well-prepared foundation. The crane is electrically powered and electric motors are used to drive the hoist and luffing winches and for slewing. The main advantage of the luffing-boom tower crane is that the boom can be raised clear of nearby obstructions when slewing.

HORIZONTAL-BOOM

The crane comprises a vertical standing lattice-framed central mast, which supports a horizontal-boom in two parts, the larger section being used for lifting and carries a 'trolley' or 'saddle' travelling on guides along the length of the boom (fig. 19.4). Thus the radius is changed by moving the trolley and not by luffing the boom. On the opposite side of the mast a shorter boom supports a kentledge block and serves as a counterbalance. The resistance to overturning when lifting (and from wind pressures) is transferred through the

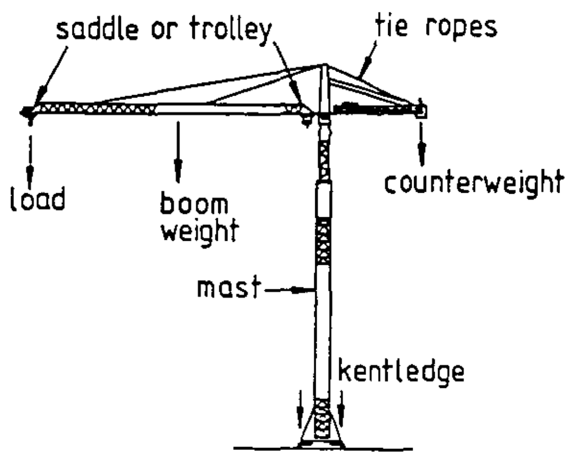
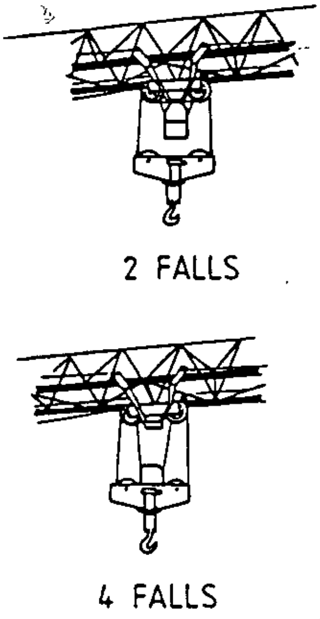


Figure 19.3 Horizontal or saddle-boom tower crane

Figure 19.4 Saddle or trolley for horizontal [illegible] crane



tower to a heavy foundation base. Like luffing-boom models, the tower crane must be designed to resist torsion from side loads acting on the boom, e.g. a swinging load, wind, etc. It is usual for crane operation to be suspended when the wind gust speed exceeds 61 km/h (38 mph) and the boom is left to swing freely on the turntable to reduce the torsional effect.

The crane has 360° slewing capability and the turntable is commonly mounted at the top of the mast. This configuration usually involves transporting the crane in sections with an associated slow assembly on site, but rapid development in self-evecting cranes (fig. 19.5) is taking place. The whole unit is supported on a strong foundation connected to the base of the mast with the main counterweight also located in this position. The centre of gravity is thus brought nearer to the ground to improve the crane's stability.

Electric power provides the drive for the slewing and hoist motors.

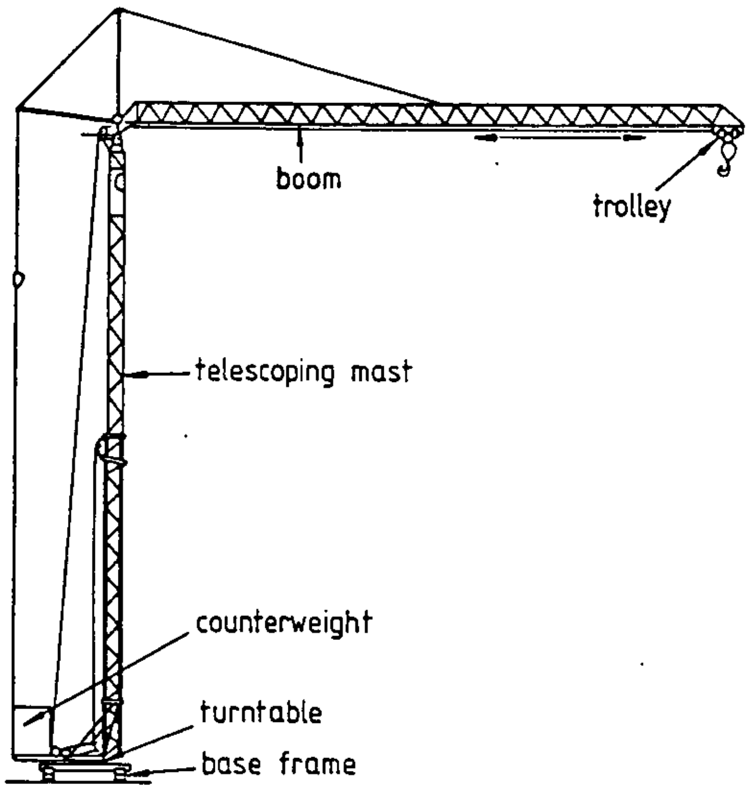


Figure 19.5 Self-erecting saddle-boom tower crane

LOAD-RADIUS DIAGRAM FOR THE LUFFING-BOOM CRANE

The tipping load of the luffing-boom is calculated from the following principles (excluding side loads).
Taking moments about the centre of rotation

$$Wr + J\frac{r}{2} = Cx$$

$$W = \frac{Cx}{r} - \frac{J}{2} \qquad (1)$$

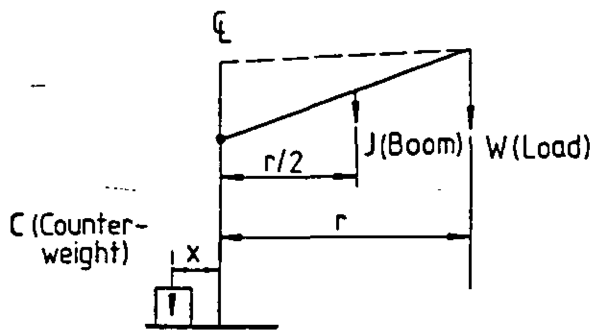


Figure 19.6 Forces acting on a luffing-boom tower crane

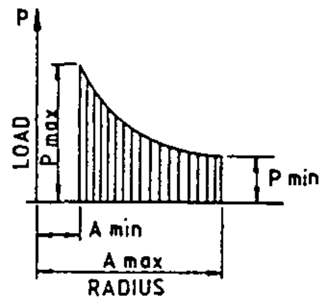


Figure 19.7 Load–radius diagram for the luffing-boom tower crane

Safe working load (P) = W − margin for safety.
Equation (1) if plotted with varied r draws the curve shown in fig. 19.7.

LOAD-RADIUS DIAGRAM FOR THE SADDLE-BOOM CRANE

Taking moments about the centre of rotation

$$Wr + Jl = C_1x + C_2y$$

$$W = \frac{C_1x + C_2y - Jl}{r} \qquad (2)$$

Safe working load (P) = W − margin for safety.
Equation (2) if plotted with varied r produces fig. 19.9, similar in shape to fig. 19.7.

For safety, the luffing-type crane operating radius of the boom is restricted to A min ≈ 0.25 A max. For the saddle-jib the minimum operating radius A min is limited to approximately 0.05–0.2 A max. In general the load-radius diagram is further restricted as shown in fig. 19.9 for loads lifted near to the mast, and A m ≈ 0.25–0.4 A max. P min ≈ 0.1–0.2 P. The saddle-boom crane is often quoted in terms of the load-moment capacity, which is calculated by multiplying the load by the radius. Usually the maximum load moment capacity occurs at A m and the minimum load moment at A max.

Tower cranes are now available to suit many situations and the data shown in tables 19.1, 19.2, 19.3, respectively for luffing-booms, saddle-booms and self- evecting models are typical of the free-standing capabilities with the crane operating with the longest possible boom.

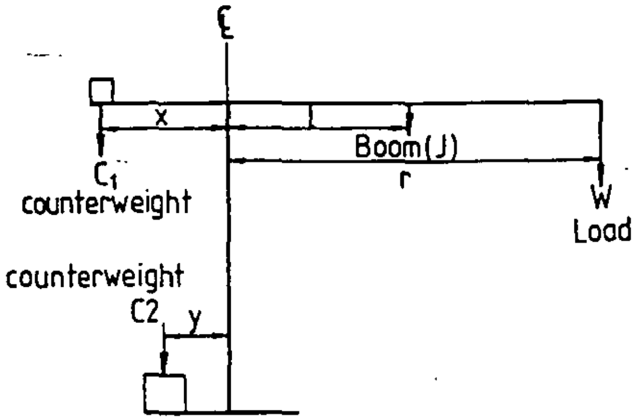


Figure 19.8 Forces acting on a saddle-om tower crane

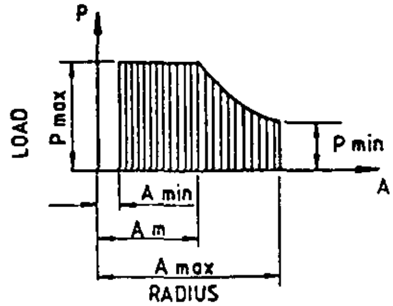
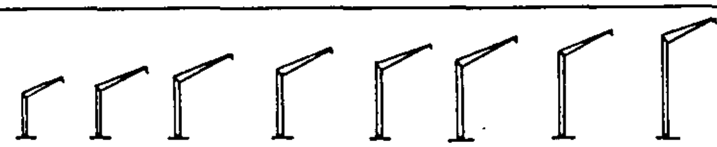


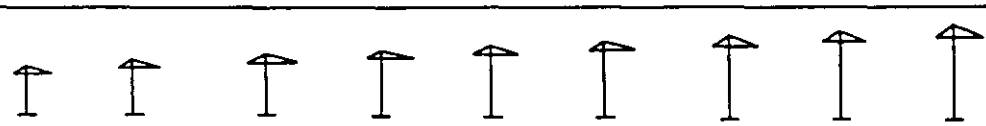
Figure 19.9 Load–radius diagram for the saddle-boom tower crane

Table 19.1 Luffing-boom load-radius data



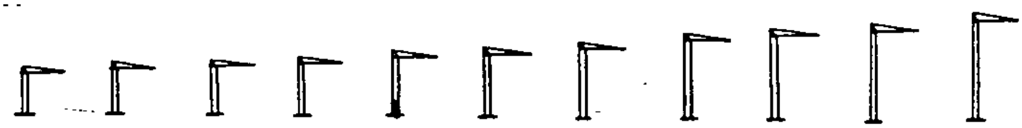
Max. radius }	m	18	26	30	36	42	50	56	75
Lifting capacity }	tonnes	0.75	1.1	0.6	1.0	1.4	1.8	2.0	5.6
Max. lifting capacity }	tonnes	1.2	5	5	11	12	21	16	45
Radius }	m	7	6	7	6	8.5	8.5	13	19
Height to jib pivot	m	18	27	29	35	46	52	58	76
Max. hook height	m	27	49	52	61	75	90	106	140
Track gauge	m	2.35	2.8	3.8–4.0	4.4–4.5	5.0	6.3	6.3–7.1	10.0
Power supply	kW	20	40	50	70	80	120	150	260

Table 19.2 Saddle-boom tower crane load-radius data



Max. jib radius	m	36	40	45	50	60	70	70	80	80
Capacity	tonnes	1.0	1.5	2.5	2.9	3.6	5.0	12.2	14.5	22.8
Max. capacity }	tonnes	3.0	8.0	10.0	12.0	20.0	20.0	50.0	64.0	64.0
Radius }	m	14.4	10.6	14.0	14.6	16.3	22.4	20.0	23.8	34.8
Max. load moment	mt	43	85	140	175	326	448	1000	1523	2238
lin. radius	m	7.0	4.0	3.0	3.0	3.0	3.9	3.9	4.0	4.0
Height under hook	m	30	38	44	47	66	80	90	140	105
Track gauge	m	2.8	4.5	4.5	4.5	8.0	8.0	10.0	15.0	15.0
Power supply	kW	40	40	60	100	150	150	170	200	200

Table 19.3 Self-erecting saddle-boom tower crane load-radius data



Max. jib radius	m	11	16	18	20	25	30	35	35	40	45	50
Capacity	tonnes	0.3	0.65	0.75	0.8	1.0	1.0	1.0	3.0	1.5	1.75	2.0
Max.capacity	tonnes	0.45	1.5	1.5	1.7	3.0	4.0	6.0	8.0	8.0	8.0	10.0
Radius	m	7.8	8.2	10.3	11.42	10.3	9.4	8.8	14.9	10.7	13.4	14.0
Max. load moment	mt	3.5	12.3	15.5	19.4	30.9	37.6	52.8	119.2	85.6	104.8	140.0
Min. radius	m	3	3	3	3	3	3	3	3	3	3	4
Track/wheel gauge	m	2.0	2.32	2.8	3.2	2.8	3.8	4.5	6.0	5.0	5.0	6.0
Power supply*	m	16	16	16	22	20	25	40	35	50	50	60
Max. hook height	kW	10.6	16.0	18.0	18	20	20.0	32.8	29.3	32.8	37.8	32.8

*Self-contained generator available on some models.
N.B. Available (1) static on outriggers; (2) travelling on rail track; (3) on crawlers.

TYPES OF TOWER CRANE (BOTH LUFFING- AND SADDLE-BOOM TYPES)

FREE-STANDING CRANE

The free-standing crane, stands self-supporting on a well prepared founda tion. The mast is bolted to a strong steel cruciform base (fig. 19.10). Ballast consisting of blocks of concrete, iron or gravel, is placed on the base to provide a counterbalance.

Free-standing cranes are available up to 100 m clear distance between the hook and ground level. To increase the height further the crane must be tied to the structure under construction.

The free-standing arrangement is commonly adopted and includes both the saddle- and luffing-boom types. It is usually preferable to position the crane outside the building, as this makes for easier erection and dismantling and also avoids the costs of leaving out and making good parts of the structure.

With the fixed-position base the crane boom must be capable of reaching all parts of the job and so tends to be used for building construction and particularly high-rise construction where regular shape structures are involved as typified in fig. 19.11.

RAIL-MOUNTED CRANES

An alternative and popular foundation is the rail-mounted base. With this arrangement the crane can travel with the load on the hook on a specially prepared track, which may be straight or curved.

The base frame is made from sturdy welded steel plate. For straight track the frame is of simple construction mounted on four wheels as shown in fig. 19.13. For curved track (fig. 19.14) the frame is more sophisticated and the

Figure 19.10 Free-standing tower crane base layout

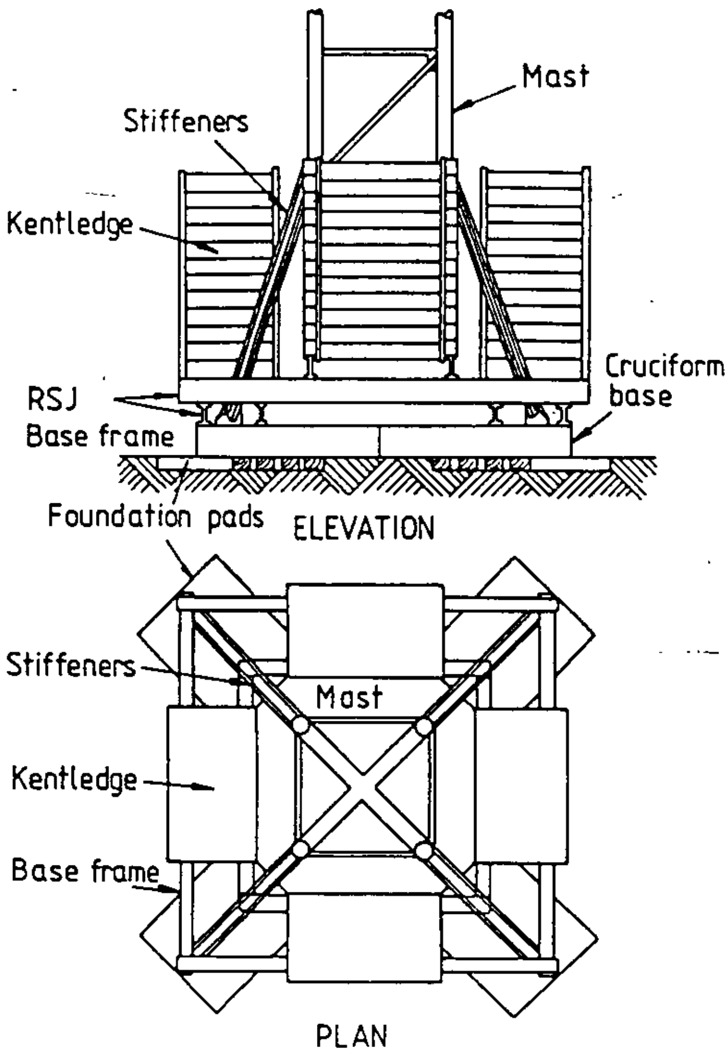
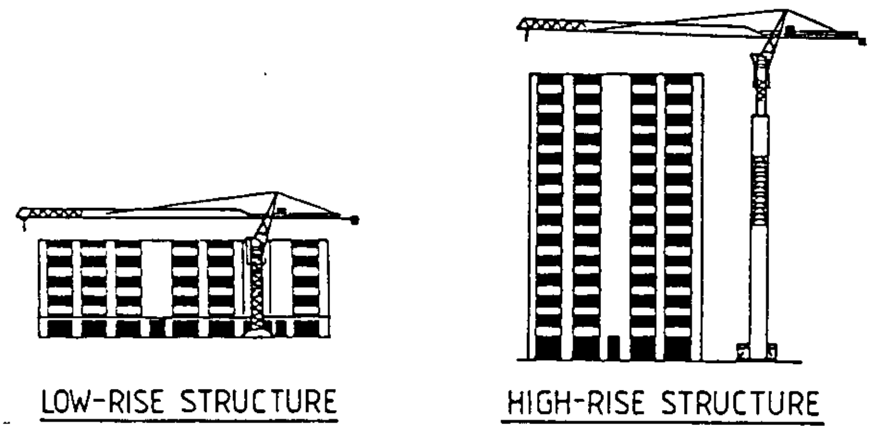


Figure 19.11 Free-standing tower crane applications



arms and wheels are pivoted to accommodate the bends in the track. Usually two wheels are provided under each support. The crane can be used to construct both high and long buildings and is able to travel to material stock-piles, concrete batching plants, etc.. located in close proximity to the structure.

There are some disadvantage compared with the free-standing tower crane. as follows:

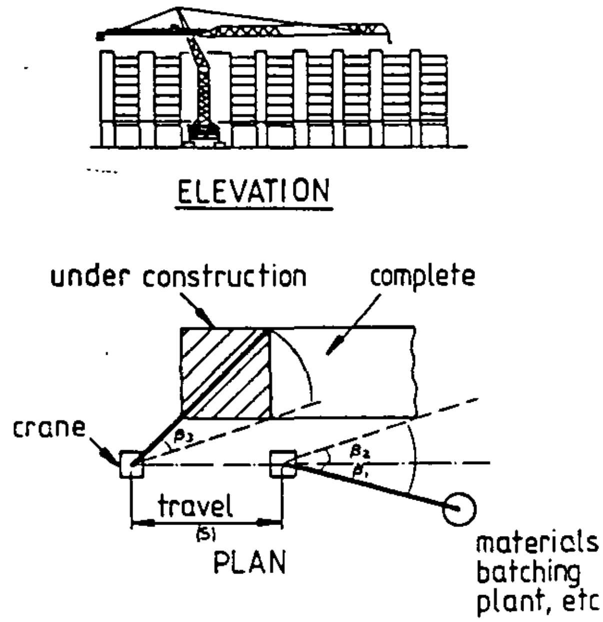


Figure 19.12 Rail-mounted tower crane

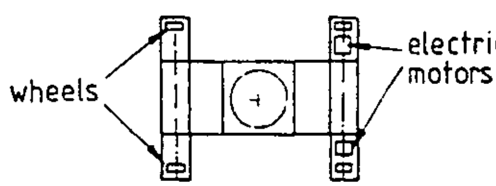


Figure 19.13 Tower crane base for straight rail track

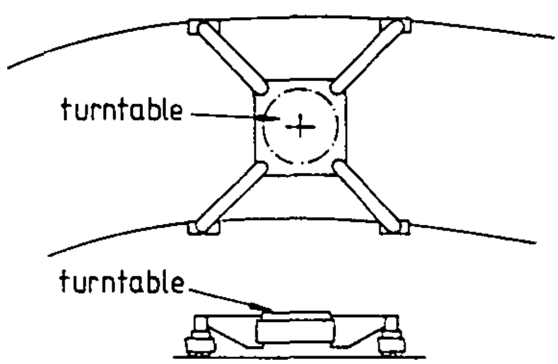


Figure 19.14 Tower crane base for curved rail track

(a) Track is expensive to purchase and lay.
(b) The site must be firm and the crane can only be used on a level track, rails out of level by only a few millimetres render the mast unstable.
(c) The crane's travel motors can accommodate gradients only up to about 1 in 200.
(d) The track must be kept clear at all times, which may entail supervision, especially on a busy site. and so add to the costs.
(e) The track may occupy a valuable area of the construction site, which cannot be used for storage of materials.

Rail-mounted cranes are electrically powered with separate three-phase a.c. motors, requiring a high voltage supply of 350–415 V. The travel wheels are driven through gears to enable the crane to move as fast as is safe with the particular load being carried. Sometimes the crane is provided with a special braking system to allow fine movement when precision positioning of the load.

CRAWLER-MOUNTED TOWER CRANES

There has been a limited demand for tower cranes with greater mobility than is provided by rails. For example, crawler-mounted tower cranes (fig. 19.15) have been successfully used to pour the concrete foundations and floors, and lift out the waste materials for the construction of housing and low-rise dwellings. For economic application, however, duties would need to include the placing of all materials plus positioning any units such as precast floors and walls. Such cranes, however, are relatively small – up to 1500 kg capacity at 40 m radius. The crane is brought to site on a low loader.

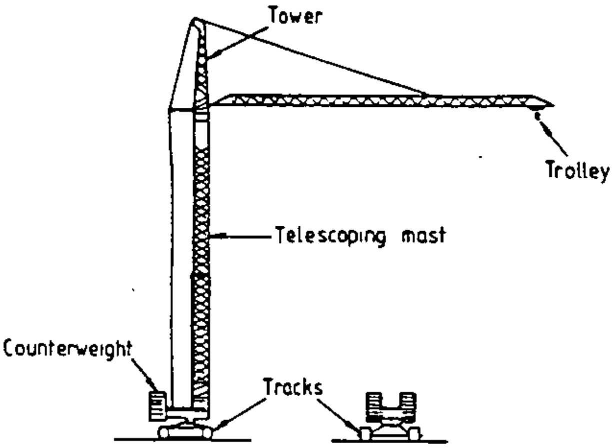


Figure 19.15 Crawler-mounted tower crane

TIED-IN TOWER CRANE

The free-standing tower crane supported on a ballasted base or rails can be used up to heights of about 100 m. Above this the crane must be tied in to the structure under construction at points recommended by the manufacturers of the crane, to coincide with suitable positions on the building, such as the floors of a high-rise dwelling. The tie frame is of special design usually in the form of a lattice frame which spreads out from the mast and is attached to the structure to resist forces from all horizontal directions. By careful selection of the fixing positions the operating height may be extended up to 200 m or more. The base may be ballasted as in fig. 19.10 but frequently, where the site conditions allow a concrete foundation block is cast into the ground as shown in fig. 19.17. The mast is then simply bolted down. Most of the overturning moment is taken by the structure rather than the foundation, resulting in a lighter and neater base than either rail or ballast types, with a consequent increase in working space on a congested site.

TRUCK-MOUNTED TOWER CRANE

The strut-boom truck-mounted crane of the lattice type may be assembled to operate as a luffing tower crane as an optional feature on the large cranes

Figure 19.16 Tied-in tower crane

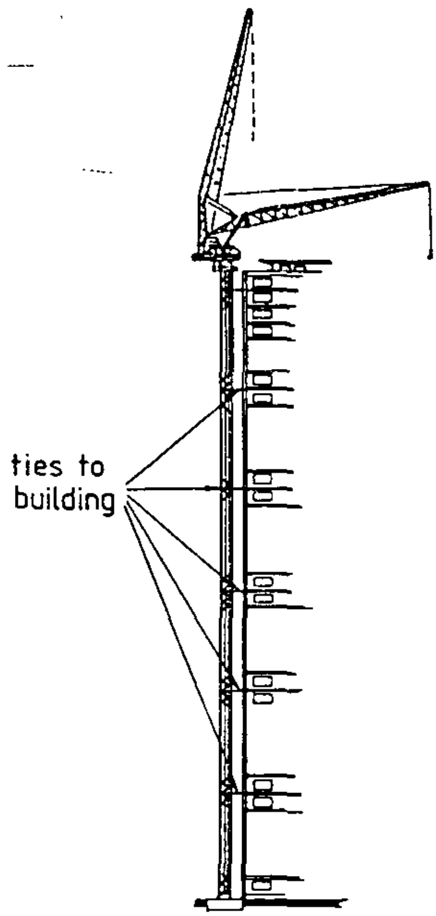
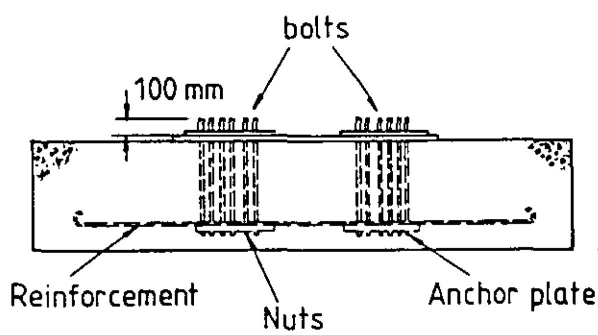


Figure 19.17 Base for a tied-in tower crane



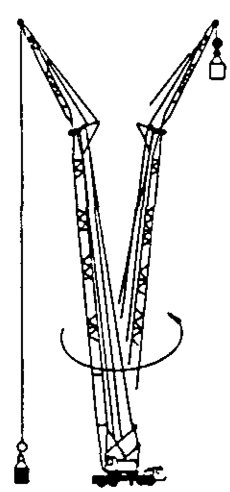


Figure 19.18 Truck-mounted tower crane

of 100 tonnes capacity and greater. The fly-jib is replaced by a much longer luffing boom which operates with the safe working load at 75% of the tipping load. The crane is suitable for short duration one-off tasks where heavy loads need to be lifted into high wide structures. Lifting data are given in table 18.1.

JACK-KNIFE CRANE

The jack-knife crane is designed to work in extremely tight quarters, for example, work between two high buildings. A free-standing horizontal boom crane would have to free sail over the top of such structures and thus might pose an unnecessarily expensive solution to the problem. Capacities are available for all the common site duties and permit about 2 tonnes to be raised at 30 m radius.

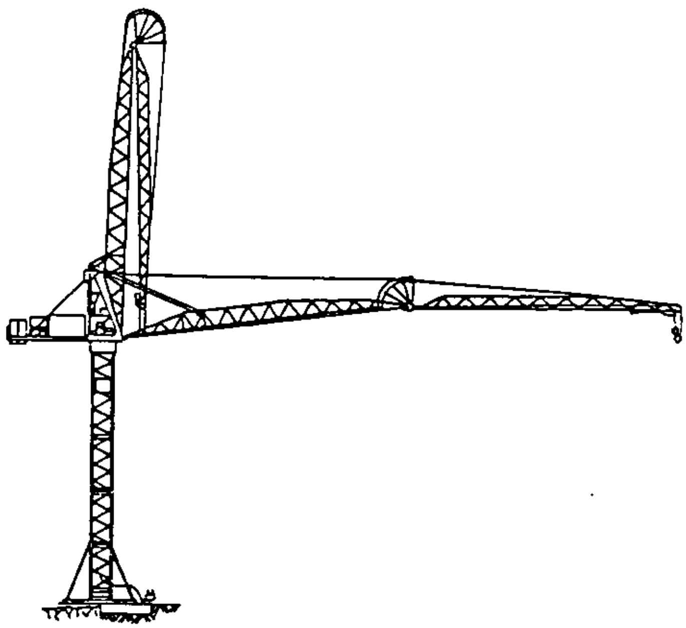


Figure 19.19 Jack-knife tower crane

CLIMBING CRANE

Where external site space is at a premium and the shape of the building allows, a tower crane may be located inside the building. Frequently the lift shaft in high-rise structures serves as a convenient position to locate the mast. Generally the tied-in crane referred to earlier is used in this situation but where the client permits, a climbing crane may be cheaper, as the foundation and most of the mast is dispensed with. The horizontal and vertical thrusts must therefore be taken by the structure itself.

Initially the mast is mounted on a fixed base which is usually part of the foundations of the structure. The crane then proceeds to erect the building around itself up to at least the second floor. It is then secured by special collars at each floor level. Thereafter it climbs without support away from the original foundations base. The two principal methods of raising the mast are by winches or hydraulic jacks.

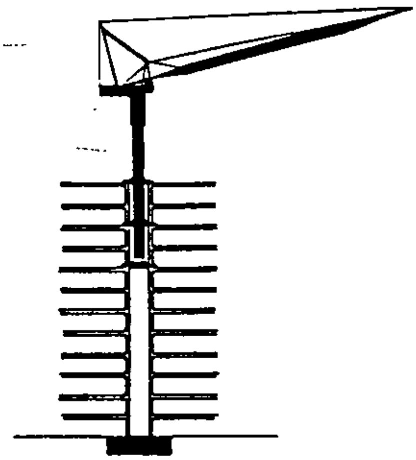


Figure 19.20
Climbing tower crane

Winch Raising

A line (F) is attached to the collar (H) which is firmly fixed to the structure. The line is passed under two pulleys (E) fixed to the base of the mast and over a third pulley (G) connected to the pulley (D). A second line (J) from a winch (A) mounted at the base of the mast is passed over the pulley (C) which is hung from the top of the mast section, it is led down and under pulley (D) and finally attached firmly to the support at (C). The winch winds in rope (J) and raises the mast, lifting it clear of the bottom collar. The collar is then firmly clamped and the lifting tackle is positioned ready for the next lift.

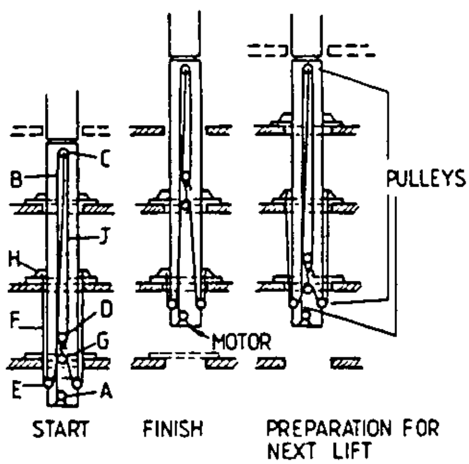


Figure 19.21 Raising a climbing tower crane by winching

Hydraulic Raising

The lifting procedure starts with the mast in position (A) supported on a climbing ladder (1), hung from a bearing plate (6). The collar (2) is firmly clamped to the crane mast, with the spring claws (5) engaged on rungs (4). A sliding collar (3) is linked to collar (2) by means of the hydraulic jack (7). To raise the mast, the claws on collar (3) are positioned to engage ladder rung (8). The jack is extended and thereby lifts the crane from position (A) to position (B). The jack is then withdrawn raising collar (3) in the process.

Dismantling requires the boom and mast to be taken apart in sections and lowered to the ground by winches.

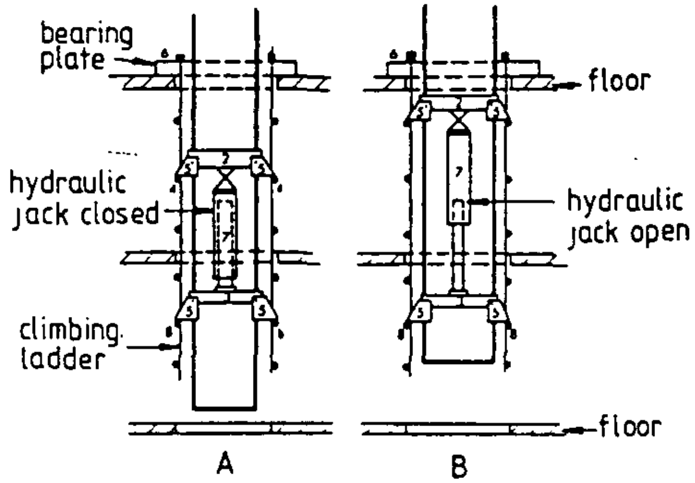


Figure 19.22 Raising a climbing tower crane hydraulic jacking

ERECTION OF TOWER CRANES

TURNTABLE AT THE TOP OF THE MAST

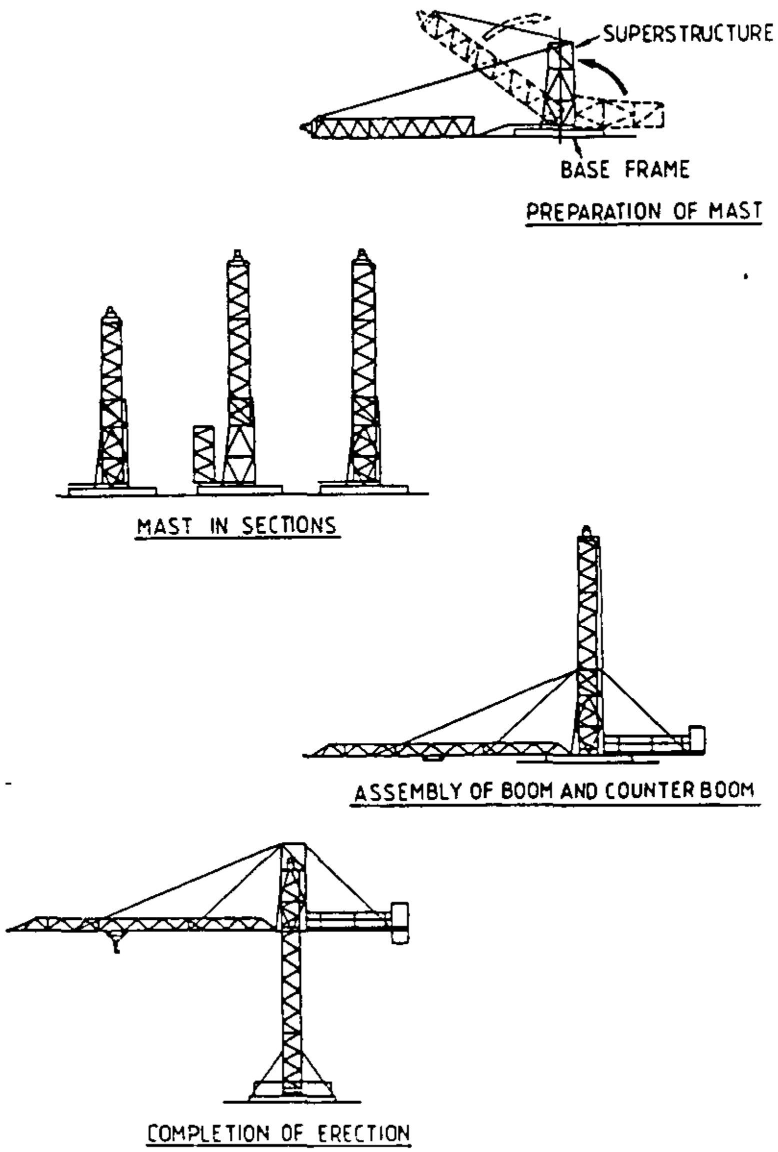
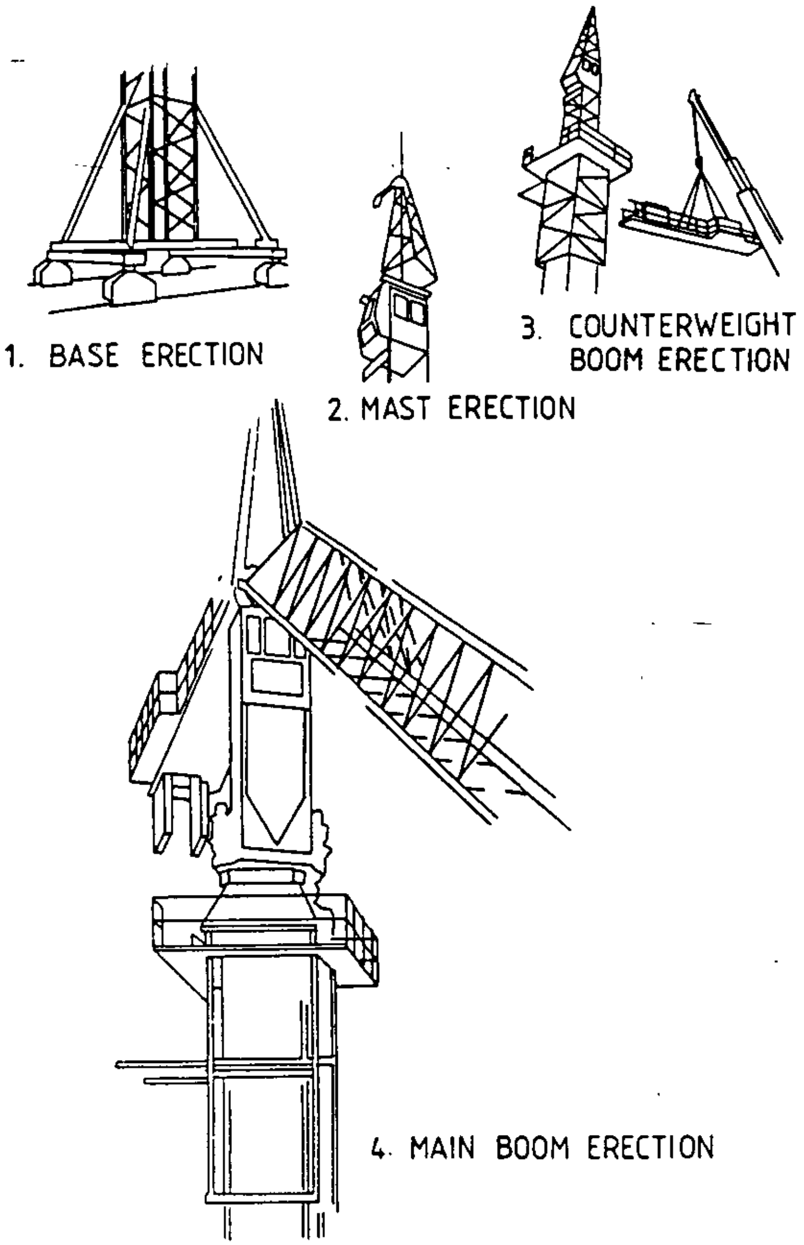


Figure 19.23(a) Example of a tower crane method of assembly

Figure 19.23(b) Erection of a tower crane



An example of a winch-assisted method of erection is shown in fig. 19.22(A). For some makes the erection procedure is lengthy and usually requires the assistance of a separate crane usually a truck-mounted telescoping boom-type mobile. All the parts are brought to site on trucks and the crane is put together in sections as follows:

Stage 1: The base is prepared and the first section of mast is lifted into position.
Stage 2: The climbing cage and turntable are next positioned over the mast.
Stage 3: The counterweight and arm are fixed in place. (Saddle-boom type only.)
Stage 4: The boom is attached and finally self-hoisted or lifted by crane into position.

The crane height is further increased either by introducing mast sections at the top of the mast as shown in fig. 19.24 or alternatively through the side as in fig. 19.25.

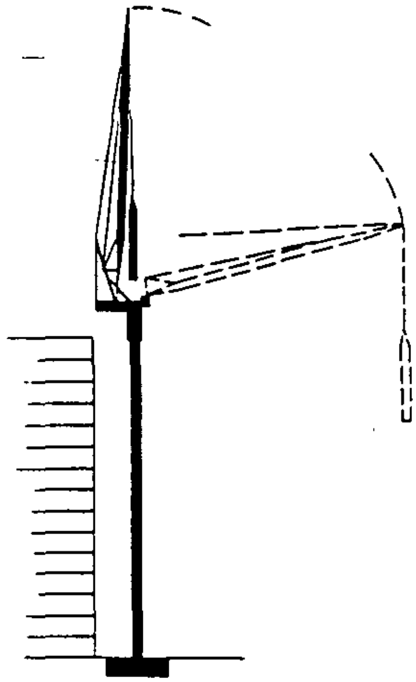


Figure 19.24
Extending the height of a tower crane from the top

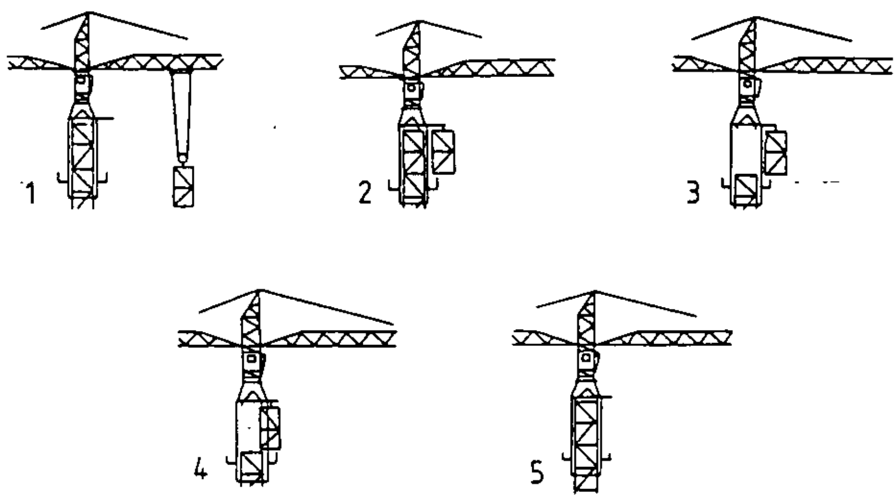


Figure 19.25
Extending the height of a tower crane through the side

TURNTABLE AT THE BASE OF THE MAST

Cranes of both the luffing- and saddle-boom types can be made self-erecting by designing the mast on a telescoping principle. The operating height, however, is limited to the design size of the crane.

The whole unit usually arrives in the collapsed arrangement towed by an articulated wagon (fig. 19.26) or even as a completely mobile truck-mounted crane (fig. 19.18).

Using built-in winches on the crane the tower and jib are unfolded as shown in fig. 19.27.

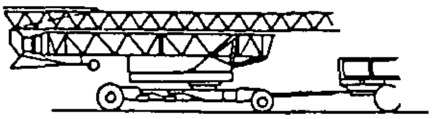


Figure 19.26
Transporting a tower crane on the highway

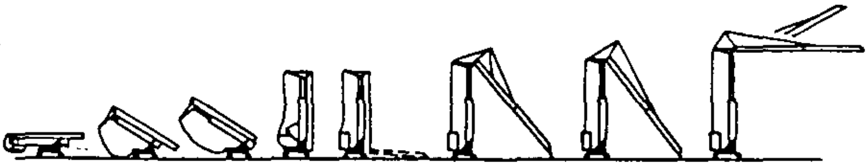


Figure 19.27 Self-erecting tower crane sequences

CRANE ASSEMBLY TIME

None-self-erecting cranes require 1 day for small units and 2 days for large units with crane assistance. Without crane assistane, using only winches, up to 4 days with three men may be required. Self-erecting cranes require from ½ to 1 day. Laying of a 10 m length of rail track takes from 1 to 2 days. The ratio of erection to dismantle time is about 60:40.

TOWER CRANE CHARACTERISTICS

				Crane size *small*	*medium*	*large*
Max. hoist speed (two part line):	raise	V_R	m/min	30.0	55–90	150
	lower	V_L	m/min	80	100	150
Slewing speed		U	rev/min	1.0	0.8	0.6
Trolley speed		V_s	m/min	40	30	2
Travelling speed (on rails)		V_T	m/min	40	30	0
Luffing		t_D	s	20	50	16 90

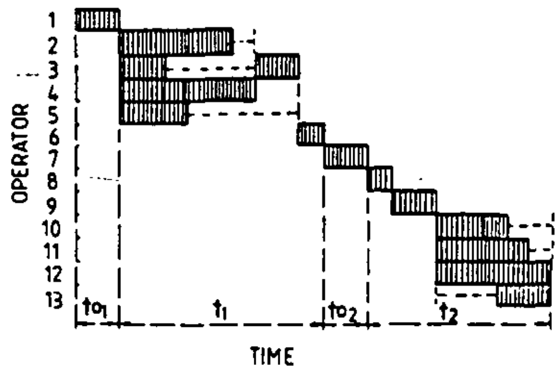
EXAMPLE: CYCLE TIME FOR A LUFFING-BOOM TOWER CRANE (see figs 19.12 and 19.28)

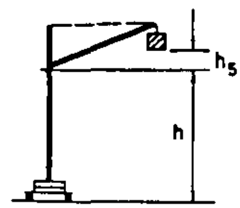
1.	Hook on load	t_{o_1}	$= 1$ min
2.	Raise load from ground level	t_{R_1}	$= (h + h_s)/V_R$
3.	Slewing $(\beta_1 + \beta_2)$	t_{s_1}	$= [\beta_1 + \beta_2)/360] \times U$
	(β_3)	t_{s_2}	$= [\beta_3)/360] \times U$
4.	Travelling along S (on tracks)	t_{T_1}	$= S/V_T$
5.	Derricking jib into position	t_{D_1}	
6.	Lowering load (h_S)	t_{L_1}	$= h_S/V_L$
7.	Unhook load	T_{O_2}	$= 1$ min
8.	Raise hook (h_s)	t_{R_2}	$= h_S/V_R$
9.	Slewing (β_3)	t_{S_3}	$= [(\beta_3)/360] \times U$
10.	Derricking (as 5)	t_{D_2}	
11.	Lowering hook $(h + h_S)$	t_{L_2}	$= (h + h_S)/V_L$
12.	Travelling along S	t_{T_2}	$= S/V_T$
13.	Slewing $(\beta_1 + \beta_2)$	t_{s_4}	$= [\beta_1 + \beta_2)/360] \times U$

The bar programme is shown in fig. 19.28.
The number of cycles per hour (L_0) is given by

$$L_0 = \frac{60}{\Sigma t_0 + \Sigma t}$$

Figure 19.28 Tower crane cycle time





This is the theoretical output which in practice may be reduced by other factors such as weather stoppages, breakdowns, etc. These may add up to total losses of 30%. The experience and skill of the driver are also important. for example the efficiency factor may be 0.95, 0.85 or 0.75 for good, average and poor drivers, respectively. Thus calling the production losses factor e_1 and the driver efficiency factor e_2

$$\text{expected output} = \frac{60e_1e_2}{\Sigma t_0 + \Sigma t} \text{ cycles per hour}$$

Typically $\frac{1}{2}$ yd^3 skip of concrete can place at the rate of about 7 m^3/h and the corresponding rate for a 1 yd^3 skip is 12 m^3/h at heights up to 25 m.

TOWER CRANE EXERCISE (1)

A tower crane required to concrete a 100 m long retaining wall is positioned as shown in fig. 19.29 such that the maximum length of boom is 20 m. The unit is mounted upon rails which run parallel and 20 m distant from the retaining wall. The crane data are as follows:

Raising speed	30 m/min
Lowering speed	60 m/min
Slewing speed	1 rev/min
Travelling speed	40 m/min
Derricking speed	40 s

The operating efficiency of the crane is 85%. The raising and lowering height of the load from ground level is 20 m. The times to fill and empty the skip are:

Fill	30 s
Empty	30 s
Position over wall	30 s

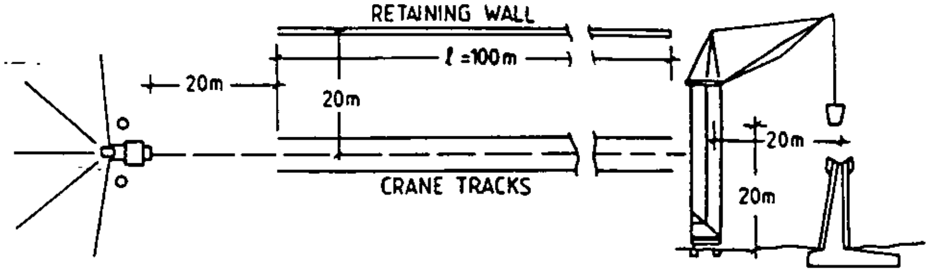


Figure 19.29 Example of a tower crane in use

Problem

(a) How many skips per hour can the crane handle?
(b) How far along the track can the crane travel without influencing the output of the crane?
(c) Draw the crane's operating cycle diagram independent of the travelling time along the track.

TOWER CRANE EXERCISE (2)

Figure 19.30 shows a plan view of the present site layout for the construction of a rectangular high-rise building using a tower crane. Criticise this arrangement

Information (i) 500 litre concrete batcher sited 40 m from tower crane.
(ii) Load moment of tower crane at 40 m radius is 30 mt.

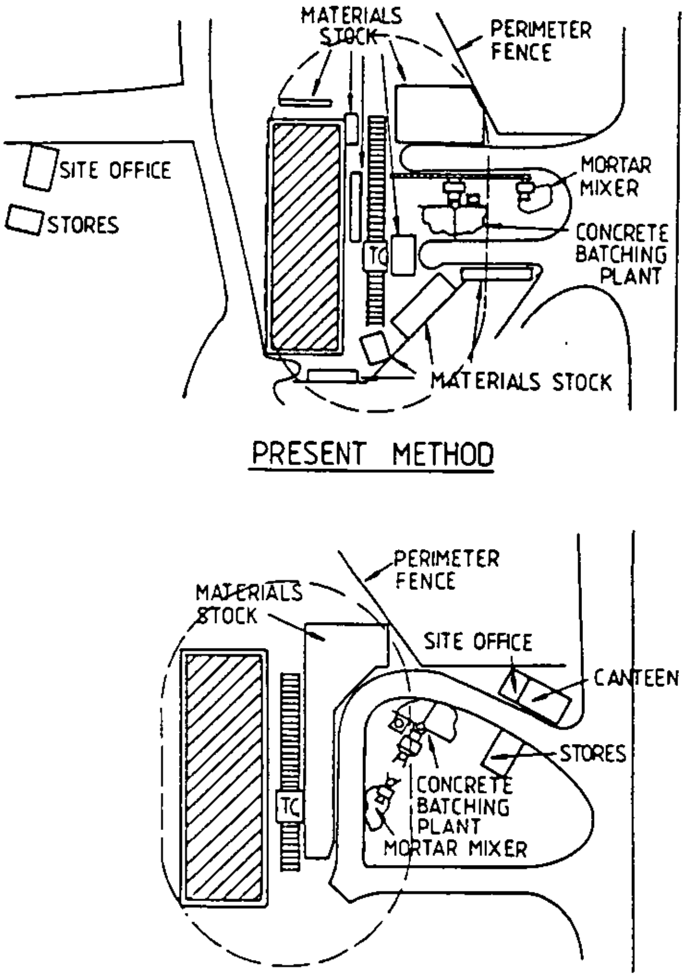


Figure 19.30 Example of a tower crane site layout

Solution

Present method

(a) Three entrances to the site (two not guarded) are required.
(b) A 500 litre batching plant is too large for the safe working load at the required radius.
(c) The mortar mixer is outside the reach of the crane.
(d) Materials are too spread over the site.
(e) Office and stores are too far from the site.

Proposed method

(a) Stores and office are located near the site entrance and exit.
(b) A through road now connects the entrances and exit.
(c) The mortar mixer is located within reach of the crane.
(d) The materials compound is arranged along crane track.
(e) The mixer is reduced to 375 litre to match the tower crane capacity.
(f) The third entrance is eliminated.

FACULTAD DE INGENIERIA U.N.A.M.
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CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MÓDULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA

CONEXIONES VIGA - COLUMNA, JUNTAS, TRASLAPES Y TORNILLOS DE ESTRUCTURAS METÁLICAS

ESPOSITOR: ING. VICTOR J. SÁEZ DE OCARIZ ALBISÚA

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OCTUBRE DE 1998

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Chapter Eighteen

Beam-To-Column Connections

18.1 INTRODUCTION

Beam-to-column connections play an important role in the load partition of structural frames. The major function of these connections is to transfer the loads that are applied to the beams and the floor system to the columns. In its simplest form, the connection is used to transfer only the end reaction of the beam to the column, and the beam is assumed to be simply supported. If restraints are provided, the end rotations of the beam are minimized, and the maximum positive moment in the beam can be reduced by the resulting end moments. Connections of this nature are often referred to as moment-resistant joints. Connections that are only capable of transferring the reaction of the beam are called shear connections.[18.1]

The behavior of beam-to-column connections is of major interest to engineers, and a significant amount of research has been done or is underway. In one category studies are aimed at developing and improving design rules for the beam-to-column connection.[16.1–16.3,18.1–18.22] This work focuses on the general requirements for connections, that is, (1) sufficient strength, (2) adequate rotation capacity, (3) sufficient stiffness, and (4) economical fabrication. The role of the beam-to-column connection in overall frame behavior is also of interest, and the prediction of the moment versus rotation characteristics of typical connections is a subject of recent and current study.[18.23–18.28]

Most of the early research on beam-to-column connections was performed on welded or riveted specimens. However, as the advantages of bolted connections and combination bolted and welded connections became more apparent because of decreased fabrication and erection costs, research on these types of connections was increased.

In current practice shop connections are often welded and field connections bolted. As a result of these fabrication procedures, a wide variety of beam-to-column connections are encountered in the field. It is still not possible to accurately describe and predict the behavior of many of these connections because of their complexity. This chapter summarizes the present state of knowledge and provides guidelines for design. The design recommendations for these joints are based on available information and result in a conservative, safe design. The ongoing experimental and theoretical work will permit the development of more liberal and improved design rules.

18.2 CLASSIFICATION OF BEAM-TO-COLUMN CONNECTIONS

Depending on their rotational characteristics, beam-to-column connections are classified as flexible, semi-rigid, or rigid connections.[18.1] Flexible connections are also called shear connections, and the semi-rigid and rigid-type connections are often referred to as moment-resistant connection.

The rotational characteristics of beam-to-column connections are important to the engineer because they affect the required beam size. For idealized rigid joints, the beam size is generally governed by the fixed end moment: for example, $M = wl^2/12$, for a uniformly loaded beam. If the same beam is attached to the column by a flexible-type shear connection, the maximum moment for the same loading case is $M = wl^2/8$. Actual situations in the field will generally be somewhat less rigid than assumed for the rigid connection and somewhat more rigid than assumed for the flexible connection. The classification of a connection depends entirely on the joint geometry and loading conditions. Generally, it is not possible to define how a joint should be classified unless test results and experience are available.

The simplest type of beam-to-column connection is the flexible connection that provides relatively low resistance against rotations. Hence, the connection mainly transfers shear to the column. Typical examples that fall into this category are the web angle connection (sometimes called the standard beam connection), web structural tee, and seat angle connections, shown in Fig. 18.1*a*. The structural T-connections, end-plate connections, and flange plate connections, shown in Fig. 8.1*c*, are typical examples of beam-to-column connections with high moment resistance. By combining web angles or a T-section with a beam seat and tension flange plate or angle, a semi-rigid connection results that has a greater moment resistance than the flexible connection. Unfortunately, the degree of restraint is difficult to evaluate unless test data are available.

Typical moment versus rotation characteristics for several types of beam-to-column connections are shown in Fig. 18.2. These relationships, combined with the beam line concept (introduced in Ref. 18.1), are often used to estimate the moment that will be developed by a particular connection, span, and beam size. The beam line defines the relationship between the end moment and end rotation of a beam. If a beam is uniformly loaded and subjected to restraining end moments M, the end slope ϕ is equal to

$$\phi = \frac{1}{24}\left(\frac{wl^3}{EI}\right) - \frac{Ml}{2EI}$$

This relationship is plotted in Fig. 18.2. The intersection of the beam line and moment versus rotation curves for the various connections indicates the moment resistance expected under these conditions. For example, the standard web angle connection (connection A in Fig. 18.2) develops about 20% of the fixed end mo-

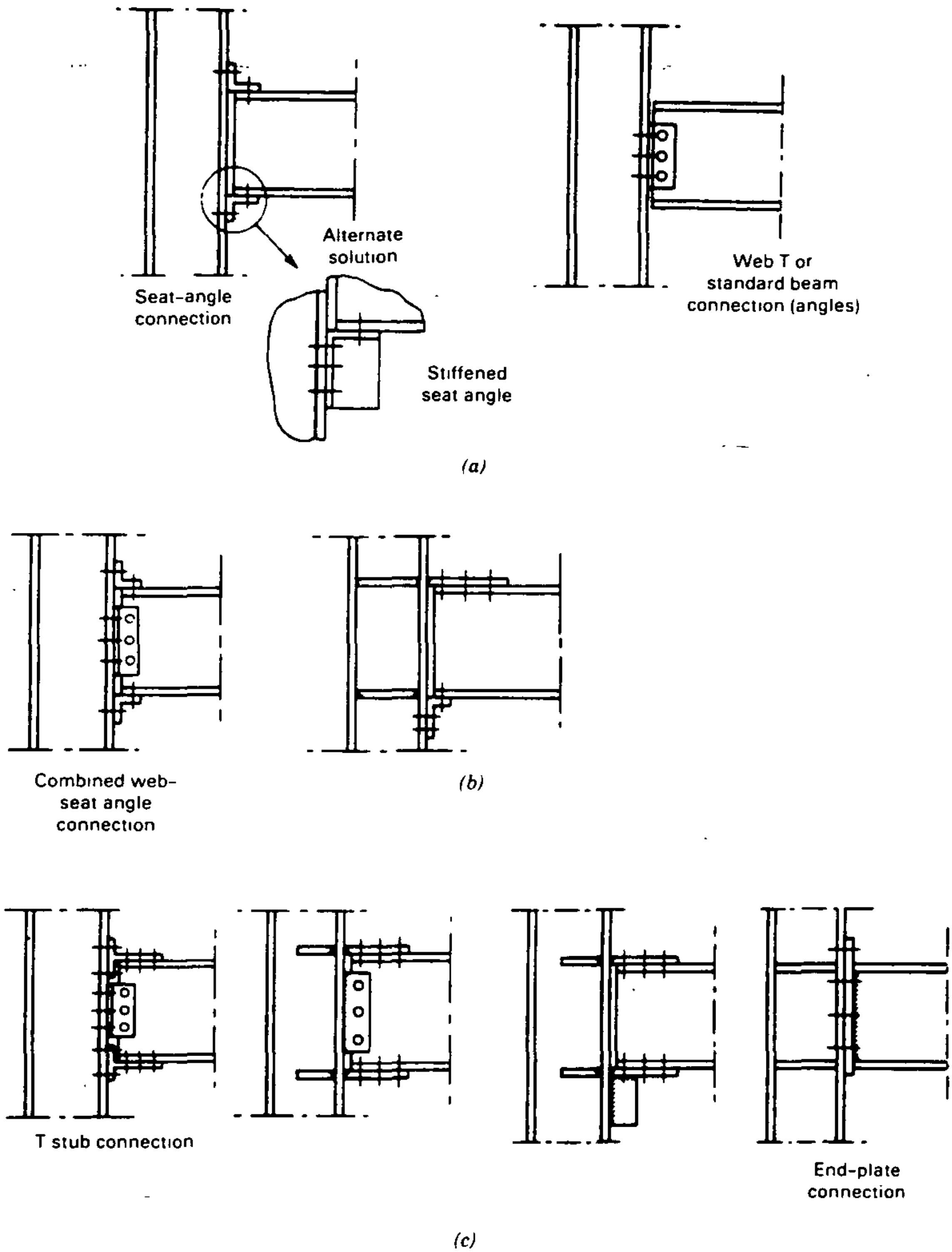


Fig. 18.1. Types of beam-to-column connections. Note. The need for column stiffeners in any of these connections must be checked. (*a*) Flexible connections. (*b*) Semi-rigid connections. (*c*) Rigid connections.

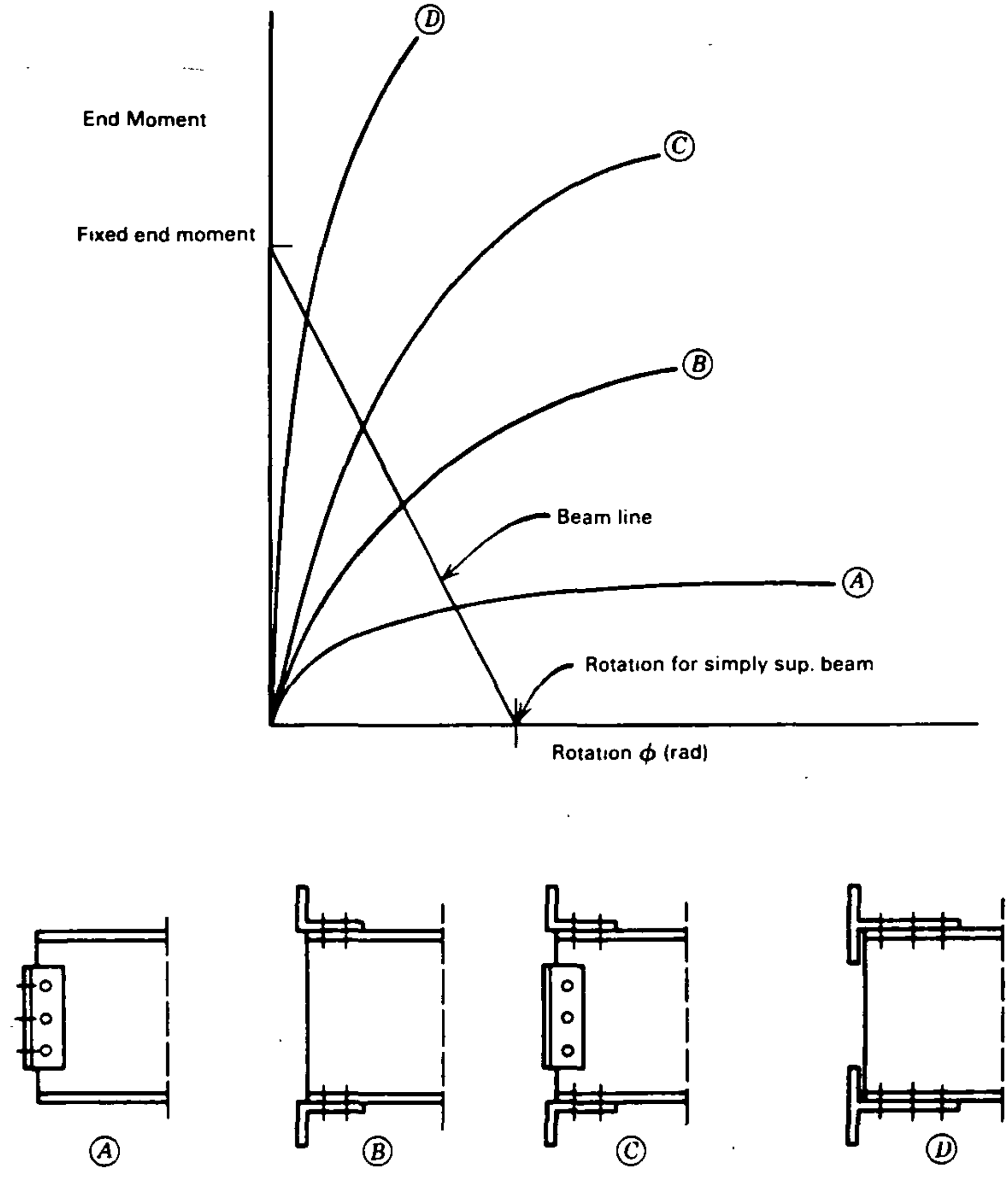


Fig. 18.2. Typical moment versus rotation curves and beam lines.

ment for this particular combination of beam and connection geometry. The same connection with added top and seat angles (connection C) develops about 75% of the fixed end moment.[18.1]

18.3 BEHAVIOR OF BEAM-TO-COLUMN CONNECTIONS

The stiffness and strength of beam-to-column connections are closely interrelated and of major importance to the performance of the connection. Strength requirements ensure that the connection has the ability to transfer the anticipated loads. Stiffness requirements relate to the ability to develop the desired restraint or lack of restraint. To meet the stiffness and strength requirements, additional stiffening of the column web or flanges may be needed, since certain joint components are subjected to highly localized, concentrated forces. Stiffeners are often necessary to prevent crippling of the column web in the compression region, excessive yielding of the column web, or deformation of the column flange near the tension flange of the beam. If the shear capacity of the column web is critical, shear stiffening may be required for that purpose as well.

The load versus deformation characteristics and approximate methods of analysis for typical beam-to-column connections are discussed in this section. Features from different types of connections are sometimes combined to meet the design requirements. Only the strength aspects of the connection are discussed in this section. Problems related to stiffening of the column web are treated separately (Section 18.4). The influence of the restraint characteristics on column or frame strength is not discussed in this Chapter.

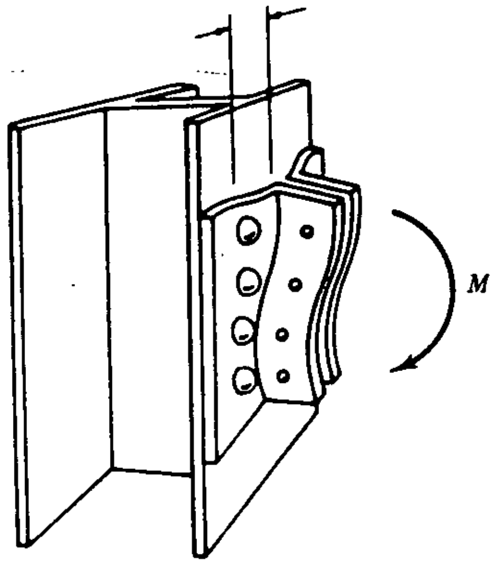
18.3.1 Flexible Beam-to-Column Connections

The web angle or standard beam connection, as well as the seat angle connection, are typical flexible beam-to-column connections. Generally, they are assumed to be completely flexible and capable of transferring only shear. To justify these assumptions, the connections must allow for ample end rotation.

The rotation capacity of the connection is governed largely by the deformation capacity of the angles, as depicted in Fig. 18.3. Experiments have indicated that most of the rotation of the connection comes from the deformation of the angles; fastener deformations play only a minor role.[18.1,18.2] To minimize rotational resistance, the thickness of the angle should be kept to a minimum and a relatively large gage, g, provided (see Fig. 18.3).

A typical moment versus rotation diagram for a standard web connection that used both bolts and rivets is shown in Fig. 18.4. In this test, the heels of the angles on the tension side began to separate from the column flanges at about 260 kip-in. The toes of the angles remained in contact with the column. Yielding of the angles decreased the rotational resistance. After the compression flanges of the beams had made contact with the column flanges, the moment resistance of the connection increased, as shown in Fig. 18.4. Failure of the connection occurred from excessive yielding and tearing of the connection angles (see Fig. 18.5).

From this test series it was concluded that web angle beam-to-column connections offer some resistance to rotations at the ends of the beam. This partial restraint is relatively small and estimated to be about 10% of the fixed end moment provided by rigid moment-resistant connections.[18.2,18.28] Rotation restraints of the same order of magnitude can be expected in seat angle connections as well.[18.3] Jones *et al.* have provided a useful review (through 1980) of test data for various types and configurations of connections and show how a B-spline fit of data can be used to provide a good representation of the load versus deformation characteristics.[18.27]



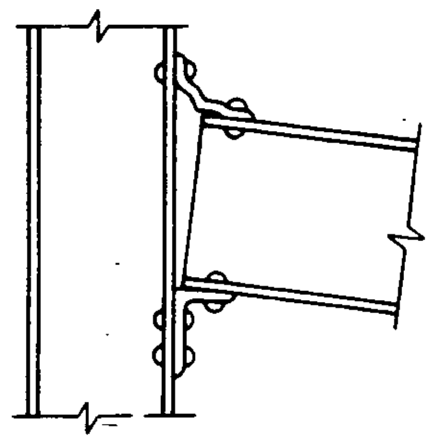
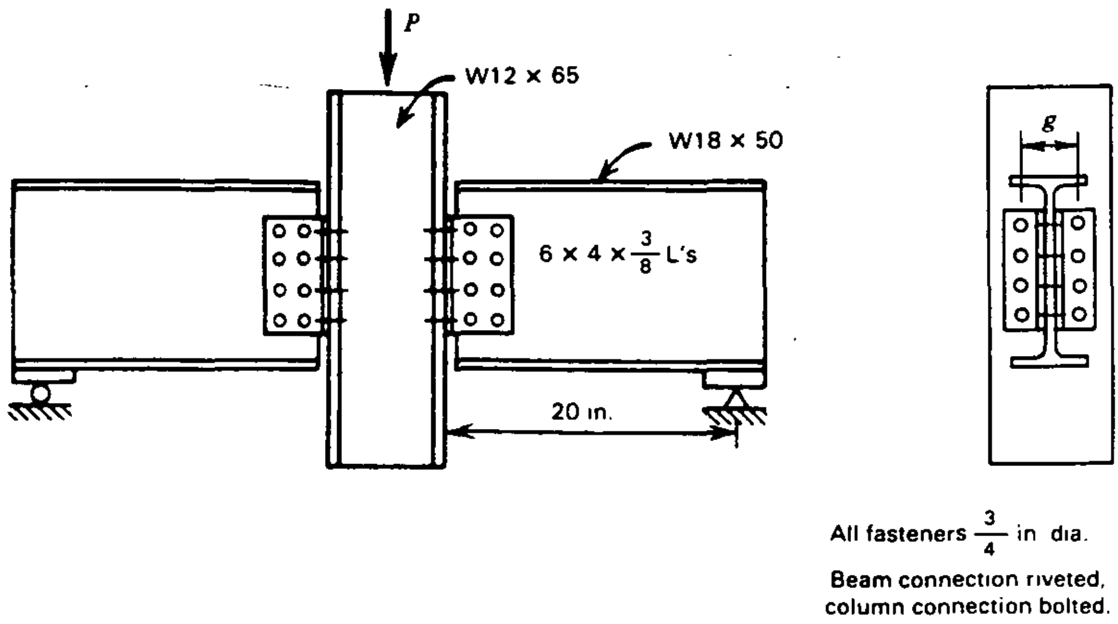


Fig. 18.3. Deformations of flexible beam-to-column connections.

Most web angle connections are checked only for their shear-carrying capacity, that is, the relatively small amount of moment present is neglected. This shear capacity can be governed by (1) the shear capacity of the fasteners, (2) the bearing capacity of the material adjacent to the bolts (angle legs adjacent to both column flange and beam web), including a check of end and edge distances, and (3) the shear capacity of the angles. Fasteners are assumed to be subjected to shear forces only; the tensile forces introduced by deformation of the angles (Fig. 18.3) are neglected. However, the effect of shear forces acting eccentrically should be included unless distances are small. The usual assumption is to consider the bolt group in the web as acted upon by an eccentric shear (Fig. 18.6), although work by Richard *et al.* on single plate framing connections indicates that this may not be a large enough allowance.[18.20]

The examination of end and edge distances for the fasteners should recognize that the rotation of the beam will result in the type of behavior shown in Fig. 18.7*a*. The upper bolts in the group will tend to push out material toward the end of the beam, and the lower bolts will tend to push out material toward the toes of the angles. It would be conservative to use the distance e_1 in checking the bearing capacity of the beam web and the lesser of e_2 and e_3 for the angles (see Fig. 18.7*b*).

The special case of coped beams should be recognized. Coping (or cutting back) of a flange might be necessary when a beam is to be connected to a column web or when a beam-to-girder connection requires that the top flanges be kept at the same elevation (Fig. 18.8*a*). In such a case, it is evident that a new mode of failure



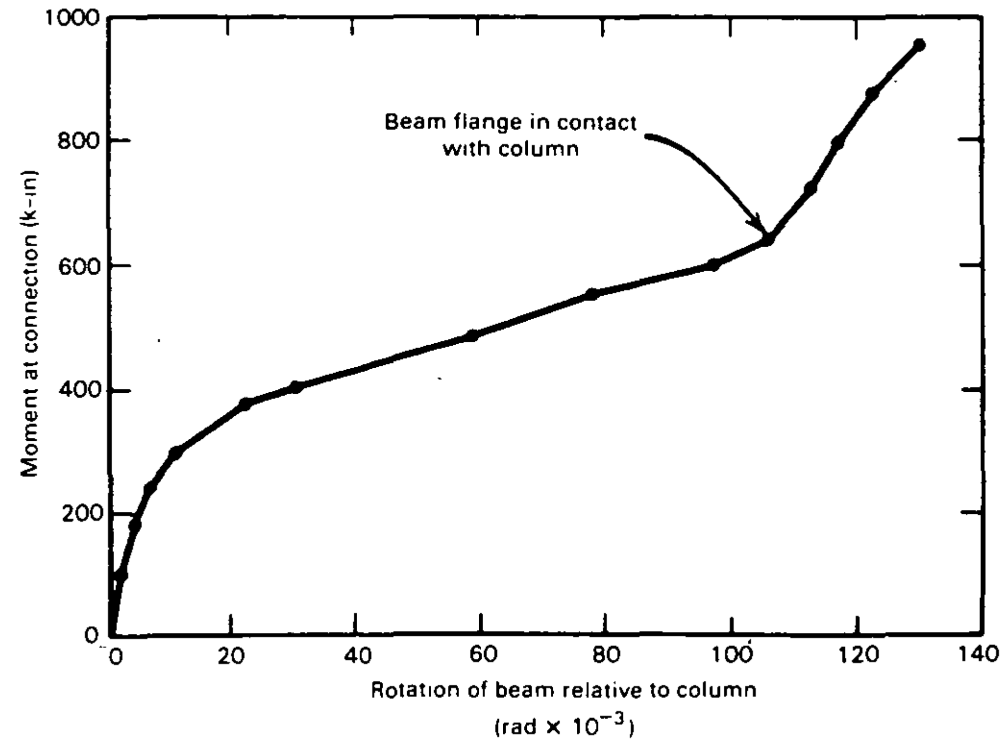


Fig. 18.4. Load versus deformation behavior of standard beam connection (Ref. 18.2).

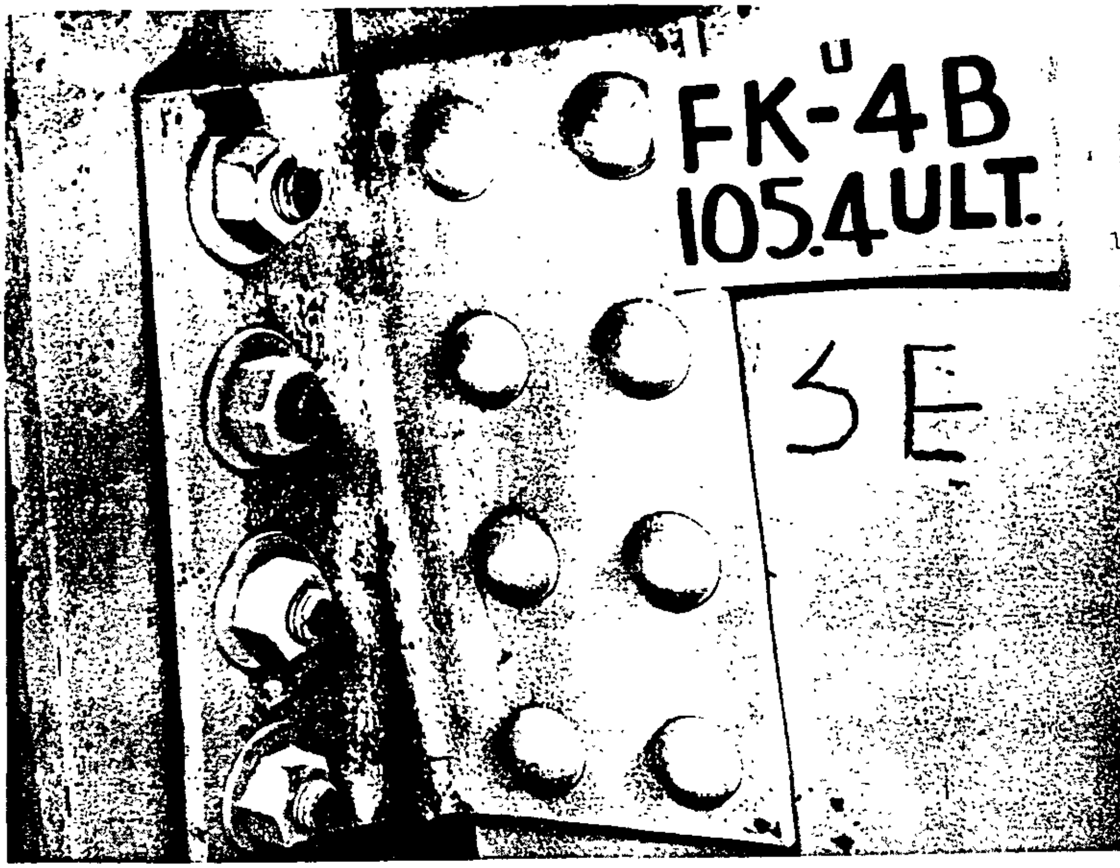


Fig. 18.5. Angle failure in standard beam connection described in Fig. 18.4. (Courtesy of University of Illinois.)

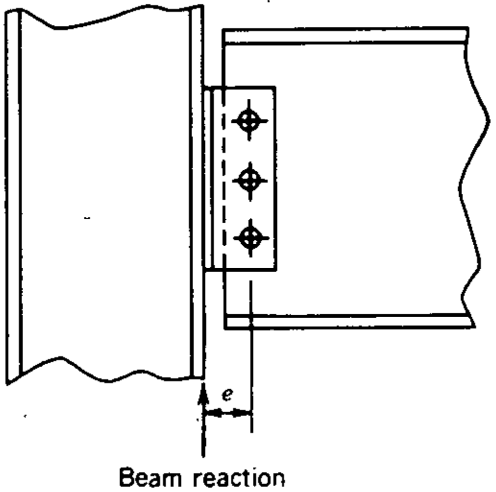


Fig. 18.6. Eccentric shear acting on bolt group.

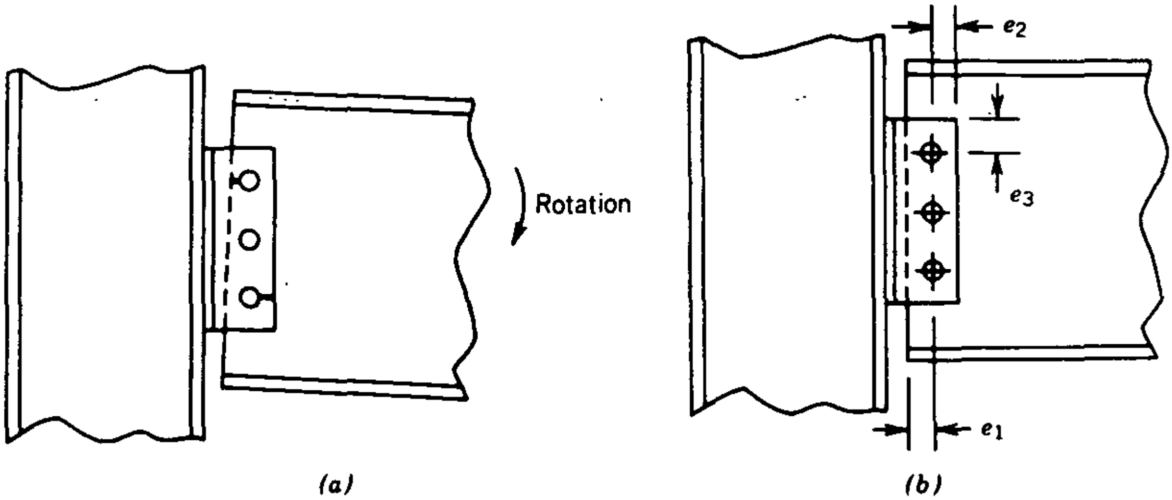


Fig. 18.7. Effect of beam rotation on bolts. (*a*) Actual; (*b*) idealization.

is possible, the removal of a block of material, as indicated in Fig. 18.8*b*, by a combination of shear and tensile forces as the beam rotates relative to the angles. It has been found[18.17–18.19] that a good representation of the ultimate strength is given by the sum of tensile resistance on the horizontal surface a-b (Fig. 18.8*b*) and the shear resistance on the vertical surface b-c. Conservatively, the shear resistance could be used over the whole length a-b-c. Of course, the effect of the cope on the strength and stability of the beam also should be examined[18.38].

Instead of the double angles attached to the beam web that have been described thus far, a single angle or a single plate on one side of the beam web can be used. Obviously, there can be a saving in material (although the single element must be relatively thicker than either component of a double element), but more important savings usually result from the reduced cost of erection. It is much quicker to erect a beam that can be moved in laterally to a single connection piece than to bring a beam web into position between two connection pieces. This type of connection has received considerable attention recently.[18.13,18.20,18.21]

The framed beam connection has elements subjected to flexure (the outstanding legs of the web angles, especially) that give it ductility and that greatly contribute to fulfillment of the assumption of no (or, at least, little) restraint. A single plate framing connection has no comparable component. The ductility in this type of connection must come from the shear deformation of the bolts, from hole distortion (in the beam web or in the plate), and from out-of-plane bending of the plate. Of course, bolt slip prior to bearing might also be present, but this cannot be relied upon. Thus, although designers usually consider the single plate framing beam connection to be a flexible connection, care must be taken to ensure that sufficient ductility does exist so that the design assumption is satisfied.

For a single plate framing connection of the type shown in Fig. 18.9, the design requirements include selection of the bolts (shear capacity, checking the bearing

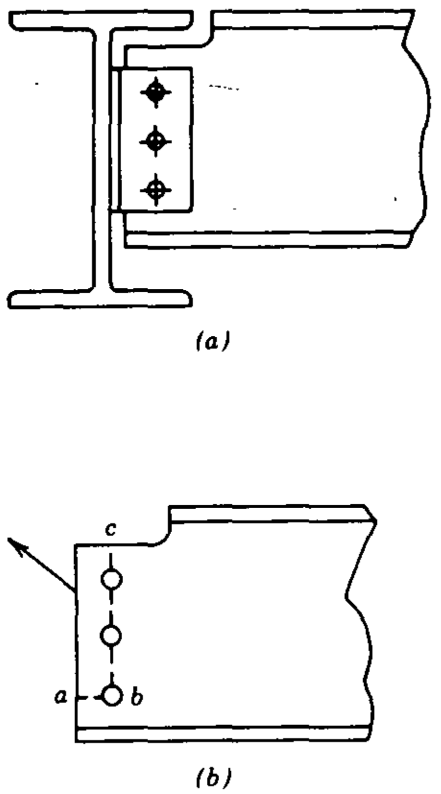


Fig. 18.8. Effect of beam cope on failure. (*a*) Actual; (*b*) tear-out.

stresses in the plates adjacent to the bolts, proper end and edge distances), proportioning of the weld at the beam-to-beam or beam-to-column junction, and selection of a suitable framing plate. Usually, the latter requires only the selection of the plate thickness; the other dimensions are controlled by the requirements for weld length and by the number of bolts and their spacing. If an appreciable portion of the end plate is unsupported, buckling of the plate should be investigated.

Richard and his coworkers conducted an extensive analytical and experimental

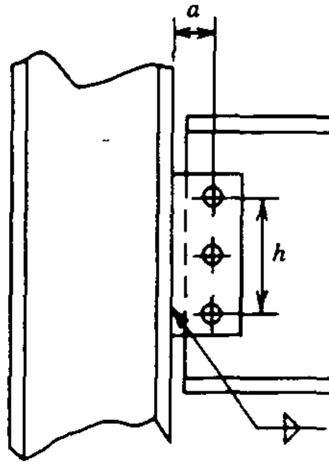


Fig. 18.9. Single plate framing connection.

study of single plate framing connections, including an analysis of test results by others.[18.20,18.21] In order that ductility of the connection be ensured, they recommend that bolt shear or transverse tension tearing of the plate in line with the bolt (see Fig. 5.33*a*) not be permitted as failure modes. In other words, bearing deformations in the plate or in the beam web should be used as the principal mechanism to produce ductility.

Richard *et al.* recommend that the ratio L/d (see Fig. 5.35*c*) be at least 2.0. This is consistent with the recommendations given in Chapter 5 and will ensure that splitting-type failure will not occur. Next, in order to establish high bearing stresses and, therefore, relatively large bearing deformations before the maximum force is reached, it is recommended that the thickness of the thinner plate element (beam web or framing plate) not exceed about 50% of the bolt diameter when A325 bolts are used or about 70% of the bolt diameter when A490 bolts are used.

For the case of a uniformly loaded beam, and using the results of about 1500 beam line analyses of various beam sizes, bolt diameters, and bolt arrangements, Richard *et al.* established a ratio $(e/h)_{\text{ref.}}$ for beam span to depth ratios equal to or greater than 6 as follows:

$$(e/h)_{\text{ref.}} = 0.035 \text{ (beam span/beam depth)} \tag{18.1}$$

A modified e/h ratio is then calculated as follows:

$$\frac{e}{h} = \left(\frac{e}{h}\right)_{\text{ref.}} \left(\frac{n}{N}\right) \left(\frac{S_{\text{ref.}}}{S}\right)^{0.4} \tag{18.2}$$

where e is the connection eccentricity and h is the depth of the connection between extreme fasteners, that is, $h = (n - 1) \times p$, using

n = number of bolts per vertical line
p = bolt pitch

The other terms in Eq. 18.2 are

N = a numerical coefficient, to be taken as 5 for $\frac{3}{4}$-in. and $\frac{7}{8}$-in. dia. bolts and 7 for 1-in. dia. bolts
S = section modulus of the beam
$S_{\text{ref.}}$ = a numerical modifier, to be taken as 100 for $\frac{3}{4}$-in. dia. bolts, 175 for $\frac{7}{8}$-in. dia. bolts, and 450 for 1-in. dia. bolts.

(Modifications to accommodate cases of concentrated loadings are given in Ref. 18.20.)

The moment at the bolt line is given by

$$M = Ve \tag{18.3}$$

and the moment at the weld line is given by

$$M = V(e + a) \tag{18.4}$$

where V is the shear force at the end of the beam and a is the distance from the weld line to the fastener line (see Fig. 18.9). The bolts, which had been selected by trial, can now be checked according to the procedure given in Chapter 13. The capacity of the weld and the plate itself can also be checked against the forces identified herein.

The procedure outlined above is believed to be satisfactory for single lines of bolts, either A325 or A490, using connected material with a yield strength of up to about 50 ksi. The use of more than a single line of bolts or the use of deep connections will be self-defeating. These arrangements will inevitably be stiffer than desirable for a flexible end connection. Design rules are also available for the case when A307 bolts are used.[18.21]

The upper angle in a seat connection (see Fig. 18.3) is mainly used to provide lateral stability for the beam. This component of the joint is not considered as load-carrying. The total shear force is assumed to be transmitted to the column by shear on the fasteners in the seat angle. The thickness of the seat angle is governed by critical bending stress on the outstanding leg. The usual practice is to consider the stress at the toe of the fillet of the outstanding leg. The required angle thickness is determined from the bending moment at that section. The reaction is assumed to act at the midpoint of the bearing length.[13.11]

18.3.2 Semi-Rigid Connections

There has been relatively little experimental work explicitly directed toward an understanding of the strength and deformation characteristics of semi-rigid connections. Most attention, particularly in the modern era, has been directed toward connections designed to be either flexible or rigid, with the recognition that neither of these ideals is exactly attainable. As was noted in Subsection 18.3.1, there has been a good deal of attention paid to the effect of all types of connections—flexible, semi-rigid, and rigid—upon the column strength.[18.23–18.27] The only type of semi-rigid connection that will be discussed in this section is the combined web-seat angle arrangement shown in Fig. 18.1*b*.

A combination web angle and seat angle connection results in significant increases in the joint restraint characteristics. Depending on the dimensions of the joint components and the loading conditions, these combination joints are sufficiently stiff to result in a substantial reduction in the midspan moment of a beam.[18.1]

Little experimental evidence is available on the load versus deformation behavior and load partition for this type of connection.[18.1] Since the behavior of the connections is complex and because of the lack of experimental data, a simplified, conservative approach is used for design. Current practice is to assume that the web angles will carry the shear. Thick top and bottom angles are used to transfer the end moment of the beam. Connections designed on the basis of these assumptions have provided satisfactory performance.

The design procedure for a shear connection is identical to that used for the web angle connection discussed in Subsection 18.3.1. The angles connecting the beam flanges to the column in the semi-rigid connection are considered to be load-carrying components; this was not the case for seat angle connections. Both angles are subjected to bending forces. However, the angle that connects the beam tension flange to the column flange is the critical one. A typical deformation condition for the tension angle is shown in Fig. 18.10*b*. Depending on the stiffness of the angle, prying forces may develop near the toe of the outstanding leg. Therefore, it is desirable to consider the influence of prying forces on the bending stress in the angle and the fastener tension. For analysis, the angle can be assumed to act like a T-stub connected to a rigid base and loaded in tension. This provides a conservative design because it assumes the angle to be fastened to a rigid base. Since the angle is fastened to a column flange, the decreased stiffness actually tends to relieve part of the restraint supplied by the angle. In general, the forces developed in a semi-rigid connection cannot be approximated in a reasonable way unless a test is conducted. This permits the stiffness and distribution of the forces in the connection to be evaluated.

The moment capacity of the connection is limited by the number of fasteners that can be placed in a single transverse line in the vertical leg of the angle connecting the tension flange to the column flanges. Because of deformation of the column flange (see Fig. 18.11), only the first fasteners on each side of the beam web may be fully effective in transferring the forces. Stiffening of the column flanges may be required unless they are at least as thick as the angle.

18.3.3 Rigid Connections

Replacing the angles of a combined web-seat angle connection (see Fig. 18.1*b*) with structural T-sections results in a connection with significantly increased mo-

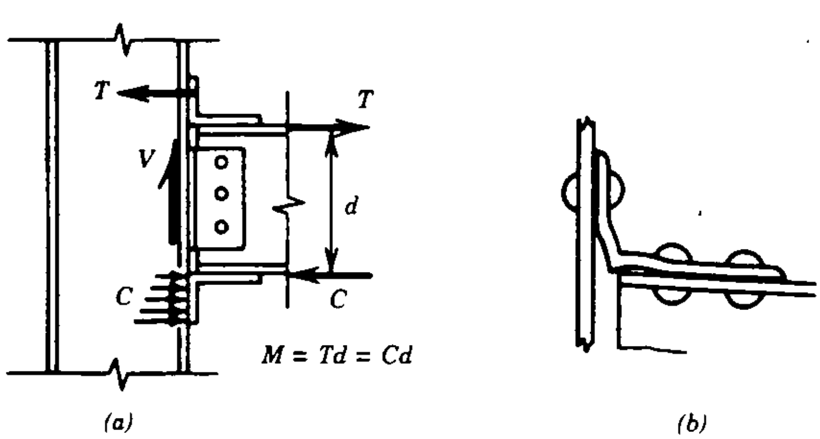


Fig. 18.10. Assumed behavior of semi-rigid connection.

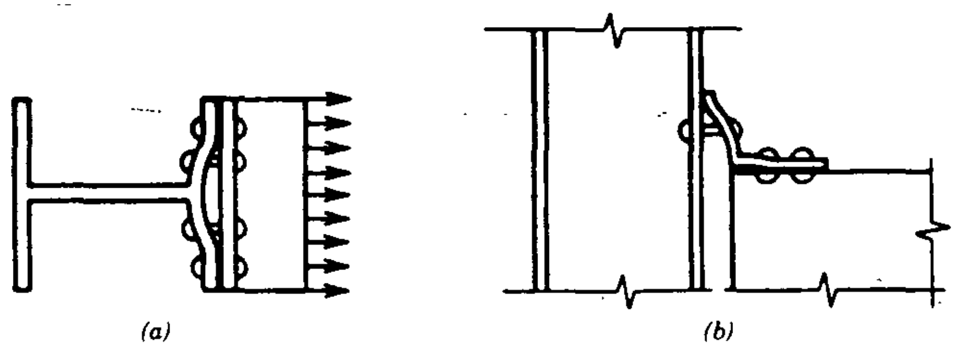


Fig. 18.11. Influence of deformations on fastener elongations.

ment resistance. Such a connection (see Fig. 18.1c) provides a rigid joint with high rotational resistance. The increase in rotational resistance is provided by the symmetrically loaded T-sections. Unlike angle connections, which are connected to the column flanges by two or more fasteners on one line, the T-section allows two or more fasteners to be used effectively on two lines to transfer the tensile forces that result from the applied moment. This results in an increase in moment capacity and joint stiffness. Since the T-sections are symmetrically loaded, they do not permit as much deformation to occur as compared with eccentrically loaded angles (see Fig. 18.3).

The design of the T-stub connection utilizes assumptions similar to those used for combined web-seat angle connections. The flange connection is assumed to transfer the moment, and the shear force is transferred by the web connection. Tests were carried out on connections of this type to evaluate the validity of these assumptions,[16.1,16.2] and typical test results are illustrated in Fig. 18.12. The effect of beam shear and the presence of the web angles on the behavior of the flange connections was investigated. In addition, these tests yielded valuable information on the rotation capacity of these connections.

The test results indicated that the behavior of the bolts connecting the T-stubs to the beam flanges was similar to the behavior observed in simulated flange plate splice tests.[16.2] The connection strength exceeded the plastic moment of the gross cross-sectional area of the beam, despite the presence of the holes in the flanges. Substantial rotational capacity was attained (see Fig. 18.12) when premature failure of the joint components was prevented. It was further concluded that the beam shear had no significant effect on the performance of the connection. The shear was largely carried by friction between the T-stubs and the column flanges. There was very little difference in bolt tension in the individual bolts connecting the tension T-stub, regardless of the magnitude of the prying forces.[16.2]

The test results generally supported the assumptions made in design. Although some shear can be transferred by the web of the T-stub, web angles are needed to assist with the shear transfer. This is particularly true if large shear forces exist.

In current (1987) steel fabrication practice, it is probably more common to use

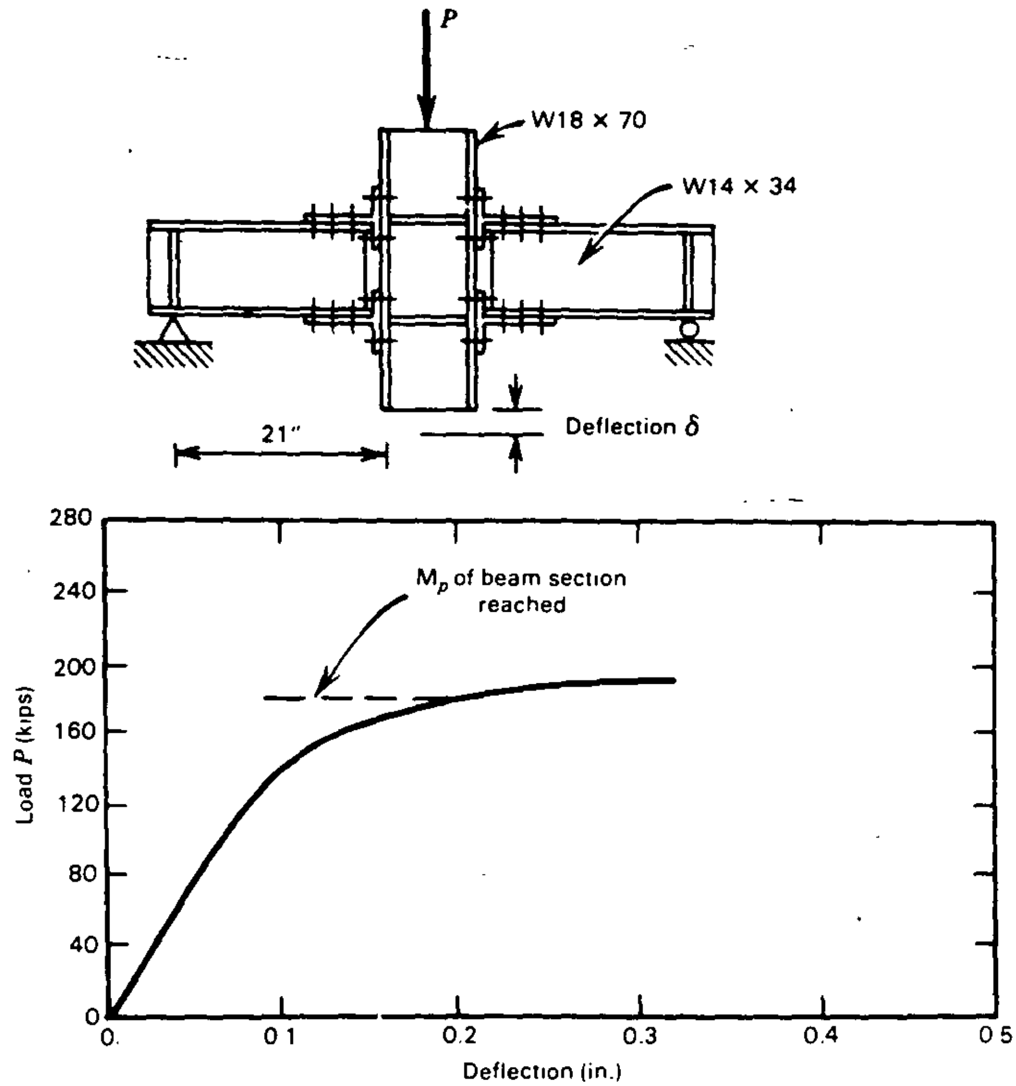


Fig. 18.12. Load versus deflection curve for a T-stub connection (Ref. 16.1).

a flat plate, groove-welded to the column flange, in place of the T-stub. This produces a simpler and more compact connection. The groove weld can be made in the shop and the bolts between the plate and the flange of the beam installed in the field. Web framing angles on one or both sides of the web or shear plates can be used to transfer the shear between the beam and the column. A single plate welded framing connection on one side of the web is the most common method used to transfer the shear between the beam and the column. The flange connections prevent the large rotations experienced by the single plate connections in simple beams.

Chen and his co-workers conducted a series of tests on various types of bolted beam-to-column moment connections[18.8, 18.9] and compared them to fully welded connections.[18.29] (In all cases, the flange plates were groove-welded to the columns.) The results that will be discussed here are only those in which the connection was made to the column flanges; tests were also conducted on beam-to-column web connections.[18.10]

Specimens were designed using the assumption that the flanges carried all of the moment and the web carried all of the shear. (One test was carried out on a connection that had no connection between the beam web and the column flange, that is, the groove welds at the beam flange level were expected to transfer both shear and moment. This connection exhibited neither adequate strength nor ductility.) The bolted parts of two of the specimens tested were designed as bearing-type connections, and a third specimen used a slip-resistant connection. A fourth test used a stiffened beam seat in addition to the flange and web details described herein; it will not be discussed.

Figure 18.13 shows the behavior of these connections.[18.8] In Fig. 18.13*a* a "fully bolted" connection, C7, is compared with a fully welded, but otherwise comparable, connection. Fully bolted means that the web shear plate and the flange connection plates were bolted to the beam. For this specimen, the bolts were designed as bearing-type. A490 bolts of 1-in. dia. installed in $1\frac{1}{16}$-in. dia. holes were used in both the flange and the web connections. Two responses representing theoretical cases are shown: one includes strain-hardening and the other does not. The fully welded connection follows the theoretical prediction that includes strain-hardening quite closely, except that there is a rounded knee, as would be expected, due to yielding. The response curve of the bolted connection shows a change in slope at about 150 kips, probably due to slip of fasteners as well as yielding. Both the ultimate strength and the rotational capacity of the bolted connection were greater than that of the fully welded connection.

In Fig. 18.13*b* the behavior of a fully welded connection and two otherwise comparable bolted connections are compared. One of the bolted connections used a bearing-type design (C9), and the other used a slip-resistant design (C8). In Specimen C9, A490 bolts of 1-in. dia. were installed in $1\frac{1}{16}$-in. dia. holes to connect the flange plate to the beam flange. The web connection was made using $\frac{3}{4}$-in. dia. A325 bolts. Slotted holes $1\frac{7}{8}$ in. long were used horizontally in the single web shear plate. A covering bar was used, in accordance with the RCSC specification. The fastener and hole arrangement for specimen C8 was similar, except that $1\frac{1}{4}$-in. dia. holes were used for the 1-in. dia. A490 bolts.

Figure 18.13*b* shows that the response curve that uses strain-hardening once again provides a good representation of the actual response of the fully-welded connection. As with specimen C7, this bearing-type bolted connection (C9) also shows two distinct slopes in the initial region. The slip-resistant connection (C8) followed the theoretical curve more closely and did not show any distinct change in slope.

The slip that occurred in the tests of these bolted connections was a series of small individual slips that took place in the second-slope region of the load versus deflection curves. Shown in each of Fig. 18.13*a* and *b* is a horizontal line illustrating the effect of the horizontal slips upon the vertical deflection. This horizontal

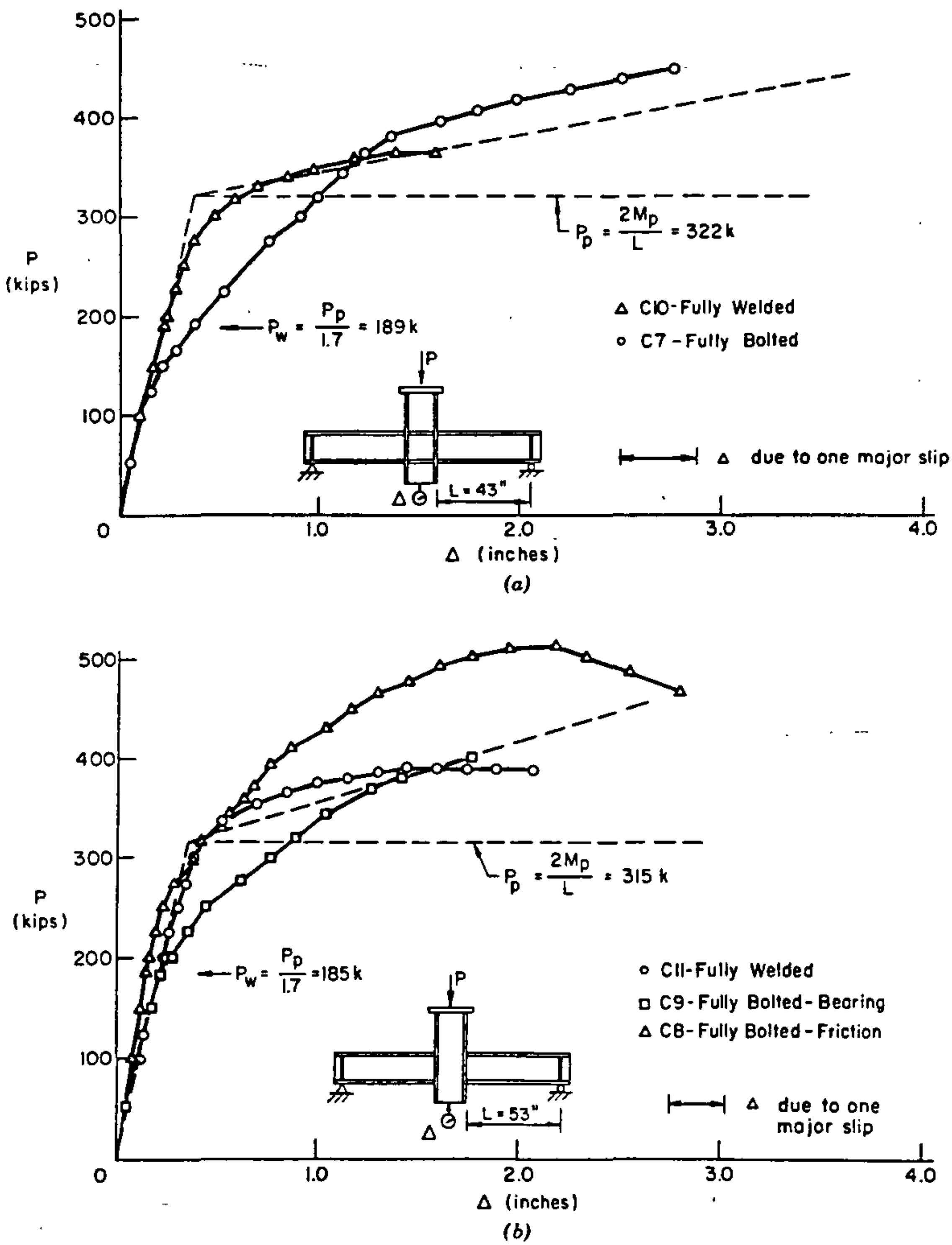


Fig. 18.13. Behavior of fully welded and fully bolted rigid connections (Ref. 18.8).

line represents the effect that would have occurred had there been one major slip, rather than the many minor slips that actually took place.

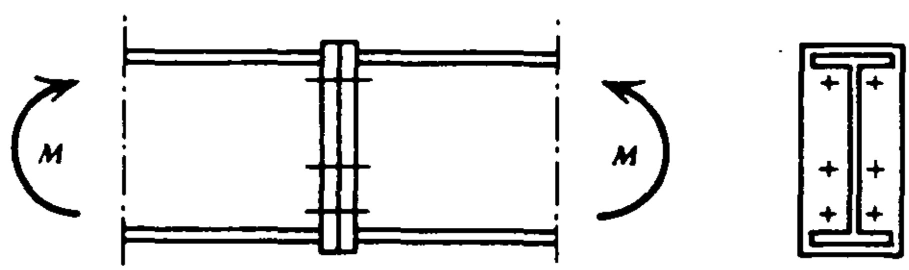
Whether the load at which the second-slope of the load versus deflection response curve will be above or below the working load of the connection depends on the particular details involved. Standig *et al.*[18.8] have shown that this load level can be predicted with reasonable accuracy using the principles outlined in this *Guide*. The connection of the beam flange to the moment plate is idealized as a slip-resistant lap joint, and the slip load obtained by this analysis then can be compared with the theoretical force in the flange plate (at the first line of bolts from the free end), assuming that all moment is carried by the beam flanges.

All three bolts specimens tested exhibited adequate rotational capacity as compared with the fully welded joints.

End plates welded to the beam cross-section have been used in beam-to-column connections and butt-type beam splices (see Chapter 16). Two types of end plates are used, as shown in Fig. 18.14. In one type the fasteners are placed only between the beam flanges, and in the other type the end plate is extended beyond the tension flange and fasteners are centered around the flange. Sometimes, this flange extension is stiffened.[18.22]

The exact load transfer in this type of connection is complex. The shear forces acting on the connection are transferred by frictional resistance and/or by shear on the fasteners. The fasteners are also subjected to tensile loads that resist the bending moment. The forces in the bolts change under the applied loads and are dependent on the magnitude of the initial bolt tension.

The end-plate connection is an economical way of fastening beams to column



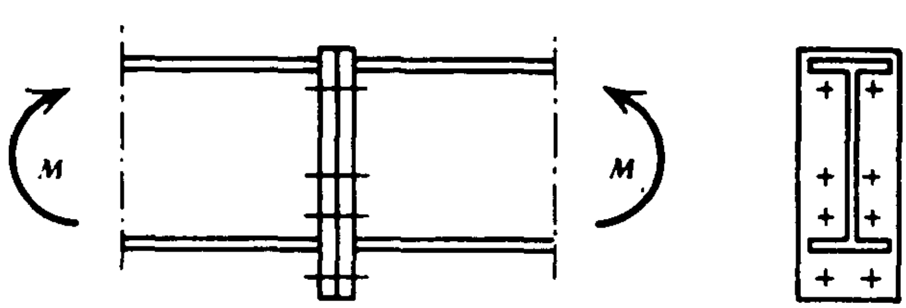


Fig. 18.14. End-plate types. *Note.* Connect end plates to beams with enough weld to develop full bending strength of beam.

flanges, and a number of studies, both analytical and experimental, have been carried out in recent years.[16.1, 16.2, 18.4–18.6, 18.11, 18.14–18.16] The problem is a complex one because the end-plate connection is highly indeterminate. The most recent analytical studies have used the finite element method to study the distribution of internal forces. Because of the relatively confined physical system involved, experimental studies have generally involved only the measurement of the moment versus rotation response of the connection and, in some cases, the measurement of bolt forces.

The end-plate connection has a number of similarities to the T-stub connection just discussed. It is evident on the basis of that examination that the following potential critical regions or effects will have to be examined for an end plate connection:

1. Buckling, crippling, or yielding of the column web opposite the beam flange that delivers the compressive force
2. Yielding of the column flange (or excessive deformation of the column flange) opposite the beam flange that delivers the tensile force
3. Yielding or fracture of the connectors (welds or bolts)
4. Failure of the end plate itself due to yielding or fracture
5. Yielding due to shear in the panel zone of the column web

This list is intended to cover the situation wherein a beam or girder is framed into the flange of a column. The situation will be somewhat different for a beam or girder splice that uses an end-plate connection. End-plate connections between beams or girders and a column web are not generally used.

A number of experimental studies have been made to examine the load versus deformation behavior of this type of connection and to develop design rules.[16.1, 16.2, 18.4–18.6, 18.11] These studies have indicated that the bolts mainly effective in resisting the tension flange force are those adjacent to the tension flange. This is illustrated in Fig. 18.15 where the bolt forces in a moment splice end plate connection are plotted as a function of the applied load.[16.2] The measured bolt forces were all similar at the start of the test. As load was applied, the forces in the bolts centered about the tension flange (levels 3 and 4) increased from about 30 kips to about 48 kips. The forces in the bolts at level 2, close to the neutral axis of the beam, showed no appreciable change as the load was applied, and the bolts on the compression side, level 1, showed a decrease in force from about 28 kips to 16 kips.

It was concluded from Fig. 18.15 that the variation of the force in the several rows of a bolt pattern depends primarily on the stiffness of the end plate and whether the plate yields before fracture of the critical fasteners takes place. At first, strains will increase in proportion to the distance of the fasteners from the compression flange. Because of the strain gradient, differences in bolt loads result, but these differences will decrease as plastic deformations of the bolt develop. If the bolts have sufficient ductility, all bolts in the tension region will develop the same

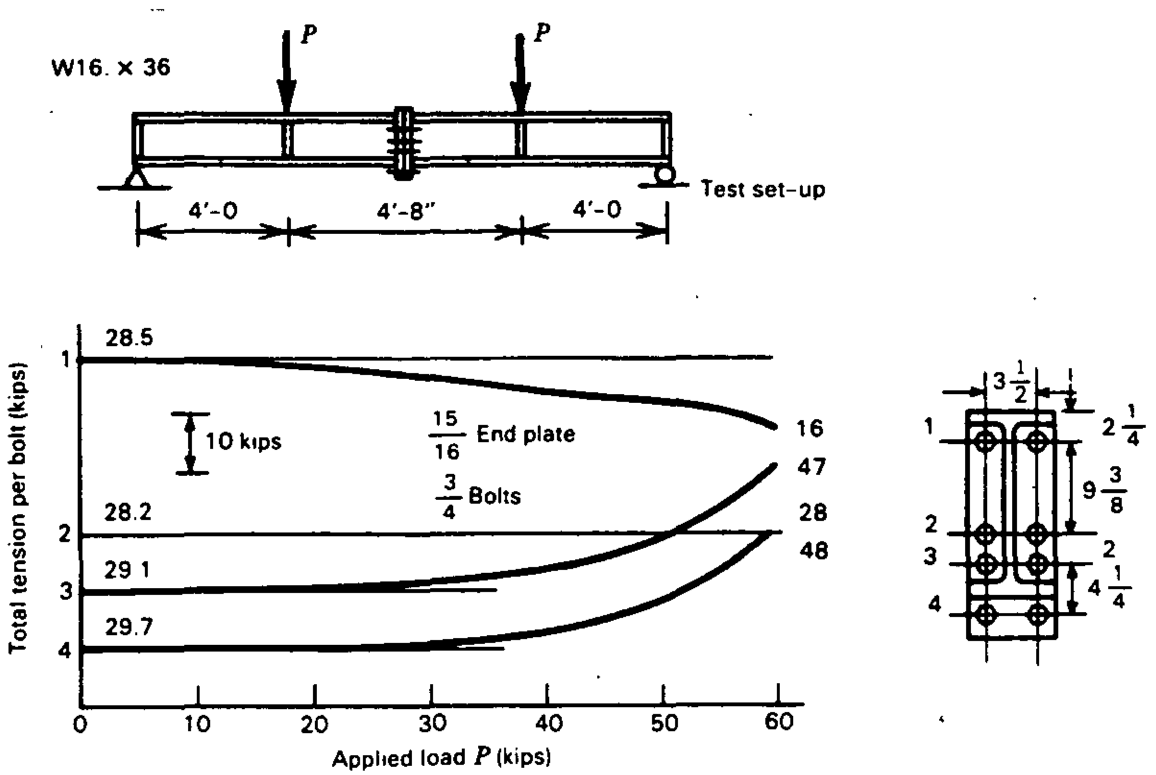


Fig. 18.15. Bolt force versus applied load (Ref. 16.2).

capacity at ultimate load.[16.2] Unless it is sufficiently thick, the end plate will yield and a linear strain distribution will not occur. This is apparent in Fig. 18.16, which shows an end-plate connection after failure.[18.5] The pressure distribution at the interface of the end plate and the column is shown in Fig. 18.17 and indicates that prying forces were developed at the edges of the end plate near the tension flange.[18.5]

Test results have shown that the bolts that are effective in resisting the moment for flexible end-plate connections are those adjacent to the tension flange. The connection is flexible if prying forces are developed at the edge of the end plate in the tension region. If a connection is designed such that no prying forces are developed, a linear strain distribution among the fastener rows can be assumed, and the inner fasteners may contribute to the capacity of the connection. The ultimate moment resistance of the connection is the summation of the products of the effective fastener loads and their respective distance from the center of rotation. At the ultimate load the center of the rotation is near the centerline of the compression flange. This is compatible with existing experimental observations.[16.2, 18.4, 18.5]

The design of end-plate connections requires that the connection provide adequate strength, that is, both the size and number of bolts and the end-plate thickness must be satisfactory, and that there be adequate rotation capacity such that the desired moment capacity can be attained. In addition, the connection must be stiff enough so that permanent deformations are not introduced under working loads.

Fig. 18.16. End-plate connection after failure. (Courtesy of University of Sheffield.)

On the beam side of the end-plate connection, the weld between the end plate and the beam must be proportioned. On the column side of the connection, the delivery of the shear and moment from the beam must be accomplished with strength and stability requirements being satisfied.

Krishnamurthy has reported the results of an extensive analytical study of end-plate connection behavior.[18.11] Two- and three-dimensional finite element analyses of many T-hanger and end-plate connections were carried out. Krishnamurthy noted that there can be significant differences in stiffness between the assembly at the bolt line and at the face of the end plate-to-beam flange junction (the "load line"), and he also observed that there is a significant difference in stiffness in the end plate in the region where it is extended beyond the beam flange and in the region of the end-plate between the beam flanges. As a consequence of these and other

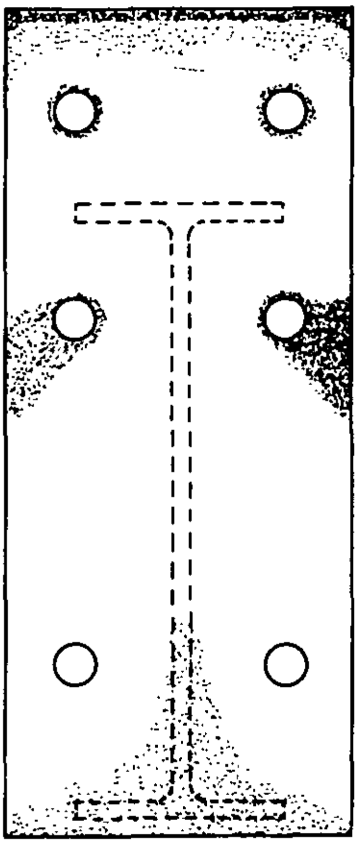


Fig. 18.17. Pressure distribution at interface as recorded on interposed paper backed up by carbon paper (Ref. 18.5).

factors, Krishnamurthy proposed a method of analysis based on a traditional approach but modified on the basis of his analytical studies to account for these effects.

The analysis of an end-plate connection, a highly redundant system, must be subjected to physical testing as well as to analytical testing. In Ref. 18.11 Krishnamurthy reported the results of 10 tests of end-plate connections (9 are reported in the main body of the report, and 1 in an appendix). In one test, the moment attained was well below the plastic moment capacity of the section (58%) because torsional buckling occurred. In two others, the presence of a slender web meant that local buckling occurred before the moment capacity of the section could be reached. Thus, it can be observed that there were seven tests that could be used to substantiate the method proposed by Krishnamurthy. For these tests, Krishnamurthy reported that the ratio p_e/d ("effective distance" from the bolt line to the load line/the bolt diameter) ranged from 0.8 to 1.4. Dismissing the results of certain tests as invalid as a suitable measure of the end-plate behavior (see above), this ratio only extends over the range of 0.8–1.1. Indeed, the mean value of the ratio is 1.0, with a standard deviation of 0.1. Furthermore, the bolt diameters in these tests were about two times the thickness of the end plate. Thus, the situation is one in which the physical tests represent the case of relatively large bolts connecting a relatively thin end plate, and wherein the bolt is located very close to the beam flange-to-end plate junction. (In most cases, the Krishnamurthy test specimens used a bolt arrangement that was at or slightly above the limit considered to be a practical minimum.[18.30]) It could be expected that these tests would not result in any bolt failures, and none were reported.

The procedure for the design of end plates and their fasteners as recommended by Krishnamurthy[18.11] and as adopted in the eighth edition of the AISC *Steel Construction Manual*[18.31] is as follows:

1. Assume that the beam flanges carry all of the moment, and calculate the force in each flange accordingly.
2. Determine the size and number of bolts required to transfer the flange force. No allowance is made for additional forces (above the nominal values) due to prying action.
3. Calculate an effective distance (p_e) between the line of action of the bolt line and the "load line" at the end plate-to-beam flange junction (see Fig. 18.18) as

$$p_e = p_f - 0.25d_b - w_t \tag{18.5}$$

4. Calculate the moment in the end plate (M_t) at the location of the load line, using the bolt forces and the effective distance calculated above. The load line location is assumed to be a location of fixed end moment.
5. Compute a modified moment at this location as

$$M_d = \alpha_m M_t$$

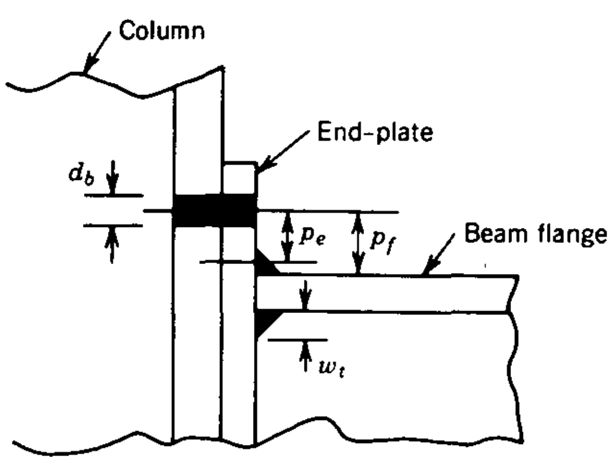


Fig. 18.18. Nomenclature for end plate design.

where

$$\alpha_m = 1.29 \left(\frac{F_y}{F_{bu}}\right)^{0.4} \left(\frac{F_{bt}}{F_p}\right)^{0.5} \left(\frac{b_f}{b_s}\right)^{0.5} \left(\frac{A_f}{A_w}\right)^{0.32} \left(\frac{p_e}{d_b}\right)^{0.25} \tag{18.6}$$

and

F_y = yield strength of beam and plate material
F_{bu} = ultimate tensile strength of bolt material
F_{bt} = allowable tensile strength of bolt
F_b = allowable bending stress in end plate
b_f = width of beam flange
b_s = width of end plate
A_f = area of tension flange of beam
A_w = area of web of beam
p_e = effective bolt distance (see Eq. 18.5)
p_f = distance of bolt from face of beam flange
d_b = bolt diameter
w_t = throat size of fillet weld
w_s = size of fillet weld.

6. Calculate the end plate thickness (t_s) using simple bending theory.
7. Compute an effective maximum end plate width as

$$b_e = b_f + 2w_s + t_s \tag{18.7}$$

If b_e is less than b_s, recalculate t_s using the value of b_e in place of b_s.
8. Check the shear stress in the plate.

Design aids are available that simplify the calculations required.[18.31] The method proposed by Krishnamurthy and adopted by the AISC will result in thinner end plates than designers have been accustomed to in the past. Agerskov[18.32] has commented that bolt prying forces are likely to be present in end-plate connections and should not be ignored, and that, with the thinner end plates, deformations between yield moment and the plastic moment levels might become excessive. Similarly, McGuire[18.33] suggests that there might be a degradation of bolt clamping forces even under working load levels because of the thin end-plates. Mann and Morris, who have analyzed end-plate connections using a yield line approach, recommend that an increase of $33\frac{1}{3}\%$ over normal bolt load levels be applied in recognition of bolt prying forces.[18.14] In the face of these criticisms, it must be noted that light end-plate connections have been used successfully in the industrialized metal building industry for many years.[18.34]

The design procedure developed by Krishnamurthy appears to give satisfactory results within the parameters examined, especially the use of bolts that have a diameter that is large relative to the end-plate thickness and that are placed (effective distance p_e) no further than about one bolt diameter from the load line at the end plate-to-beam flange junction. When many repetitions of a connection type are required, this procedure will be advantageous because it reduces material cost. On the other hand, when the number of connections is not large, a reduction in end-plate thickness may not be significant since the labor component will not be much reduced over that for a thicker end plate. In these cases, the designer and fabricator might prefer a more conservative approach since it provides more leeway in detailing the connection and, thereby, in ease of fabrication and erection. The bolts and end plate adjacent to the tension flange can be conservatively designed by assuming that they are equivalent to a T-stub connection loaded in tension. Design procedures for this idealization are given in Chapter 17.

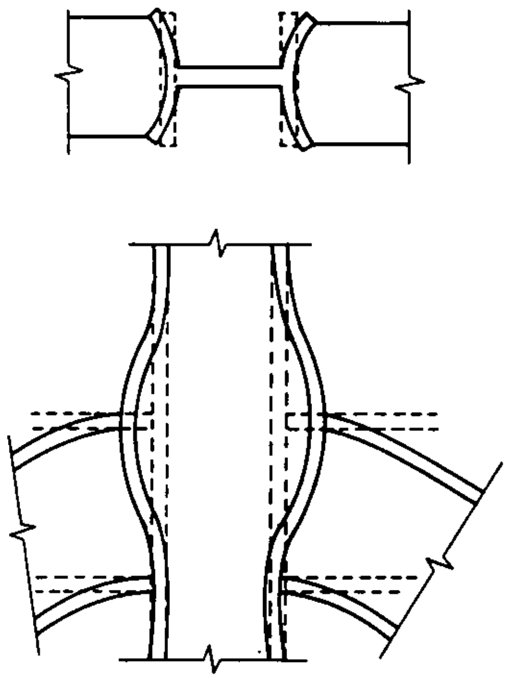
Although the primary transfer of shear is concentrated near the compression side of the joint, it can be conservatively assumed that all bolts carry an equal part of the shear load. Hence, the fasteners in an end-plate connection are subjected to combined shear and tension. The magnitude of initial clamping force does not influence the ultimate strength of the connection; it does influence the shear resistance of slip-resistant joints.

When end plates do not extend beyond the tension flange, their behavior is not well known because available data are not extensive. In general, these types of end-plate connections are less efficient and require thicker end plates. Reference 16.2 suggested that end plates that do not extend beyond the tension flange should be proportioned to resist a moment equal to the product of the beam flange force and the distance between the center of the beam flange and the nearest row of bolts. Plate thicknesses determined in this manner appear to provide a linear variation in fastener strain throughout the connection depth. Additional test data are needed to verify this suggested method for a range of sizes.

All of the foregoing discussion on end-plate connections has assumed that the bolts adjacent to the tension flange of the beam will be arranged in two lines, one just above the beam flange and one just below. If this arrangement does not provide a sufficient number of fasteners, it may be necessary to use more than one bolt line. Extension of the end plate above the tension flange of the beam to accommodate two bolt lines is practical only if a very thick end plate is used or if the end-plate extension is stiffened. Work on stiffened end plates has been reported by Murray and Kukreti.[18.22]

18.4 STIFFENER REQUIREMENTS FOR BOLTED BEAM-TO-COLUMN CONNECTIONS

The full capacity of a moment-resisting beam-to-column connection can only be developed if the column does not exhibit premature failure. The column is subjected to highly localized forces resulting from the applied moments and can deform as shown schematically in Fig. 18.19*a*. Excessive deformations of connected parts should be avoided. There are two major effects of the beam flange forces that have



(a)

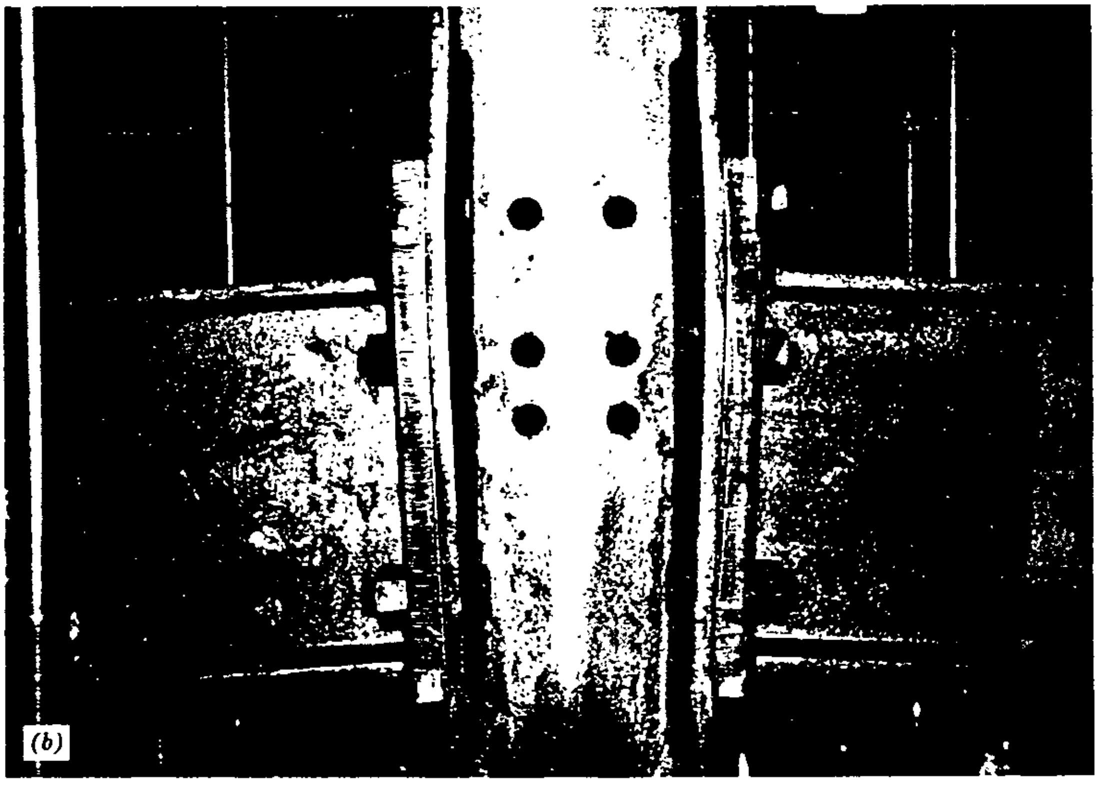


Fig. 18.19. Deformation of column in moment resistant connection. (*a*) Distortion of unstiffened column. (*b*) Web crippling in beam-to-column connection. (Courtesy of British Steel Corp.)

to be examined because they may result in excessive deformations. On the compression side of the beam, crippling or overall buckling of the column web can occur. On the tension side, excessive yielding and distortion may result in fracture of the column web or bolts. Web buckling is illustrated in Fig. 18.19*b* where an end-plate connection at ultimate load is shown. Because of the lack of stiffening in the compression region, the column web buckled and the connection could not develop the plastic moment capacity of the beam.[18.4]

Several investigators[18.4–18.6] have examined the stiffening requirements for bolted beam-to-column connections. Many joint geometries and boundary conditions exist; the problem is therefore extremely complex and no satisfactory general design approach is possible. Often the requirements developed for stiffening welded beam-to-column connections are used.[18.35–18.37] Since the concentrated forces are more localized in welded connections, application of the rules developed for welded connections to bolted connections results in a conservative design for the same moment capacity.

Standig *et al.*[18.8] and Huang *et al.*[18.9] have confirmed the adequacy of this approach for stiffening bolted connections. It is noted, however, that a bolted moment connection can have an actual moment capacity that is considerably larger than an all-welded connection designed for the same conditions. If advantage is to be taken of this increased capacity, stiffening requirements might require modification. Pending further research, criteria based in part on the requirements used for welded beam-to-column connections are reasonable.

The requirements for stiffening of the column are summarized as follows. As proposed in Ref. 18.35, the compression flange force on the column is assumed to be distributed on a 2.5:1 slope from the point of contact to the column k-line (see Fig. 18.20). If the compression flange force is distributed to the column flange by either an end plate or a structural T-section, it can be assumed to be distributed over a region on the column face about twice as great as the beam flange thickness. Hence, the force in the beam flange is assumed to be resisted by a length of column web equal to $(Q + 5k_c)$, where Q is the sum of the beam flange thickness and twice the end-plate thickness (for the plate connection) or the web thickness of the T-stub and twice its flange thickness, and k_c is the column fillet depth. For equilibrium, the resistance of the effective area of the web must equal or exceed the applied concentrated force of the beam tension or compression flange. This yields the following condition;

$$\sigma_{yc} w_c (Q + 5k_c) \geq A_f \sigma_{yb} \tag{18.8}$$

where w_c is the thickness of the column web, and A_f is the flange area of the beam. The yield point of the column web is given by σ_{yc} and the yield point of the beam flange by σ_{yb}. If the column web resistance is less than provided by Eq. 18.8, stiffeners are required.

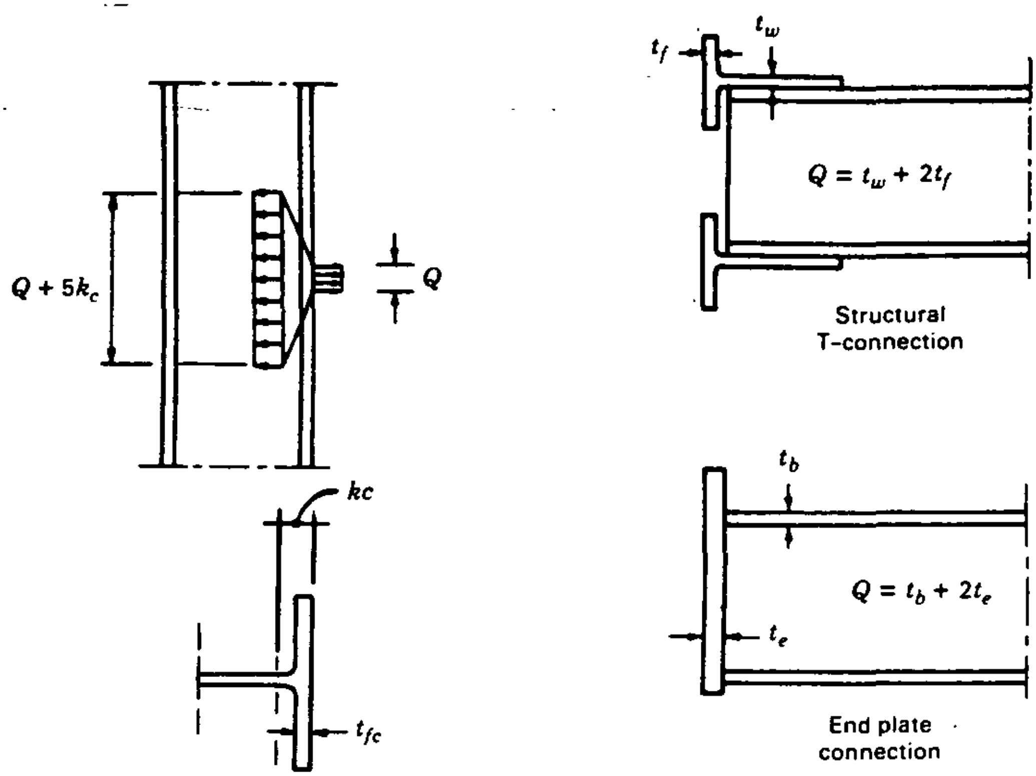


Fig. 18.20. Assumed distribution of compression flange force in bolted beam-to-column connections.

If flange splice plates are welded to the column on the compression or tension side of the beam, the provisions developed for welded connections are directly applicable.[18.35] The force from the compression flange is resisted by a length of the column web equal to ($t_s + 5k_c$), where t_s is the splice plate thickness.

An upper limit must be placed on this analysis of strength because, at some value of the column web slenderness, the region in compression will buckle. Chen and Oppenheim have suggested that a square panel ($d_c \times d_c$) can be used for the analysis.[18.36] Using plate buckling theory and the usual values for E and ν for steel, this upper limit can be expressed as

$$\frac{d_c}{w_c} \leq \frac{180}{\sqrt{\sigma_{yc}}} \tag{18.9}$$

If the actual web slenderness of the column exceeds the value given by Eq. 18.9, a capacity prediction based on test data is given by[18.37]

$$P = \frac{4100\, w_c^3 \sqrt{\sigma_{yc}}}{d_c} \tag{18.10}$$

For the tension flange, Ref. 18.35 has shown that the column flange provides adequate resistance against excessive deformations from the concentrated forces delivered by the tension splice plate if

$$t_{fc} \geq 0.4 \left(A_f \frac{\sigma_{yb}}{\sigma_{yc}} \right)^{1/2} \tag{18.11}$$

where t_{fc} is the column flange thickness. Tests of welded connections proportioned to these recommendations indicated that the connections were able to develop the full plastic moment of the beam.[18.9, 18.35]

If a T-section or an end plate is bolted to the column flange, the concentrated tension force is distributed into the column flange by the fasteners. The system of applied forces differs significantly from the case of the splice plate welded to the column. The application of Eq. 18.11 is likely to yield overly conservative results.

European practice is to use a yield line analysis to examine the requirements for column flange thickness when bolts are used to deliver the load from the tension flange of the beam.[18.16]

When column stiffeners are required, they should be proportioned to carry the excess between the beam flange force and the calculated resisting capacity of the column web or flange.

If a single beam frames into a column or if the moments from two beams at an interior connection differ by a large amount, the web of the column can be subjected to large shears. In such situations it may be necessary to provide shear stiffening in the form of diagonal stiffeners or doubler plates. Design of such stiffeners is treated in many design handbooks.[13.11, 15.1, 17.6]

18.5 DESIGN RECOMMENDATIONS

Depending on the anticipated behavior, bolted beam-to-column connections are designed either as slip-resistant or bearing-type joints. The design recommendations in Chapter 5 for fasteners in butt joints are also applicable to the design of bolted beam-to-column connections.

The bolts in an end-plate connection are subjected to combined tension and shear. The elliptical interaction curve for bolts subjected to combined loading conditions (see Eq. 4.8) can be used to examine the adequacy of the fasteners.

With the exception of end-plate connections, it can be assumed for design that the web connection or the seat angle transfers the shear component. Web shear connections should be designed as eccentrically loaded joints in accordance with the recommendations given in Chapter 13. The moment on a beam-to-column connection is transferred by structural components connected to the beam and column flanges. The recommendations given in Chapter 16 for beam and girder splices are applicable to the design of the beam flange connection. The tension connection between the beam flange and column flange is usually critical for design.

Prying forces should be considered for the design of the fasteners as well as joint components. The bolts and end plate adjacent to the tension flange can be treated as an equivalent tee stub connection, loaded in tension. Design recommendations for the T-stub connection are given in Chapter 17. Alternatively, the method recommended by Krishnamurthy for the design of end plates and their connectors can be used.

Special attention should be given to the bending stiffness of the column flanges to which the T-section or the end plate is fastened. The deformations of the column flanges and the T-section (end plate) may introduce prying forces (see Chapter 17), depending on their stiffnesses.

Stiffening the column may be required to prevent premature failure of a joint component due to column web crippling or column flange deformation. For connections with flange splice plates welded to the column, the requirements for welded connections can be applied.[2.11,18.35] If the compression flange force is transferred through an end plate or a T-section, Eq. 18.8 can be used to determine whether additional column stiffening is needed.

$$\sigma_{yc} w_c(Q + 5k_c) \geq A_f \sigma_{yb} \tag{18.8}$$

For slender webs the stability of the compression region may govern rather than strength alone. Reference 18.37 has suggested that the following relationship (see Eq. 18.10) be satisfied when $d_c/w_c > 180\sqrt{\sigma_{yc}}$

$$w_c^3 \leq \frac{\sigma_{yb}}{\sigma_{yc}} A_f \frac{d_c\sqrt{\sigma_{yc}}}{4100} \tag{18.12}$$

where d_c is the column web depth.

The flanges of the column must not deform excessively under the action of the concentrated flange tensile forces. If splice plates welded to the column are used, adequate flange resistance is provided by

$$t_{fc} \geq 0.4 \left(A_f \frac{\sigma_{yb}}{\sigma_{yc}}\right)^{1/2} \tag{18.11}$$

For bolted T-connections in tension (including end-plate connections), the use of Eq. 18.11 will be conservative. A yield line analysis can be used as an alternative.

When stiffeners are required, they must be proportioned to carry the difference between the concentrated force calculated to be in the beam flange and the calculated resistance of the column web. For stiffeners opposite the beam compression flange, the required stiffener area can be determined from equilibrium. This yields

$$\sigma_{ys} A_{st} = \sigma_{yb} A_f - w_c(Q + 5k_c)\sigma_{yc} \tag{18.13}$$

If $C_1 = \sigma_{yb}/\sigma_{yc}$ and $C_2 = \sigma_{yc}/\sigma_{ys}$, Eq. 18.13 can be expressed as:

$$A_{st} = [C_1 A_f - w_c(Q + 5k_c)]C_2 \tag{18.13a}$$

If Eq. 18.12 governs the column web thickness, the required stiffener area becomes

$$A_{st} = \left[C_1 A_f - \frac{4100 w_c^3}{d_c\sqrt{\sigma_{yc}}}\right]\sigma_{yc} \tag{18.14}$$

A comparable requirement can be developed for stiffeners opposite the tension flange by considering the needed additional flange area to be resisted by stiffeners. Equilibrium yields

$$\sigma_{ys} A_{st} = \sigma_{yb} A_f - \sigma_{yb} A_f' \tag{18.15}$$

where $A_f \sigma_{yb}$ is the actual beam flange tension force, and $A_f' \sigma_{yb}$ is the beam flange tension force that would not require stiffeners. This latter force can be estimated from Eq. 18.11 for the column flange thickness furnished. This yields

$$A_f' = \frac{\sigma_{yc}}{\sigma_{yb}} t_{fc}^2 \frac{100}{16} \cong 6 \frac{\sigma_{yc}}{\sigma_{yb}} t_{fc}^2$$

Substitution into Eq. 18.15 yields

$$\sigma_{ys} A_{st} = \sigma_{yb} A_f - 6\sigma_{yc} t_{fc}^2 \tag{18.15a}$$

Hence, the required stiffener area opposite the team tension flange becomes

$$A_{st} = [C_1 A_f - 6t_{fc}^2]C_2 \tag{18.15b}$$

As a practical requirement, if stiffeners are required opposite both the beam tension and compression flanges, they are generally made the same size.

The fastener shear stresses and the bearing stresses suggested in Chapter 5 were shown in Refs. 18.7, 18.9, and 18.36 to be fully applicable to beam-to-column connections.

REFERENCES

18.1 J. C. Rathbun, "Elastic Properties of Riveted Connections," *Transactions, ASCE*, Vol. 101, 1936.

18.2 W. H. Munse, W. G. Bell and E. Chesson, Jr., "Behavior of Riveted and Bolted

5.4 DESIGN RECOMMENDATION

5.4.1 Introduction

The mathematical model presented in Section 5.2 provides a reasonable prediction of joint behavior under either working loads (allowable stress design) or factored loads (load factor design). However, it is not suitable directly for design because of its complexity. The results of analyses carried out using this "exact" solution are used to form the basis for the design recommendations that follow. They are presented in such a way that they can be used in either working stress design or load factor design specifications.

Current design practice that is founded on an allowable stress format treats mechanically fastened bearing-type joints on the basis of allowable stresses acting on either the gross or net area of the member and on the average stresses in the fasteners. Most design specifications do recognize, however, that the assumption that each fastener carries an equal share of the load becomes less and less accurate as joint length increases. The accommodation for this effect is generally applied in a step-wise fashion (usually just one reduction in allowable shear stress with length), although many European specifications provide a linearly varying reduction with joint length. Specifications that use a load and resistance design format treat the design of the fasteners in the same way; that is, the average fastener load is generally applied, and the effect of joint length is recognized for longer joints.

In either case of allowable stress design or load factor design, once the member forces are known from a structural analysis, the required number of fasteners can be determined on the basis of the permissible shear stress for the fastener, including consideration for the effect of joint length. Hence, the load transmitted by a bolted joint with n fasteners and m possible shear planes per bolt through the bolt shank can be expressed as

$$P = mn\tau_b A_b \tag{5.21}$$

where τ_b represents the permissible shear stress on the fastener (allowable stress design or load factor design, as appropriate), and A_b represents the nominal bolt area. If the shear planes pass through the threaded parts of the bolt, Eq. 5.21 is modified to

$$P = 0.70mn\tau_b A_b \tag{5.22}$$

as discussed in Section 4.10.

The strength of a slip-resistant joint can be expressed in its most basic form, Eq. 5.2, in which a slip coefficient is used. Alternatively, although the bolts are not actually subjected to shearing forces, an equation such as Eq. 5.21 can be used. In this format, an equivalent permissible shear stress is calculated, but it must be remembered that the load is actually transferred by the frictional resistance on the faying surfaces.

Design criteria for connections can be based upon performance, strength, or both. In a slip-resistant joint, unsatisfactory behavior would result if a major slip occurred: a performance criterion. The function of the structure may be impaired due to misalignment or other unsatisfactory conditions that may result from the slip. However, most slip is minor and will not be detrimental to the performance of the joint. In these cases, strength is the factor that should govern the design; it is identified as the shear stress on the fastener, the bearing stress in the material adjacent to the fastener, or as the tensile stress on the net or gross cross-section of the member.

The ultimate capacity of both slip-resistant and bearing-type bolted joints is limited by failure of one or more components of the joint. Joint strength provides an upper bound for either joint type. Hence, in allowable stress design, the permissible strength of a slip-resistant joint can, at best, equal the capacity of an otherwise comparable bearing type connection. In other words, to design a slip-resistant joint, the slip resistance of the joint is determined on the basis of factors such as the surface condition, the bolt type, the tightening procedure, the number of bolts, and the number of slip planes. This slip resistance is then compared with the bolt shear capacity of the joint based upon the number of shear planes per bolt and their location (through the shank or through the threaded part of the bolt) and the number of bolts in the joint as well as the bolt quality. Of course, the smaller value of the shear strength and the slip resistance is governing.

In load factor design, the ultimate strength of the member or connection is checked against the effect of the factored loads. The factored load is determined by multiplying the working loads by a factor that is greater than 1.0. In addition, it is necessary for the member, joint, and structure as a whole to be "serviceable" at the working load level. This means that consideration must be given to control of deflections, deformation, and fatigue of the structure at its service or working load level.

To meet the requirements of load factor design, the ultimate strength of a bearing-type bolted joint is checked directly against the effect of the factored loads. Unless fatigue is a factor, the other requirements for serviceability are not operative since, by definition, any small slips that may occur are judged not to be detrimental.

On the other hand, a slip-resistant connection designed under load and resistance factor design must be checked under both service (working) load levels and factored load levels. The obvious requirement is that the connection not slip under working loads. In addition, however, it is still a requirement that the ultimate strength of the connection loads be checked under factored loads.

A connection that is subjected to fatigue loading must meet exactly the same requirements as those described for slip-resistant joints under either working stress design or load and resistance factor design, as appropriate. Of course, the governing permissible stress for the fatigue case is used to evaluate the resistance of the joint under the working loads.

5.4.2 Design Recommendations—Fasteners

i. Allowable Stress Design Bolted Joints. The balanced design concept has been used to develop design criteria for mechanically fastened joints in the past. This design philosophy results in wide variations in the factor of safety for the bolt because the ratio of the yield point to the tensile strength changes with various types of steel.[5.49] For example, the 1972 specifications[1.4] provided ratios of tensile strength to allowable tensile stress equal to 2.64, 2.48, and 2.00 for A36, A440, and A514 steels, respectively. Furthermore, the balanced design concept has no meaning when applied to long joints because the end fasteners may "unbutton" before the plate material can attain its full strength or before the interior bolts can be loaded up to their full strength. This "long joint" effect depends on the type of joint material as well as on the type of fastener.

All of these factors resulted in a variable factor of safety, as illustrated in Fig. 5.48. The factor of safety against failure of the joint is plotted as a function of joint length for several steels fastened with A325 bolts. An allowable shear stress of 22 ksi was used to proportion the fasteners. This was the allowable shear stress prescribed in the RCSC specification up to 1974. The allowable tensile stress on the net section of the joint was taken as 60% of the yield stress or 50% of the tensile strength of the plate material, whichever was smaller. It was apparent that a different approach is desirable; one that would provide both a rational method of determining the allowable stresses and a uniform, or at least a more consistent,

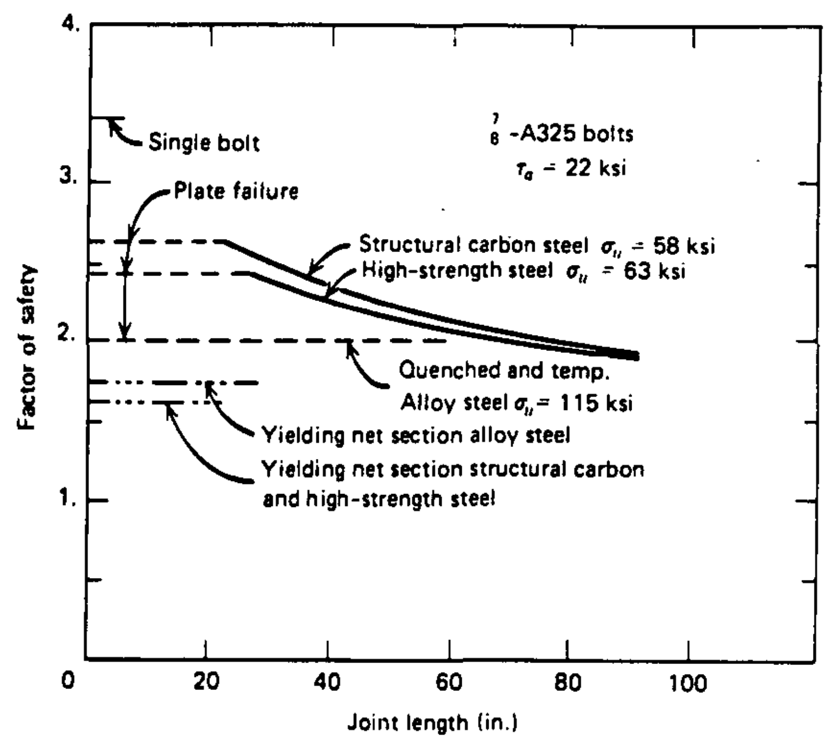


Fig. 5.48. Factor of safety versus joint length for A325 bolts.

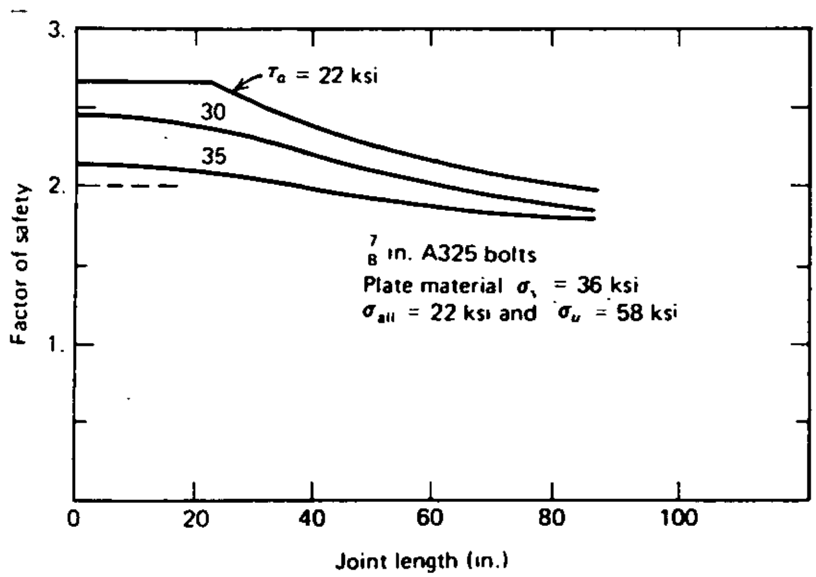


Fig. 5.49. Factor of safety for structural carbon steel joints fastened by A325 bolts.

factor of safety. It appeared that a more logical criterion to establish allowable stresses for the fasteners was to consider the fastener strength over the full range of joint behavior.

To determine the magnitude of the factor of safety deemed adequate for the fasteners, two aspects can be considered: (1) what the factor of safety has been in the past, and (2) what it ought to be. If past practice for riveted or bolted structural carbon steel joints is studied, the factor of safety against shear failure of the fastener is found to vary from approximately 3.3 for compact joints* to approximately 2.0 for joints with a length in excess of 50 in. This is illustrated in Fig. 5.48 for A325 bolts. The lower factor of safety for the longer joints was apparently adequate in the past. In fact, according to past practice, the largest and often most important joints have probably had the lowest factor of safety. Experience has shown that this factor of safety has provided a safe design condition. This indicated that a minimum factor of safety of 2.0 has been satisfactory; the same margin is also used for fasteners in tension. In addition, it was recognized that specified minimum mechanical properties of both the bolt and plate material were used to determine these lower bound conditions. Materials actually used as components of the joint usually provide strengths that exceed specified minimum properties. This results in an increased factor of safety. Finally, it can be noted that a minimum factor of safety equal to 2.0 for bolts in shear is not only in line with the factor of safety used for bolts in tension, but the same factor of safety against ultimate is also provided by quenched and tempered alloy steel tension members.[2.11]

*A compact joint is defined as a joint in which the average fastener shear stress at the ultimate load level is equal to, or almost equal to, the shear strength of a single fastener. The "unbuttoning" effect is negligible in these joints.

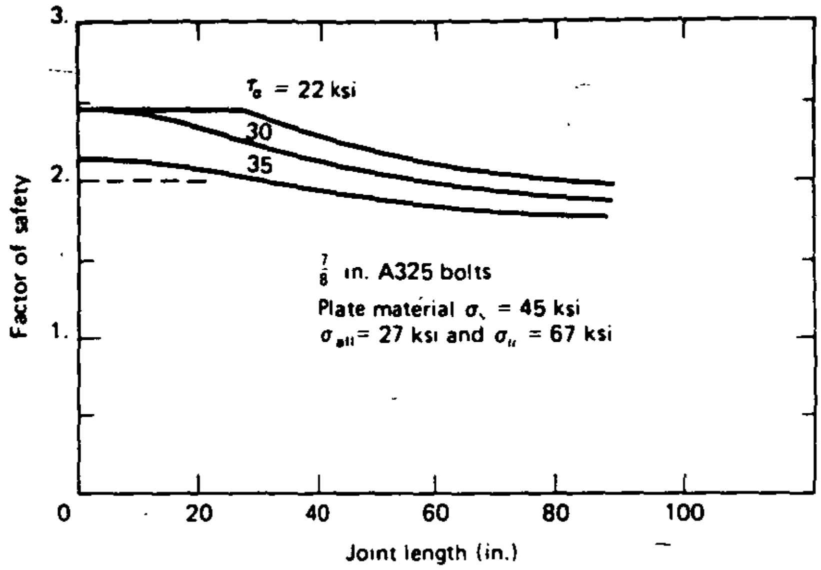


Fig. 5.50. Factor of safety for high-strength steel joints fastened by A325 bolts.

In Fig. 5.49 the factor of safety is plotted as a function of the joint length for different allowable shear stresses in $\frac{7}{8}$-in. dia. A325 bolts, installed in structural carbon steel with a yield stress of 36 ksi and a tensile strength of 58 ksi. Joint length is defined as the length required to transfer the load from the main plate into the splice plates. Hence, for a symmetric butt splice, the joint length is equal to half the total length of the lap plate. For a single lap joint it is equal to the overall length of the joint. Figures 5.50, 5.51, and 5.52 show plots for other combinations of plate material and bolt grades. A minimum factor of safety of 2.0 is provided when the 30 ksi allowable shear stress is used for A325 bolts installed

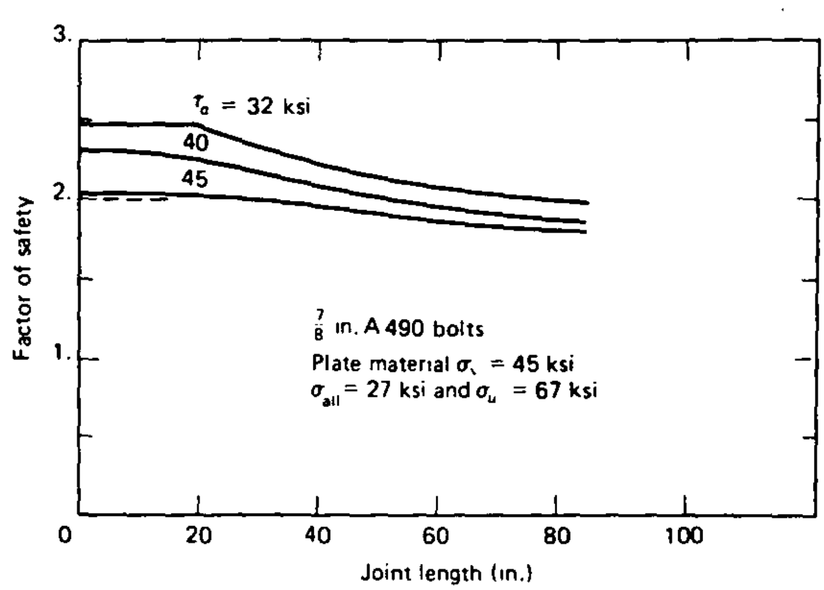


Fig. 5.51. Factor of safety for high-strength steel joints fastened by A490 bolts.

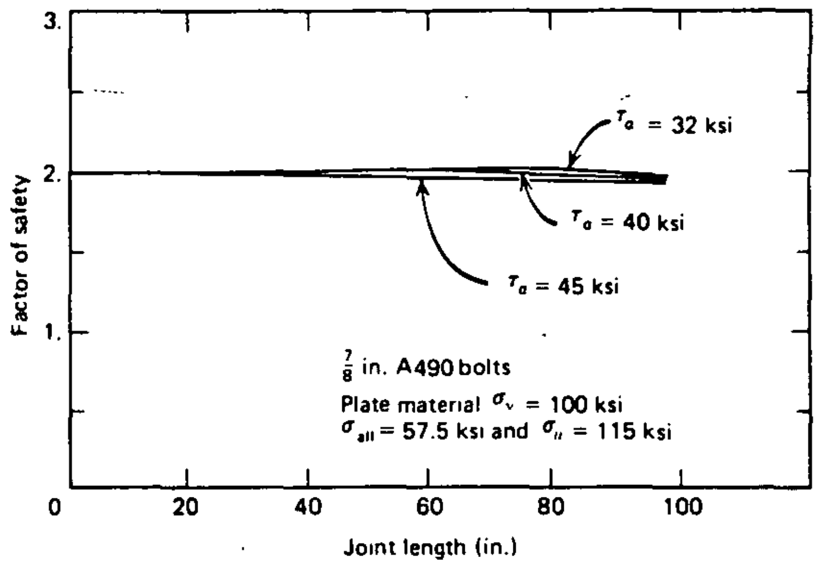


Fig. 5.52. Factor of safety for quenched and tempered alloy steel joints fastened by A490 bolts.

in structural carbon steel up to a joint length of 60 in. High-strength steel with a tensile strength of 66 ksi and fastened by A325 bolts provides a minimum factor of safety of 2.0 up to a joint length of about 50 in. Figures 5.51 and 5.52 show that a 40 ksi allowable shear stress for A490 bolts would provide the needed margin for joint lengths up to about 50 in. For joints with a length exceeding 50 in., the allowable shear stress in the bolts must be reduced to ensure a minimum factor of safety of 2.0. A 20% reduction in the allowable shear stress provides this margin for joint lengths between 50 and 90 in., as illustrated in Figs. 5.49 to 5.52.

Design Recommendations for Bolted Joints

ALLOWABLE STRESS DESIGN

Shear Stresses for High-strength Bolts

$$\tau_a = \beta \tau_{\text{basic}}$$

where τ_{basic} = 30 ksi − A325 bolts
τ_{basic} = 40 ksi − A490 bolts
β = 1.0 unless joint length exceeds 50 in., in which case β = 0.8.

Allowable Joint Loads

1. Shear planes pass through bolt shank

$$P = mn\tau_a A_b$$

2. Shear planes pass through bolt threads

$$P = 0.70mn\tau_a A_b$$

ii. Load Factor Design Bolted Joints. In load factor design, the connections and structural members are proportioned so that the product of maximum strength and a reduction factor ϕ is at least equal to the effect of the applied design loads multiplied by their respective load factors. The reduction factor ϕ is introduced to assure that the maximum strength of a structure is limited by the capacity of its members rather than by premature failure connections. The ϕ factor also accounts for the variability in strength of a connection. A uniform ϕ factor of 0.80 has been suggested for mechanical fasteners loaded in shear.[5.50]

The shear strength of a single fastener is about 60% of its tensile strength (see Section 4.2). A ϕ factor of 0.80 yields shear stresses comparable to those obtained by factoring the suggested working allowable shear values by 1.6. The same ϕ factor is applicable to A307 bolts and to A502 rivets. The ultimate shear capacity of a high-strength bolted connection is affected by the location of the shear planes. If a plane intersects the bolt threads, only the root area is effective in resisting the shear. This reduces the joint shear capacity by about 25% (see Section 4.10).

DESIGN RECOMMENDATIONS FOR BOLTED JOINTS

Load Factor Design—Shear Loading

$$\text{Design strength} = \phi F$$

where F – average shear strength $= 0.60\sigma_u$
ϕ – reduction factor $= 0.80$
If joint length exceeds 50 in. $\phi = 0.64$

Factored Joint Loads

1. Shear planes pass through bolt shank

$$P = mn\phi F A_b$$

2. Shear planes pass through bolt threads

$$P = 0.70mn\phi F A_b$$

iii. Slip-Resistant Joints. If it is assumed that equal clamping forces are present throughout a joint, then the slip resistance of a connection can be expressed as

$$P_s = mnT_i k_s \tag{5.23}$$

For a given joint geometry, the slip resistance is directly proportional to the product of the initial clamping force, T_i and the slip coefficient, k_s. Both quantities have considerable variance, and this must be considered when determining design criteria for slip-resistant joints. Since the frequency distributions for k_s and T_i are known for different surface conditions, bolt types, and tightening procedures (see Subsections 5.1.5 and 5.1.6), the joint frequency distribution for the product $k_s T_i$ can be determined[5.33] and suitable design expressions formulated. As an alternative to Eq. 5.23, an equivalent allowable bolt shear stress can be developed.

Considering Eq. 5.23, it will be desirable to reformulate this expression so that deterministic values can be used for T_i and k_s. Over and above this, it will be appropriate to provide design information for different levels of slip probability (the probability that the load predicted by Eq. 5.23 may be exceeded) in order that the designer might have the option of selecting a slip probability level suitable for his structure. Equation 5.23 can be written as

$$P_s = mn\alpha T_{i_{spec}} k_s \tag{5.24}$$

where

$$\alpha = T_i/T_{i_{spec}} \tag{5.25}$$

and $T_{i_{spec}}$ is the specified minimum bolt tension. In a further step, Eq. 5.24 will be expressed as

$$P_s = DmnT_{i_{spec}} k_{s_{mean}} \tag{5.26}$$

where D is a multiplier that provides the relationship between $k_{s_{mean}}$ and k_s, incorporates α, and reflects the slip probability level selected.

The frequency distribution curve for the product of the two variables in Eq. 5.23, that is, T_i and k_s, is shown in Fig. 5.53*a* for A325 bolts fastening material in the clean mill scale condition and installed by the turn-of-nut method. Similar curves can be constructed for other fastener and faying surface conditions. A cumulative frequency curve constructed from this information is shown in Fig. 5.53*b*. If a very high value of $k_s T_i$, relative to the value actually present in the joint, were to be selected by the designer, then there would almost certainly be slip. On the other hand, if a very low value of $k_s T_i$ were selected as the design level, there would be very little likelihood of slip.

Two of the slip probability levels that might be chosen, 5% and 10%, are shown in Fig. 5.53*b*. The 5% slip probability (or 95% confidence level) corresponds to past practice for slip-resistant connections. If a lower slip probability is desired, the 1% level could be chosen; if a higher slip probability can be justified, 10% could be used.

Information like that given in Fig. 5.53*b* can be tabulated. Table 5.2 gives values of D for use in Eq. 5.26 for either A325 or A490 bolts installed by turn-of-nut and corresponding to various slip probability levels. The slip coefficients

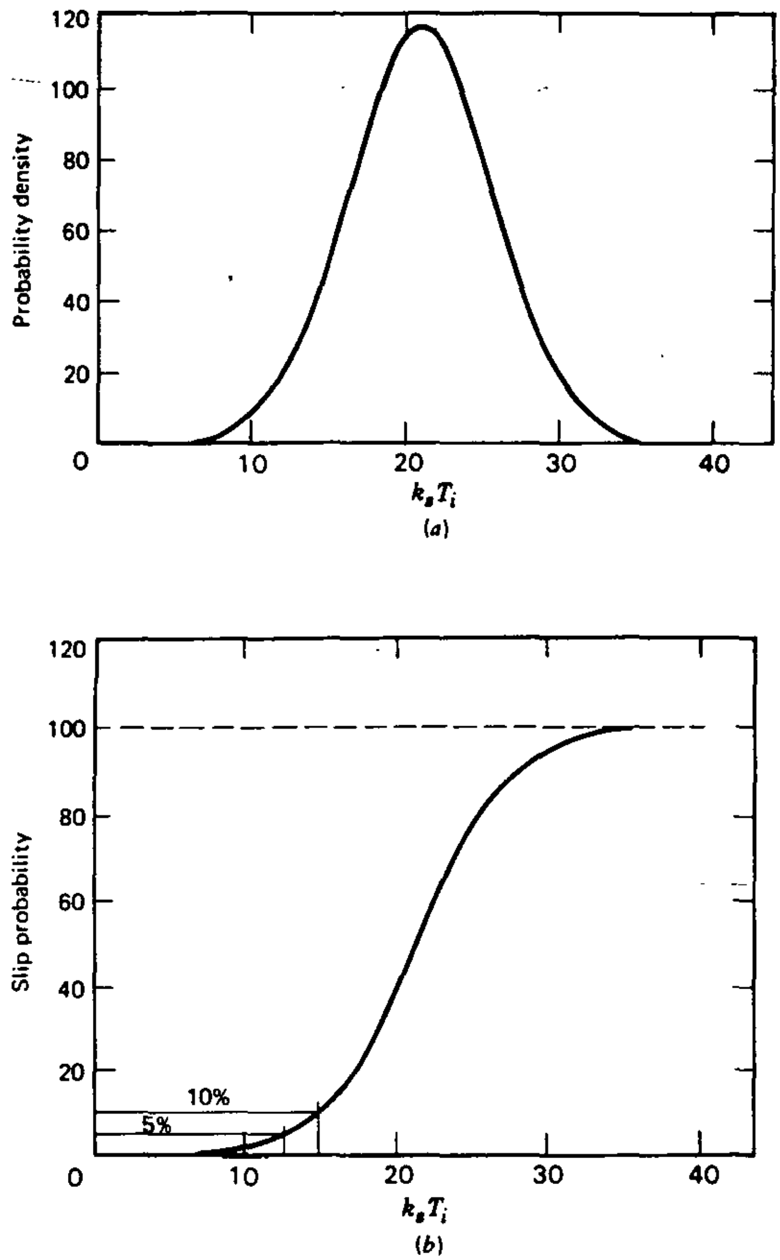


Fig. 5.53. Slip resistance. (*a*) Frequency distribution. (*b*) Cumulative frequency curve.

listed (mean values) are 0.20, 0.25, 0.33, 0.40, 0.50, and 0.60. The standard deviations used with these values in order to develop the table were 0.07 for mean values between 0.20 and 0.40 and 0.09 for the remainder. Table 5.3 gives similar information for A325 or A490 bolts installed using the calibrated wrench method.

A comparison of Tables 5.2 and 5.3 indicates that slip-resistant connections using bolt installed by the turn-of-nut method will have a slightly greater resistance than if the bolts were installed by calibrated wrench. For example, at the 5% slip-probability level, A325 bolts installed by turn-of-nut gain a premium of about 14% over A325 bolts installed by calibrated wrench. The difference reflects the higher preloads obtained in bolts installed by the turn-of-nut method. For A325 or A490

Table 5.2. Slip Factor D for use in Eq. 5.26: Turn-of-Nut Installation

	Slip Probability					
	A325 Turn-of-Nut			A490 Turn-of-Nut		
k_s (mean)	1%	5%	10%	1%	5%	10%
0.20	0.253	0.551	0.728	0.243	0.520	0.684
0.25	0.383	0.677	0.831	0.376	0.642	0.782
0.33	0.590	0.820	0.942	0.568	0.776	0.887
0.40	0.696	0.896	1.001	0.671	0.848	0.942
0.50	0.702	0.899	1.002	0.672	0.850	0.944
0.60	0.772	0.947	1.040	0.738	0.895	0.979

Note: Standard deviation of k_s (mean) taken as 0.07 for $k_s \leq 0.4$ and as 0.09 otherwise.

bolts installed by calibrated wrench, α is 1.13, whereas it is 1.35 for A325 bolts or 1.26 for A490 bolts installed by $\frac{1}{2}$ turn-of-nut, respectively.

The same information represented by Eq. 5.26 and Tables 5.2 and 5.3 can be expressed in terms of a permissible shear stress. (This is a convenience only; it must be remembered that the fastener in a slip-resistant connection is not actually acting in shear.) Equating the slip resistance (Eq. 5.3) to an equivalent shear force gives

$$mnT_i k_s = mn\tau_b A_b \tag{5.27}$$

where τ_b is the equivalent shear stress and A_b is the nominal bolt area. Using α (Eq. 5.25) and expressing the specified bolt tension as

Table 5.3. Slip Factor D for use in Eq. 5.26: Calibrated Wrench Installation

	Slip Probability A325 or A490 Calibrated Wrench		
k_s (mean)	1%	5%	10%
0.20	0.235	0.478	0.622
0.25	0.372	0.594	0.714
0.33	0.547	0.718	0.810
0.40	0.639	0.784	0.862
0.50	0.643	0.787	0.864
0.60	0.702	0.829	0.897

Note: Standard deviation of k_s (mean) taken as 0.07 for $k_s \leq 0.4$ and as 0.09 otherwise.

$$T_{i_{spec}} = 0.7A_s\sigma_{u_{spec}} \tag{5.28}$$

where A_s is the stress area of the bolt, then Eq. 5.27 can be rewritten as

$$\tau_b = 0.7k_s\sigma_{u_{spec}}\,(A_s/A_b) \tag{5.29}$$

The ratio of the stress area to the nominal bolt area varies from only 0.736 for a $\frac{5}{8}$-in. diameter bolt up to 0.774 for a 1-in. diameter bolt. An average value of 0.76 will be used herein. The minimum specified tensile strength for A325 bolts in sizes $\frac{1}{2}$ through 1 in. diameter is 120 ksi. Substituting these values into Eq. 5.29 yields

$$\tau_b = 63.8k_s\alpha \tag{5.30}$$

Equation 5.30 relates the equivalent shear stress on the fastener to the known parameters α and k_s (as described in Section 5.1). An expression similar to Eq. 5.30 can be developed for A490 bolts; only the multiplier changes (to 78.7).

Of course, the frequency distribution and cumulative frequency distribution curves corresponding to Eq. 5.29 look just the same as those shown in Fig. 5.53*a* and *b*. Table 5.4 gives the equivalent permissible shear stresses for slip-resistant joints using A325 or A490 bolts installed by the turn-of-nut method, and Table 5.5 presents the same information for use when calibrated wrench installation is used. The slip coefficients selected ($k_{s_{mean}}$) and their standard deviations are the same as those used in Tables 5.2 and 5.3.

In evaluating conditions for A325 bolts, the specified minimum tensile strength was presumed to be 120 ksi. The specified tensile strength for A325 bolts in sizes over 1 in. diameter is in fact 105 ksi. Experience has shown that the actual strength of A325 bolts over 1 in. diameter usually ranges from 20 to 34% above the minimum specified tensile strength. Furthermore, the A_s/A_b ratio for these sizes is

Table 5.4. Equivalent Shear Stress for Use in Slip-Resistant Connections: Turn-of-Nut Installation

	Slip Probability					
	A325 Turn-of-Nut			A490 Turn-of-Nut		
k_s (mean)	1%	5%	10%	1%	5%	10%
0.20	3.23	7.03	9.29	3.82	8.18	10.77
0.25	6.11	10.80	13.25	7.40	12.63	15.39
0.33	12.42	17.27	19.84	14.74	20.16	23.03
0.40	17.75	22.85	25.53	21.12	26.70	29.66
0.50	22.39	28.67	31.98	26.44	33.44	37.14
0.60	29.56	36.24	39.79	34.82	42.28	46.23

Table 5.5. Equivalent Shear Stress for Use in Slip-Resistant Connections: Calibrated Wrench Installation

	Slip Probability					
	A325 Calibrated Wrench			A490 Calibrated Wrench		
k_s (mean)	1%	5%	10%	1%	5%	10%
0.20	3.00	6.10	7.94	3.70	7.52	9.79
0.25	5.93	9.47	11.39	7.32	11.69	14.05
0.33	11.52	15.11	17.06	14.21	18.64	21.04
0.40	16.31	20.01	22.00	20.12	24.69	27.14
0.50	20.51	25.09	27.56	25.29	30.95	33.99
0.60	26.87	31.73	34.33	33.15	39.14	42.35

about 0.81 as compared with the value 0.76 for sizes less than 1 in. diameter. An increase in the A_s/A_b ratio increases the shear stress, as is apparent from Eq. 5.29. Hence, the values listed in Tables 5.2 through 5.4 are assumed applicable to all commonly used A325 bolt sizes.

A reduction factor must be applied to account for the effect of fabrication factors on the slip resistance of joints; for example, depending on the amount of oversize of the hole or the direction of the slotted holes with respect to the expected slip direction, a reduction in slip resistance may result. Chapter 9 deals specifically with oversize and slotted holes and discusses in greater detail the influence of these fabrication factors on the slip resistance of a joint.

Strength as well as performance must be considered in the design of slip-resistant joints. As mentioned in Subsection 5.4.1, the permissible load of a slip-resistant connection must not exceed its capacity based on considerations of strength. In other words, the permissible load for a joint evaluated on the basis of its strength capacity (as governed by shear of the bolts or bearing of the connected parts) forms the upper bound for the design of a slip-resistant connection. Slip-resistant connections governed by this upper bound are likely to be only those in which the slip coefficient is high or the probability of slip selected is high, or some combination of both of these. For example, a joint with a $k_{s_{mean}}$ value of 0.50 using A325 bolts installed by turn-of-nut will have a permissible equivalent shear stress of 32.0 ksi when designed against slip resistance. However, its capacity when checked as a bearing-type connection will be based on a permissible shear stress of only 30.0 ksi (Subsection 5.4.1). Thus, the latter governs even though this was a connection designed as slip resistant.

DESIGN RECOMMENDATIONS FOR SLIP-RESISTANT JOINTS

Slip-resistant joints may be proportioned in accordance with either Alternative A or Alternative B, as given below. The result will be the same in either case.

Alternative A

$$P_s = DmnT_{i_{spec}}k_{s_{mean}}$$

where D is obtained from Table 5.2 or 5.3

Alternative B

$$P_s = mn\tau_a A_b$$

where τ_a is obtained from Table 5.4 or 5.5 and A_b is the cross-sectional area corresponding to the nominal diameter of the bolt.

If slotted or oversize holes are used, the joint capacity calculated by either Alternative A or Alternative B must be reduced by multiplying by 0.70. See Chapter 9 for details on slotted and oversize holes.

In either allowable stress design or load factor design, the resistance described using either Alternative A or Alternative B is to be compared with the effect of the working loads (sometimes called specified loads in load factor design.) In allowable stress design, the slip-resistant joint must also be checked against its shear capacity (Subsection 5.4.2i) and its bearing capacity (Subsection 5.4.4i). In load factor design, the slip-resistant joint must likewise be checked against its shear capacity (Subsection 5.4.2ii) and its bearing capacity (Subsection 5.4.4ii) using factored loads.

5.4.3 Design Recommendations—Connected Material

It was noted in Section 5.2 that is was desirable that yielding through the gross cross-section of a member occur prior to failure at the net cross-section in order that the member behavior be ductile. That requirement is included in the design recommendations that follow. It includes a multiplier that reflects the fact that, while the actual yield and ultimate strengths can both be expected to be greater than their specified minimum values, the margin on yield is usually greater than that on ultimate.

i. Static Loading

a). Allowable Stress Design. In allowable stress design, practice in the United States since 1978 has been to place a limit on the stress at the gross cross-section of the member, established at 60% of the yield strength of the material, and to require in addition that the stress on the net cross-section of the joint not be in excess of 50% of the tensile strength of the material. This provides a factor of safety of 1.67 against unrestricted plastic flow of the main member and a factor of safety of 2.0 against fracture. It will be recalled that the allowable shear stresses for bolts in bearing-type connections were established so that the factor of safety against fastener failure was at least 2.0. Thus, it can be expected that the tension member will reach its ultimate load prior to any (potential) failure of the bolts that make up its connection.

DESIGN RECOMMENDATIONS

Allowable stresses

Through gross cross-section of member,

$$\sigma_a = 0.60\ \sigma_y$$

or, through net cross-section at connection,

$$\sigma_a = 0.50\ \sigma_u$$

but,

$$\frac{A_n}{A_g} \geqslant \frac{\sigma_y}{0.9\ \sigma_u}.$$

Thus, the allowable load on the member is the lesser of

$$P_1 = 0.60\ \sigma_y \mathrm{A}_g$$

or

$$P_2 = 0.50\ \sigma_u \mathrm{A}_n.$$

b). Load Factor Design. The limit of strength of a tension member is its capacity as established by fracture at the net section. This capacity should be compared with the effect of the factored loads. A reduction (ϕ) will be applied to this nominal capacity ($A_n \sigma_u$) to reflect the possibility of undersize of member, accuracy of analysis, and actual material properties. For a safety index of 3.0, which is the value used for beams, columns, and beam-columns, a value of ϕ = 0.90 is appropriate. It is worth noting that the safety index established for mechanically fastened connections[5.50] is 4.5, reflecting the desire that connections do not reach failure before the ultimate strength of the member has been attained.

In addition to strength, another limit state exists for tension members. This is unrestricted plastic flow of the main member, that is, yielding through the gross cross-section of the member. This could occur at loads only slightly greater than the working load level if only the strength limit were applicable. Thus, it is necessary that a second limit be applied, as noted below.

As was the case for tension members designed under the allowable stress method, the ductility of the member must also be ensured.

DESIGN RECOMMENDATIONS

Member capacity under factored loads shall be taken as the lesser of

$$P_f = \phi\ A_n \sigma_u$$

or

$$P_f = \phi\, A_g \sigma_y$$

where $\phi = 0.90$.
But

$$\frac{A_n}{A_g} \geqslant \frac{\sigma_y}{0.9\,\sigma_u}.$$

ii. Repeated Loading. Results of fatigue tests on slip-resistant as well as other types of bolted joints were discussed in Section 5.3. It was shown that the type of failure was related to the manner in which the applied load was carried by the joint. If transmitted by frictional resistance on the contact surfaces alone, failure was through the gross section. When slip occurred and part of the load was transmitted by bearing and shear, failure generally occurred through the net section. The fatigue strength at the gross section of slip-resistant joints was about equal to the fatigue strength at the net section of joints that had slipped into bearing under nonreversible loading.

Design category B, which was derived from tests on plain welded beams,[5.51] provides a reasonable lower bound estimate for the stress range versus life relationship of bolted joints. The allowable stress ranges determined from this stress range versus life relationship for different loading conditions are summarized in Table 5.6. A stress range of 16 ksi was estimated for a life of 2 million cycles or more.

For the design of high-strength bolted joints under cyclic loading, the suggested stress range can be applied to: (1) the gross section area of slip-resistant joints with a slip probability of 5% or less, and (2) the net section area for other bolted joints. This provides design stresses for clean mill scale conditions that are in reasonable agreement with current practice. Joints subjected to reversal of stress should always be designed as slip-resistant joints in order to prevent excessive movement of the connected parts.

The stress range on the net section area governs the design of bolted joints that have a slip probability greater than 5%. These joints should not be used in situations

Table 5.6. Allowable Range of Stress for the Plate Material

Design Load Cycles	Stress Range for 95% Survival (ksi)
20,000–100,000	45.0
100,000–500,000	27.5
500,000–2,000,000	18.0
Over 2,000,000	16.0

where reversal of load occurs. However, slip in the direction of the maximum applied load is not critical unless the load is reversed.

Application of the stress ranges given in Table 5.6 provides a conservative design for both slip-resistant and bearing-type bolted joints. Better estimates of the stress range-life relationship may be developed when additional experimental data become available.

DESIGN RECOMMENDATIONS FOR JOINT MATERIAL UNDER REPEATED LOADING

Slip-Resistant Joints

Calculate stress range on gross section area if the slip probability is less than or equal to 5%.

Other Bolted Joints

Calculate stress range on the net area if the slip probability is greater than 5%. Stress reversal is not permitted. Allowable stress range for both types is given in Table 5.3

iii. Bearing Stresses. In Section 5.2.9 it was shown that the lower bound *L/d* ratio that prevents a single fastener from splitting out of the plate material can be expressed as:

$$\frac{L}{d} \geq 0.5 + 0.715 \frac{\sigma_b}{\sigma_u^P} \tag{5.31}$$

Butt joints with a single fastener were more critical than joints with multiple fasteners in a line. The clamping force in a high-strength bolt also has a favorable influence on the bearing strength of the connection. Hence, design recommendations based on test results of finger-tight single fastener specimens provide a conservative estimate of the required end distance.

The test results indicate that Eq. 5.31 provides an acceptable lower bound solution to the strength of the end zone for an *L/d* ratio up to 3.0 as illustrated in Fig. 5.54. When the *L/d* ratio exceeds 3.0, the failure mode changes gradually from a "shearing-type" failure to one in which large hole and material deformation occurs.

An alternative relationship can be used which directly relates the *L/d* ratio to the bearing stress-tensile strength ratio:

$$\frac{L}{d} \geq \frac{\sigma_b}{\sigma_u^P} \tag{5.32}$$

This relationship is also plotted in Fig. 5.54 and it is also in good agreement with the test data.

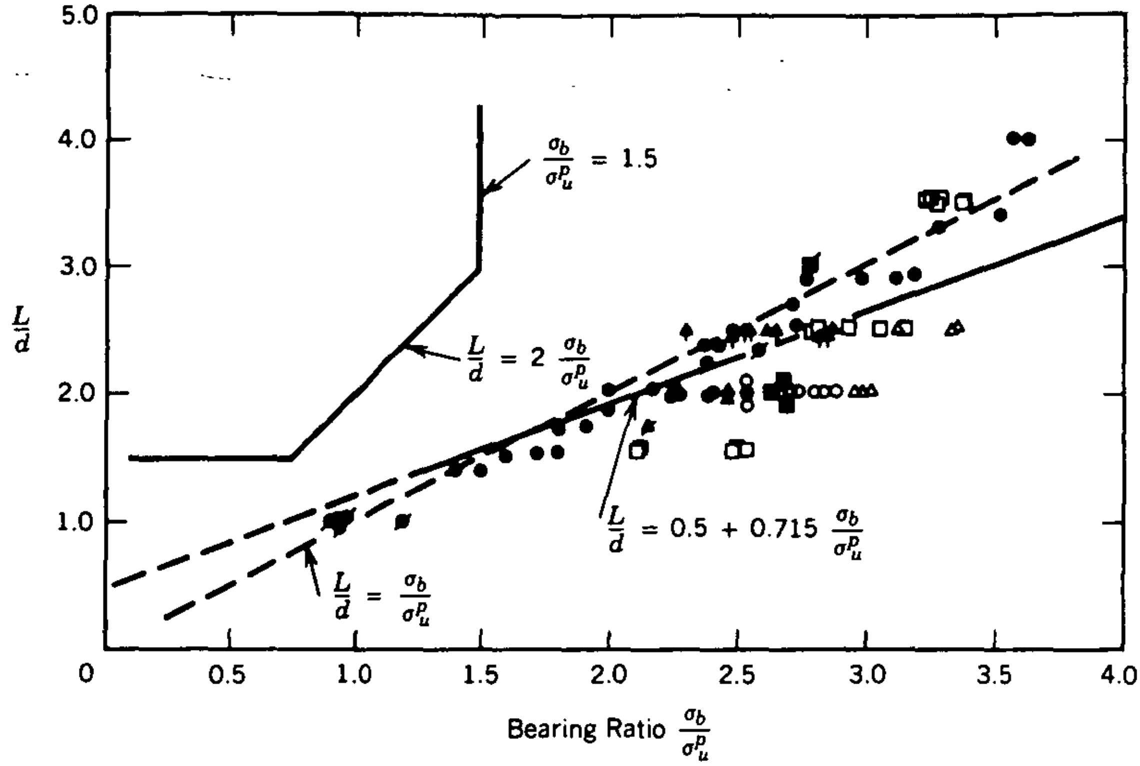


Fig. 5.54. Comparison of design recommendations for allowable stress design with test results.

a. Allowable Stress Design. If a minimum factor of safety with respect to ultimate load of 2.0 is selected, the required *L*/*d* ratio becomes

$$\frac{L}{d} \geq 0.5 + 1.43\,\frac{\sigma_b}{\sigma_u^P} \tag{5.33}$$

As is shown in Fig. 5.54, Eq. 5.31 defines the *L*/*d* ratio up to a bearing stress-tensile strength ratio of about 3.0. The suggested factor of safety of 2.0 against bearing failure is comparable to the factors of safety against shear or tension failure of the fasteners and the tensile strength of the net section.

If the alternate formulation is used, the required *L*/*d* ratio becomes:

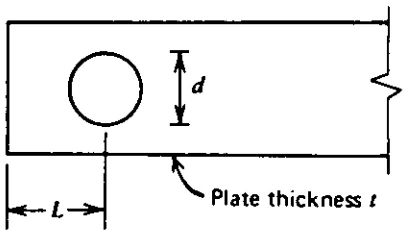
$$\frac{L}{d} \geq 2\,\frac{\sigma_b}{\sigma_u^P} \tag{5.34}$$

To properly install a bolt or rivet, a minimum distance from the center of the fastener to any edge of the member must be maintained. A minimum *L*/*d* ratio of 1.5 is suggested since this conforms to current practice.

The design region shown in Fig. 5.54 is further bounded by a vertical line at a bearing stress-tensile strength ratio of 1.5. This prevents use of bearing stresses that may lead to excessive hole deformations and the upsetting of material in front of the fastener. Although the strength in such a situation is still adequate, large deformations may limit usefulness. Furthermore, a high σ_b/σ_u^P ratio corresponds to a large ratio of bolt diameter to the plate thickness. Thin plates that may deform out of their plane due to instability of the end section may limit the ultimate capacity of the end zone. These conditions may arise if the lap plates of a butt joint are critical in bearing. Due to "catenary action," the ends of the lap plates tend to bend outward. A high compressive force on the end panel may cause a dishing-type failure and decrease the ultimate bearing strength.

5.4.4 Design Recommendations for Bearing Stresses

i Allowable Stress Design



Bearing stress $\sigma_b = P/dt$
σ_u^P = tensile strength plate material

Following conditions are to be satisfied:

1. $L/d \geq 0.5 + 1.43\ \sigma_b/\sigma_u^P$; alternatively, $L/d \geq 2\sigma_b/\sigma_u^P$
2. $L/d \geq 1.5$
3. $\sigma_b/\sigma_u^P \geq 1.5$

ii *Load Factor Design.* A lower bound to the shear resistance of the end zone behind the fastener was expressed as (see Subsection 5.2.9):

$$F = (2t)\left(L - \frac{d}{2}\right)(0.7\sigma_u^P) \qquad (5.35)$$

A ϕ factor of 0.85 is believed adequate to account for the uncertainties in the strength of the end zone. Hence the shear strength of the end zone panel for load factor design becomes

$$\phi F = (0.85)\ (1.4)\left(L - \frac{d}{2}\right) t\sigma_u^P \qquad (5.36)$$

A minimum L/d ratio equal to 1.5 is desired for installation. In order to limit deformations of the hole, the bearing ratio σ_b/σ_u^P should not exceed 3.0 at the factored load level.

A ϕ factor of 0.85 provides bearing stresses on the fastener that are equal to those obtained by factoring the allowable bearing stress values given by Eq. 5.33.

DESIGN RECOMMENDATIONS FOR BEARING STRESSES

Load Factor Design

Shear strength end zone

$$F = (1.4)\left(L - \frac{d}{2}\right) t\sigma_u^P$$

Reduction factor $\phi = 0.85$
Following conditions are to be satisfied

1. Design load × load factor $\leq \phi F$; alternatively, $L/d \geq 1.7\ \sigma_b/\sigma_u^P$
2. $L/d \geq 1.5$
3. $\sigma_b/\sigma_u^P \leq 3.0$

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Chapter Fourteen
Combination Joints

14.1 INTRODUCTION

Most connections use a single fastening system to connect plates or members together and provide the means of transferring the forces acting in or on the joint. However, situations do arise where it is desirable or necessary to combine two different methods of fastening in a single connection. This generally involves rivets and bolts or bolts and welds. In these connections the two fastening systems share the load. Joints of this type are generally referred to as combination joints or load-sharing joints.

There are two general types of combination connections, as illustrated in Fig. 14.1. One type, shown in Fig. 14.1*a*, utilizes two different fastening systems to share the load on a common shear plane. This condition may occur when reinforcing or strengthening an existing joint. For example, high-strength bolts may be used to replace several rivets. In other situations, space may not be available for additional fasteners, and welds are added to the joint. In either case, the applied loads are transferred by both types of fasteners on a common shear plane.

Combination joints that combine fasteners on a common shear plane have the advantage of being compact. This reduces the required space and the amount of splice material. In addition, they can help overcome field erection problems. Although welded connections are generally more compact than bolted connections, fabrication tolerances for welding are more rigid than the tolerances allowed for bolted connections. Before the welding process is started, positioning and holding of components in place must also be considered and accounted for. Bolted connections with regular hole clearance ($\frac{1}{16}$-in.) provide for some relative movement between the connected parts after initial assembly and before final tightening of the bolts. Therefore, a member in a frame can be more easily installed with bolts. After the member has been positioned and aligned properly, the bolts are tightened. It is easy to add welds to a connection after it has been first bolted into place (see Fig. 14.1*a*).

Combination joints of the type as shown in Fig. 14.1*a* have a wide application for reinforcement of existing mechanically fastened joints. Simple shear splices or

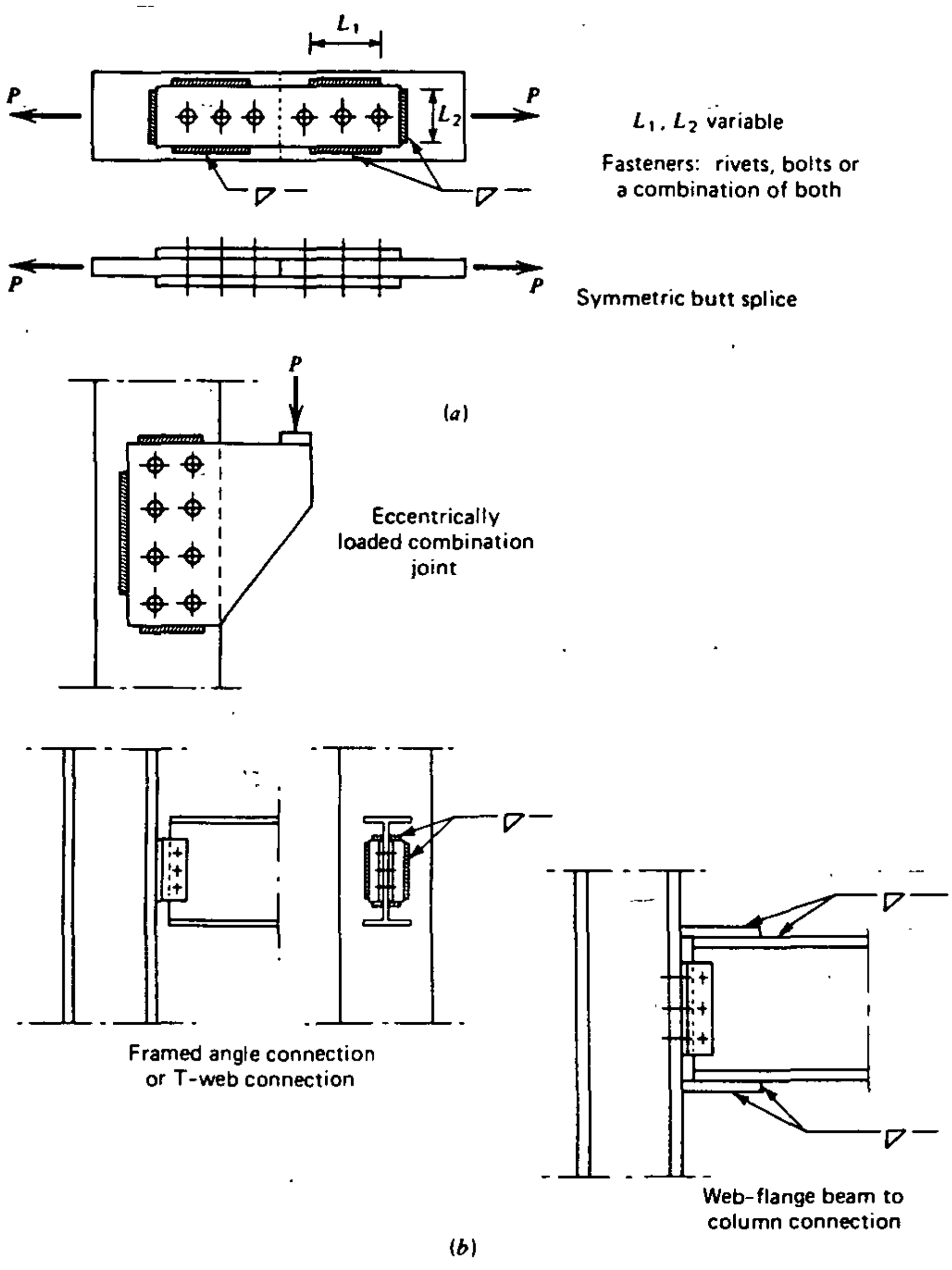

Fig. 14.1. Typical combination joints. (*a*) Load sharing on a common shear plane. (*b*) Combination joints with two different shear planes.

eccentrically loaded shear splices are typical connections that can utilize a combination of mechanical fasteners and welds on a common shear plane.

The behavior of small combination joints with bolts and welds or with bolts and rivets combined on a single shear plane has been studied to evaluate joint behavior and develop design recommendations.[5.5, 9.2, 14.1, 14.2] These tests have demonstrated the applicability of this type of joint. The work in this area is not extensive, and further research would be desirable.

In the other major type of combination connection, two different fastening methods are used but they do not act on a common shear plane. Examples of this category of combination joint are shown in Fig. 14.1*b*. These connections include the simple combination framed beam connection that utilizes shop welds to connect the web angles to either the beam web or the member into which the beam frames and bolts for the field connection. In this particular case, both the bolts and the welds are resisting the beam shear force. Variations of this type of combination joint are possible, such as welding the flanges of beam to column joints and providing a bolted shear connection for the web.

Usually, this type of combination joint will provide greater economy and increased flexibility during erection as compared with the same joint configuration that uses only one type of fastener. The many possibilities for combination joints that exist will only be limited by the ingenuity of the engineer. All available evidence shows that they provide a satisfactory joint with adequate strength and stiffness when proper design procedures are used for the component parts.[14.4]

The remainder of this chapter discusses the behavior of bolted-welded and riveted-bolted combination joints where the fasteners are sharing the load on common shear plane. Other combinations of fastening systems are not considered for this type of combination joint because of the lack of information and because of their limited use in structural applications.

Discussion of the behavior of the other major type of combination connection, where different types of fasteners are used but not on a common shear plane, is given in Chapter 18.

14.2 BEHAVIOR OF COMBINATION JOINTS THAT SHARE LOAD ON A COMMON SHEAR PLANE

Before the combined action of two different fastening methods acting in a common shear plane is discussed, it is desirable to reexamine the load versus deformation behavior of the different types of individual fasteners. Figure 14.2 shows typical load versus deformation curves for welded, bolted, and riveted tension specimens. This figure indicates that high-strength bolted connections with normal hole clearance provide a very high initial stiffness up to the slip load of the connection. During slip, the deformations increase significantly until the bolts come into bearing. After the bolts are in bearing, the load versus deformation curve shows an increase in joint stiffness. Joint slip can be minimized by installing fitted bolts in matching drilled holes.

Compared to slip-resistant high-strength bolted joints where the load is transferred by friction, riveted connections are generally more flexible. Often a sudden change in the slope of the load versus deflection curve can be observed that is directly comparable to slip in a high-strength bolted connection. However, this "slip" is usually less than one-third the slip observed in high-strength bolted connections.

A typical characteristic of a welded connection as compared with riveted or

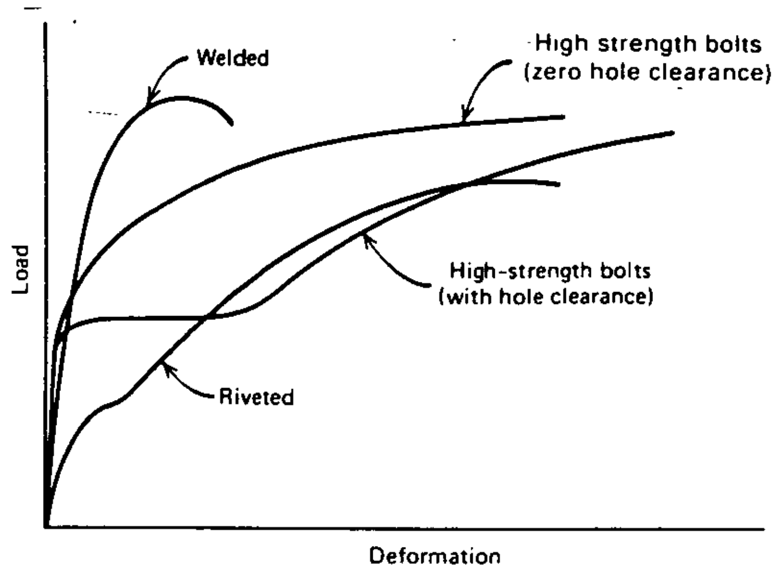

Fig. 14.2. Load versus deformation relationships for different fastening methods (Ref. 9.2.).

high-strength bolted connections is the reduced deformation capacity. Slip does not occur in welded connections, and the initial stiffness of the joint only changes as the ultimate load is approached. From these load versus deformation relationships for typical fasteners, one can conclude that combination of these fasteners would be most appropriate where compatible deformation characteristics exist. The preferred combinations appear to be welds with slip-resistant high-strength bolts and rivets with bolts.

14.2.1 High-Strength Bolts Combined with Welds

A comparison of the load versus deformation capacity of welded and high-strength bolted connections with normal $\frac{1}{16}$-in.-hole clearance indicates that the total deformation capacity of the welds is of the same order of magnitude as the maximum slip of a high-strength bolted connection. Therefore, if both fastening methods are used on a common shear plane, the capacity of the resulting combination joint might be taken as the sum of the weld strength and the slip resistance provided by the bolts.

The question arises as to what constitutes failure in a welded-bolted combination joint. As discussed above, because the weld shear deformation capacity and the observed values of slip in bolted joints are about the same, the weld shear failure can be expected to occur at the same time as the bolts slip into bearing. If the joint was designed to be slip-resistant, this would constitute failure. If the joint were designed as a bearing type, the connection now consists of a bolted connection (with some broken welds) whose capacity can be determined according to the usual rules (see Chapter 5). Thus, in new work it is not logical to consider using both high-strength bolts and welds in the same joint unless it is categorized as a slip-resistant connection. It must also be noted that in load factor design a slip-resistant connection must also be checked with respect to the ultimate limit state. The ultimate resistance of the bolted-welded joint, as defined by complete separation of the parts, will be the greatest of the shear capacity of the bolts, the bearing capacity of the plates, or the shear capacity of the welds. The resistance so determined must be at least equal to the effect of the factored loads.

In renovation or repair work, two separate loading cases should be identified. If, for example, welds are added to a bolted joint that has little or no load, the case is the same as that described for new work. On the other hand, if the joint is already under load, the existing component, bolts or welds, must be initially carrying that load. Load applied subsequent to the addition of welds or bolts will be shared between the original fastening elements and those that have been added. Whether the joint is to be considered now as slip resistant or bearing type will depend upon individual circumstances. Similarly, if the joint is a bearing type, the identification of the critical fastening element will have to be done on a case-by-case basis.

Tests have been performed to evaluate the validity of the assumption made for new work (or for renovations done under no load), namely, that the shear capacity of the welds and the slip resistance of the bolts can be added.[5.5,9.2,14.1,14.2] The test joints were generally small tension type butt splices with two bolts on either side of the splice, as shown in Fig. 14.3. The influence of the location of the welds, that is, either transverse or parallel to the applied load, was also studied. Furthermore, the ratio of the capacity of the welds with respect to the slip resistance of the bolts was considered as a test variable.

Figure 14.3 summarizes the results observed in a typical series of test joints.[9.2] The load versus deformation behavior of the plain welded and the plain bolted connection is shown, as is that for the combination bolted and welded joint. It is apparent that the behavior of the combination joint can be adequately described by the sum of the slip load of the plain bolted connection and the strength of the welds. Other combinations of weld length, weld location, and slip resistance of the bolted joint resulted in similar conclusions.[9.2,14.1]

The tests reported in Ref. 9.2 were limited to small connections with only a few bolts in line. In larger connections, some misalignment may exist and the bolts come into bearing before failure of welds occurs. The load carried by the bolted connection is then transmitted by friction and bearing. The failure load of these connections is likely to exceed the estimated ultimate load determined from the slip resistance of the bolts and the strength of the welds. Reducing the hole clearance would also bring the bolts into bearing and increase the ultimate strength of a bolted-welded combination joint. The maximum capacity of a combination joint is developed when fitted bolts are installed in matching drilled holes. Tests have indicated that these connections have an ultimate load that exceeds the summation of the weld strength and the slip load of the bolted connection.[9.2,14.2] Obviously, such joints are not very economical in new work. However, in existing work holes would of necessity be drilled, and they would of course be matched. In this case, fitted bolts could be easily used.

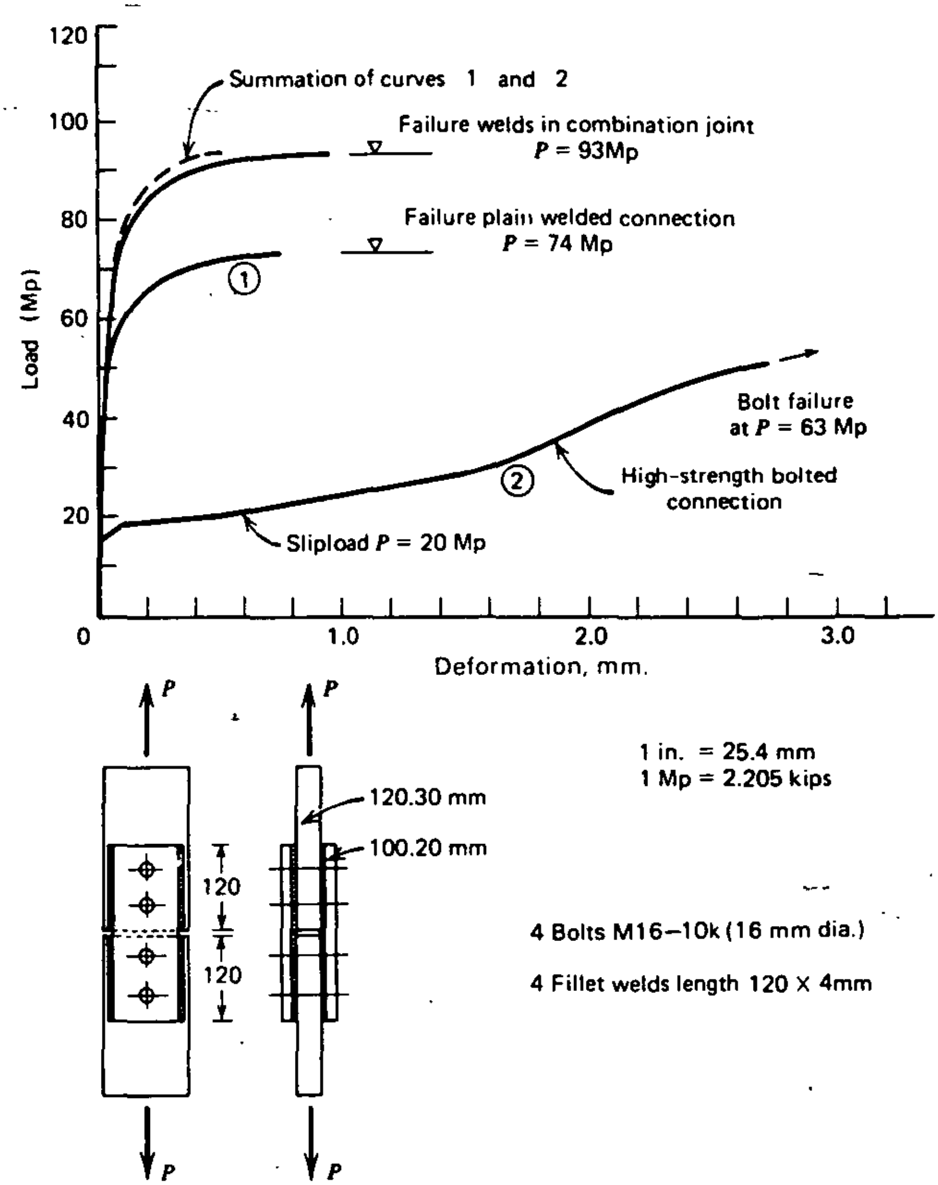

Fig. 14.3. Test results of welded, bolted, and combined welded-bolted joints (Ref. 9.2).

Another aspect that has to be considered is the behavior of combination joints under repeated loading conditions. The behavior of high-strength bolted connections subjected to repeated loading conditions is discussed in Chapter 5. Tests performed in Germany indicated that the fatigue strength of a high-strength bolted connection decreases when weldments are added.[9.2] This reduction in fatigue strength is expected, because the weld toe is the critical region, and crack growth will occur just as in a welded joint. The weld toe was more critical than the bolt holes in all test joints.[9.2] A comparison of the few data available for welded joints indicates that the fatigue strength is not significantly different from the fatigue strength of a similar plain fillet welded connection. Hence, the design criteria for welded joints should be used for cyclic load conditions when the welds are positioned on the boundaries of the combination joint.

Some tests have indicated that an improvement in fatigue strength can result when the welds are placed on the joint interior.[14.3] This removes the weld from the more highly stressed joint boundary where the geometric discontinuity is more severe and places it in a lower stressed region. In addition, the stress concentration condition is generally decreased, since the connected parts are more nearly subjected to about the same strain conditions. However, caution must always be exercised when adding weld to existing bolted joints. The danger exists that conditions favorable to crack growth will be created, particularly if these welds are used at plugs or slots.

14.2.2 High-Strength Bolts Combined with Rivets

A combination of rivets and high-strength bolts intersecting the same shear plane would not be used in new construction. However, high-strength bolts are often used to replace one or more rivets in existing riveted connections. This is done to either repair the joint or to strengthen the connection.

The addition of high-strength bolts to a riveted connection results in a number of improvements. For a given diameter, a high-strength bolt has a greater shear strength than a rivet, and so the ultimate strength of the whole connection will be increased. If the connection is slip-resistant, the stiffness will be increased by the addition of high-strength bolts. If the slip resistance is exceeded, the presence of the rivets, which have less hole clearance than do high-strength bolts, means that less slip will take place as compared with a fully-bolted joint. Furthermore, replacing rivets by high-strength bolts has been shown to improve the fatigue strength of the joint.[14.5]

Tests to evaluate the load versus deformation behavior of short bolted-riveted combination joints have indicated that the ultimate strength of the joint is adequately approximated by the summation of the resistance of the two types of fasteners.[9.2] This is illustrated in Fig. 14.4, where the load versus deformation curves of a riveted, a bolted, and a bolted-riveted combination joint are compared. This figure clearly shows the increased stiffness of the combined joint as compared with the riveted joint. The improved slip behavior of the combination joint is also evident.

Since the joint strength of short combination joints is an aggregate of the strengths of the individual fasteners, it does not matter how the fasteners are arranged in the combination joint. Hence, either the outermost rivets or rivets located in the joint interior can be replaced by high-strength bolts. Either arrangement yields about the same ultimate load. Based upon the observed behavior of long riveted and bolted joints, the fastener location will influence the joint strength. Because of "unbuttoning," replacing the outermost rivets of a long joint by high-strength bolts will be more effective in increasing the joint strength than replacing the same number of interior fasteners. Experimental verification is not available on long joints at the present time (1987).

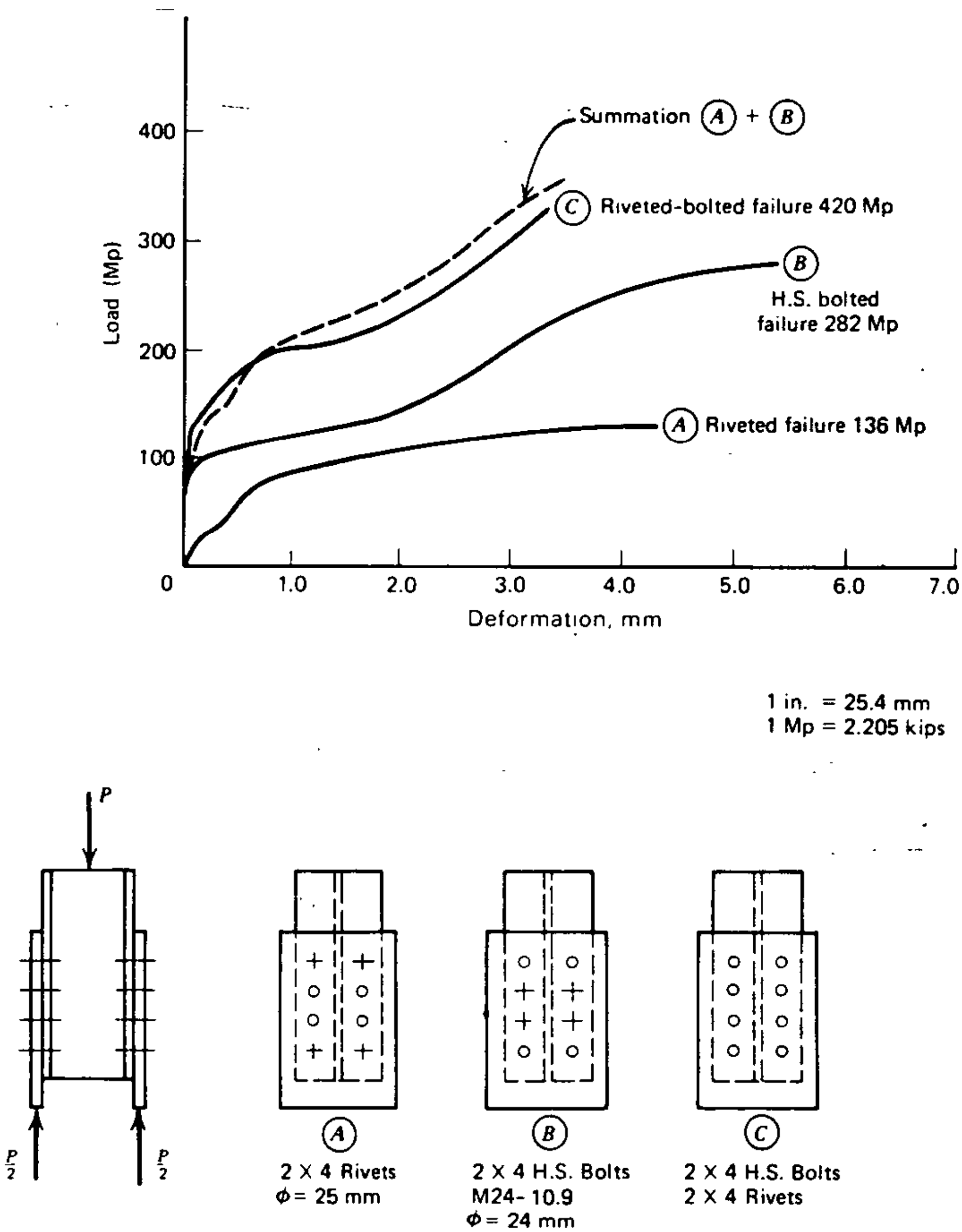

Fig. 14.4. Test results of riveted, bolted, and combined riveted-bolted joints (Ref. 9.2).

As was discussed for the case of bolted-welded joints, the amount of load on a rivet joint at the time that high-strength bolts are added must be known. If the existing load is small or zero, the rivets and bolts can be assumed to share the load, as discussed above. However, if the joint is already under load at the time of reinforcement, the rivets will already be under load when the high-strength bolts are introduced. Additional load will be shared between the rivets and the bolts.

The joint strength will have to be assessed on a case-by-case basis, and this can be done by relating the fastener deformations and loads.

Consider the replacement of some rivets at the extremities of a loaded, riveted joint by high-strength bolts. Under the existing load, the shear force per rivet can be calculated (see Subsection 5.2.5) and the corresponding shearing deformations, Δ_e, established using values similar to those shown in Fig. 3.2. The rivet shearing deformation at ultimate, Δ_m, is also obtainable from Fig. 3.2. When high-strength bolts are added, they can only be subjected to the difference between the two rivet shear deformations, that is, $\Delta_b = \Delta_m - \Delta_e$. The force per bolt can then be obtained from a figure such as Fig. 4.11 or from the mathematical expressions developed to describe this relationship.[5.22] Finally, the ultimate capacity of the riveted-bolted combination joint can be calculated as the sum of the rivet forces and the bolt forces established as above. Although the procedure described is believed to be sound, there have not been any tests that would verify its applicability.

Many test programs have indicated that high-strength bolted shear splices subjected to repeated-type loading generally exhibit a significantly higher number of load cycles before failure than do comparable riveted specimens (see Chapter 5). This difference is mainly attributed to the high clamping force provided by the bolts, which results in a more favorable stress distribution around the bolt hole as compared with the the stress flow around the holes in a riveted connection. Hence, the replacement of rivets by high-strength bolts will increase the fatigue strength of a connection.

Fatigue strength tests have been carried out on both small bolted-riveted combination joints[9.2] and on full-size specimens.[14.5] In the latter program, 16 full-scale tests were conducted, including both modeled joints and actual connections taken from a structure in service. The study showed that the replacement of rivets with preloaded high-strength bolts at locations of observed or anticipated cracking increased fatigue life by a factor of from two to six. Proper removal of the rivets to be replaced and proper installation of the replacement bolts is necessary so that no new mechanical flaws (burrs, nicks, and gouges) are introduced during the rehabilitation process. The tests also showed that if cracking is retarded in the critical region by rivet replacement, other locations not as highly stressed may become critical.

Regression analyses of the data were carried out that enabled the prediction of the fatigue strength of the rehabilitated joints.[14.5] For cases involving structural sizes similar to those tested, these could be used. Alternatively, the conservative prediction might be used; that is, rehabilitation of a joint by replacement of rivets with preloaded high-strength bolts will result in a fatigue life twice as great as that of the unrehabilitated joint.

14.3 DESIGN RECOMMENDATIONS

Although only limited test data are available, a knowledge of the behavior of the different fastener responses enables design recommendations to be developed for combination joints that utilize two different types of fasteners to transfer load on a common shear plane.

14.3.1 Static Loading Conditions

For welded-bolted cases in which the load in the joint to be reinforced is small or zero, the capacity can be taken as the sum of the slip resistance of the high-strength bolted part and the ultimate load of the welded part. This summation corresponds to the slip resistance of the connection. If load factor design is being used, the ultimate resistance (separation of the parts) must also be calculated and compared with the force introduced into the joint by the factored loads. For the welded-bolted joint, this will always be the ultimate shear capacity of the bolts or the bearing capacity of the connected parts.

If the welded-bolted combination joint arises as a result of reinforcement under load, then it must be recognized that the original fastening element is already loaded, and only loads applied after the reinforcing connector is introduced will be shared. The identification of the critical fastening element and the joint resistance will have to be handled on an individual basis, considering the deformation and load responses of the individual elements and enforcing compatibility and equilibrium requirements.

Bolted-riveted combination joints will similarly have to be distinguished as to loading case. If the combination is formed under low or zero load, the rivets and bolts can be assumed to share all the applied load. The capacity of the joint will be the sum of the individual contributions. If reinforcement is made under load, usually by the replacement of rivets with high-strength bolts, then the load and deformation originally present in the rivets must be calculated. The load applied after the reinforcement will be carried by the rivets and bolts in proportion to their deformations.

14.3.2 Repeated Loading Conditions

When high-strength bolts and fillet welds are combined to resist forces on a common shear plane, the fatigue strength is governed by the welded joint. Crack growth occurs first from the weld toe termination, and fatigue provisions for the welded detail should be used for design.

When high-strength bolts have been used to strengthen riveted joints, a significant improvement in fatigue strength has been noted when the bolts were placed at the joint ends where the stressed plates are most critical. Regression analyses are available that will enable the prediction or the fatigue strength of the rehabilitated joints.[14.5] Alternatively, it would be conservative to assume that the rehabilitation of a joint by replacement of rivets with preloaded high-strength bolts will give a fatigue life (at the point of rehabilitation) twice as great as that of the unrehabilitated joint. The possibility that other regions of the connection might now become critical must also be considered.

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Chapter Nine

Oversize and Slotted Holes

9.1 INTRODUCTION

Since the first application of high-strength bolts in 1947, bolt holes $\frac{1}{16}$ in. larger than the bolts have been used for assembly. A similar practice was adopted in Europe and Japan, where a hole diameter 2 mm greater than the nominal bolt diameter became standard practice.[9.1]

Restricting the nominal hole diameter to $\frac{1}{16}$ in. in excess of the nominal bolt diameter can impose rigid alignment conditions between structural members, particularly in large joints. Sometimes erection problems occur when the holes in the plate material do not line up properly because of mismatching. Occasionally, steel fabricators must preassemble structures to ensure that the joint will align properly during erection. With a larger hole size, it is possible to eliminate the preassembly process and save both time and money. To determine the feasibility of oversize holes, it was necessary to evaluate the performance of bolted connections with greater amounts of oversize.

An oversize hole provides the same clearance in all directions to meet tolerances during erection. However, if an adjustment is needed in a particular direction, slotted holes can be used, as shown in Fig. 9.1*a* and *b*. Slotted holes are identified by their parallel or transverse alignment with respect to the direction of the applied load (see Fig. 9.1*a* and *b*).

When oversize and slotted holes are used, additional plate material is removed from the vicinity of high clamping forces. The influence of this condition on the behavior of connections has been investigated experimentally.[4.26,8.2,8.3,9.1,9.3] The effect of oversize and slotted holes on such factors as the loss in bolt tension after installation, the slip resistance, and the ultimate strength of shear splices has been examined. Tightening procedures were studied as well. Provisions based on these findings are now included in specifications.[1.4]

9.2 EFFECT OF HOLE SIZE ON BOLT TENSION AND INSTALLATION

The load versus deformation characteristics of joints assembled with high-strength bolts installed in oversize or slotted holes depend, among other factors, on the bolt

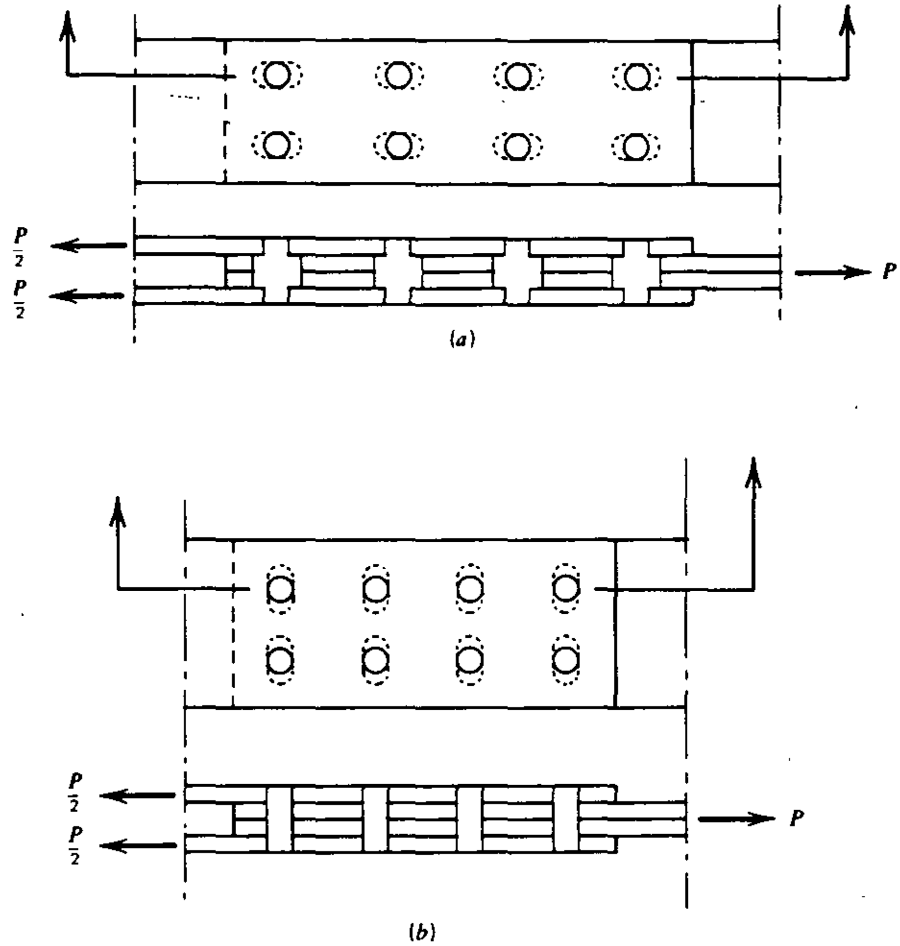


Fig. 9.1. Slotted holes. (*a*) Parallel slotted holes. (*b*) Transverse slotted holes.

clamping force. Hence, it is necessary to examine the effect of varying hole diameters on the bolt installation. This includes the degree of scouring around the hole and the clamping force induced by standard installation procedures. These factors are of primary interest when slip-resistant joints are used.

Tests have indicated that oversize and slotted holes can significantly influence the level of bolt preload when bolts are installed in accordance with common practice.[4.26] This is illustrated in Fig. 9.2, where the observed bolt tension after installation by the turn-of-the-nut method is shown for several different hole clearances.[4.26] The 1-in. dia. A325 bolts installed in 1 $\frac{1}{4}$-in. dia. holes, that is, with $\frac{1}{4}$-in. clearance, showed that the average bolt tension was about the same irrespective of whether or not a washer was used under the nut. The bolt tension attained was about 118% of the required minimum tension. This is about 15% lower than the average tension that is observed in joints with the normal $\frac{1}{16}$-in. clearance (Subsection 5.1.7). Depressions in the plate occurred under the bolt heads during tightening and were greater than the depressions observed with the usual $\frac{1}{16}$-in. hole clearance. Severe galling of both plate and nut occurred with oversize holes when washers were omitted from under the turned element, as is illustrated in Figs. 9.3 and 9.4.[4.26] One-inch diameter bolts installed with only one washer under the turned

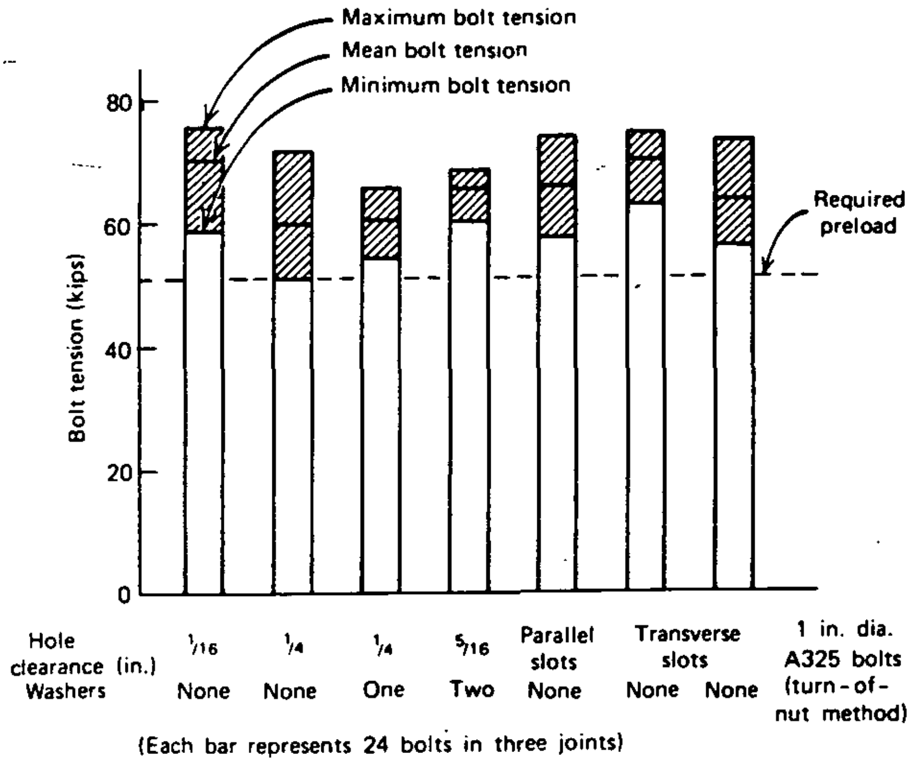


Fig. 9.2. Range of bolt tensions for normal, oversize, and slotted holes.

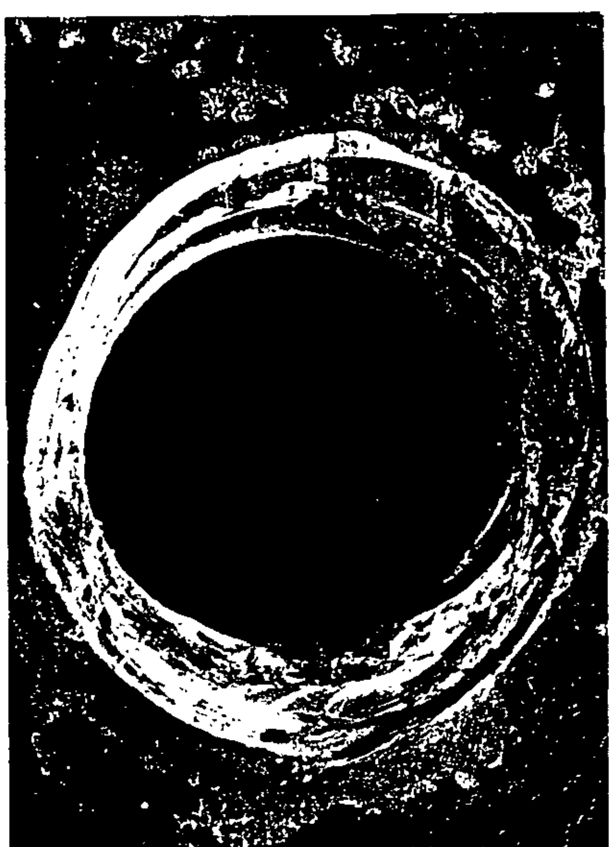


Fig. 9.3. Severe galling of plate under turned element ($\frac{1}{4}$ in. clearance, no washer).

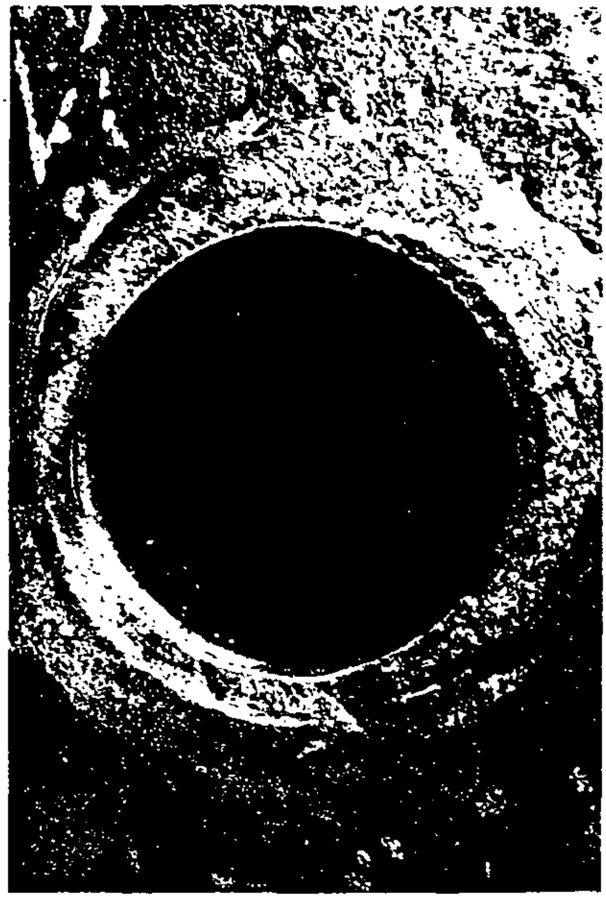


Fig. 9.4. Plate area under element in which washer was used ($\frac{1}{4}$ in. clearance).

element in $1\frac{5}{16}$-in. diameter holes (not shown in Fig. 9.2) failed to achieve their minimum required tension. The bolt heads had recessed severely into the plate around the holes. When washers were placed under both the nut and bolt head, the range of bolt tension achieved ranged from 110 to 144% of the minimum required tension, with an average value of 125%. In other, unpublished, tests, large diameter ($1\frac{1}{8}$-in.) A490 bolts were installed in $\frac{5}{16}$-in. oversize holes. Standard washers were used under both the nut and the bolt head. Although scouring was observed, it was principally dishing of the washers under the very high preload that prevented the specified minimum preload from being attained. Only when thicker washers were used ($\frac{5}{16}$ in.) could the specified minimum preload be obtained in these tests.

The depression of the bolt into the plate or the dishing of the washer means that prescribed rotation of the nut may not produce the required amount of bolt elongation. Consequently, the bolt preload may be less than that specified. In the calibrated wrench procedure, if the deformation characteristic of the calibrator is stiffer than that of the joint with oversize holes, the same problem can arise.

Assuming that the bearing pressure developed under the flat areas of the bolt heads with $\frac{1}{4}$-in. clearance holes is the maximum permitted on A36 steel plate, a theoretical maximum hole clearance for any size bolt can be determined. The area of the plate remaining under the flat of the bolt head must be sufficient so that this

Table 9.1. Hole Clearance for Different Hole Sizes

Bolt Size	Maximum Hole Diameter (in.)	Amount of Clearance
$\frac{1}{2}$	$\frac{11}{16}$	$\frac{3}{16}$
$\frac{5}{8}$	$\frac{13}{16}$	$\frac{3}{16}$
$\frac{3}{4}$	$\frac{15}{16}$	$\frac{3}{16}$
$\frac{7}{8}$	$1\frac{1}{16}$	$\frac{3}{16}$
1	$1\frac{1}{4}$	$\frac{1}{4}$
$1\frac{1}{8}$	$1\frac{7}{16}$	$\frac{5}{16}$
$1\frac{1}{4}$	$1\frac{9}{16}$	$\frac{5}{16}$
$1\frac{3}{4}$	$1\frac{11}{16}$	$\frac{5}{16}$
$1\frac{1}{2}$	$1\frac{13}{16}$	$\frac{5}{16}$

pressure is not exceeded. The results of such computations are summarized in Table 9.1. The hole diameters have been rounded off to the nearest sixteenth of an inch. All of the available test results substantiate that the specified minimum preload can be reached or exceeded for A325 bolts if the hole and bolt diameter combinations shown in Table 9.1 are used. As has already been noted, additional precautions in the form of thicker washers will be necessary for large diameter A490 bolts. Bolts installed by the turn-of-nut method in slotted holes also showed a decrease in the mean bolt tension when compared with similar bolts installed in standard holes with a $\frac{1}{16}$ in. oversize.[4.26] Hence, the use of either oversize or slotted holes is likely to reduce slightly the mean clamping force in the fastener.

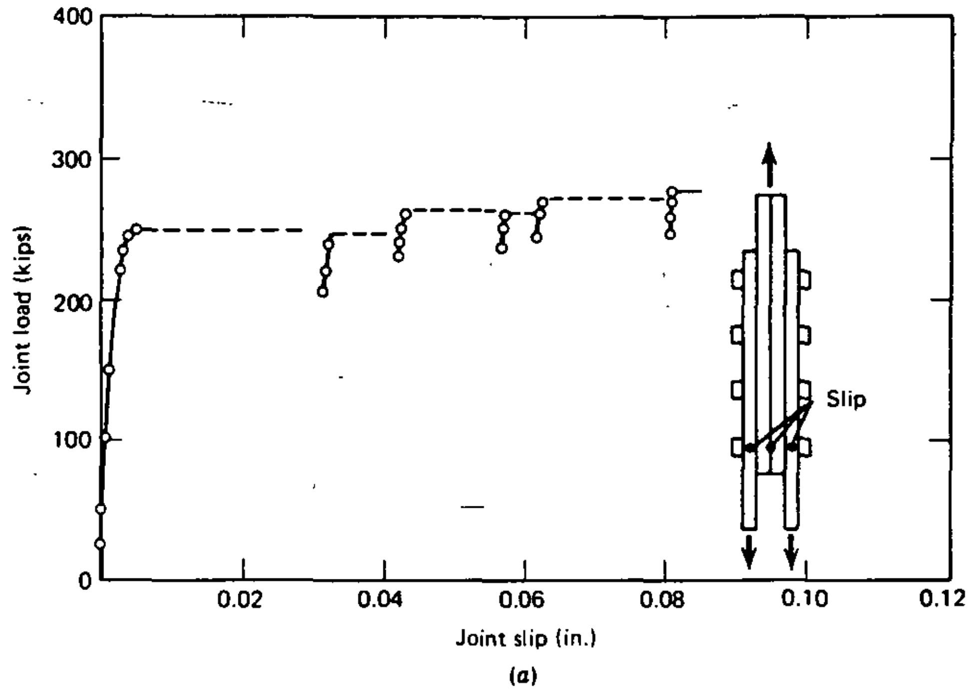
Immediately after a bolt is tightened, a loss in bolt tension occurs. This is thought to result from creep and plastic deformation in the threaded portions and plastic flow in the steel plates under the head and the nut. These deformations result in an elastic recovery and subsequent loss in bolt tension. Studies on bolts installed in holes with a standard hole clearance are summarized in Ref. 4.26 and in Chapter 4. In general, the total loss in preload was about 5 to 10% of the initial preload, depending on grip length (3 to 6 in.) and whether washers were used. Most of the loss in preload occurred within a short time after the bolt was tightened.

A few relaxation tests have been conducted on bolts installed in oversize holes and are reported in Ref. 4.26. It was observed that none of the variations in the hole diameter or the presence of slots had any significant effect on this loss. Virtually all of the losses occurred within 1 week after installation, as was also observed with earlier studies. The loss in tension was observed to be about 8% of the initial preload. This is directly comparable to earlier studies on regular size holes with a standard clearance of $\frac{1}{16}$ in.

9.3 JOINT BEHAVIOR

9.3.1 Slip Resistance

Figure 9.5 shows typical load versus slip relationships of joints with oversize or slotted holes.[4.26] The response is almost linear until the load approaches the major



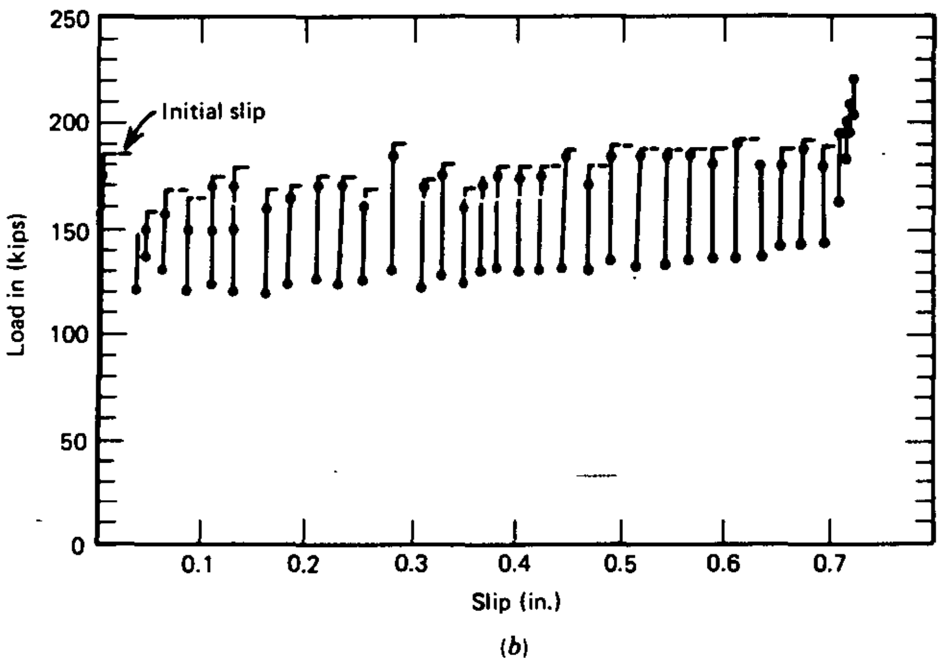


Fig. 9.5. Typical load versus slip diagrams. (*a*) Joint with oversize holes. (*b*) Joint with slotted holes.

slip load. The initial slip was always observed to be less than the amount of hole clearance. Subsequent loading of the joint after major slip had occurred produced small slips until the joint came into bearing. These small slips occurred at loads near the major slip load. The test results shown in Fig. 9.5 were obtained using double shear splices like those illustrated in Fig. 9.1[4.26] The fasteners were 1-in. dia. A325 bolts, and the connected material was A36 steel in the clean mill scale condition. A summary of the observed slip coefficients as a function of the hole geometry for both oversize and slotted hole conditions is shown in Fig. 9.6. It was

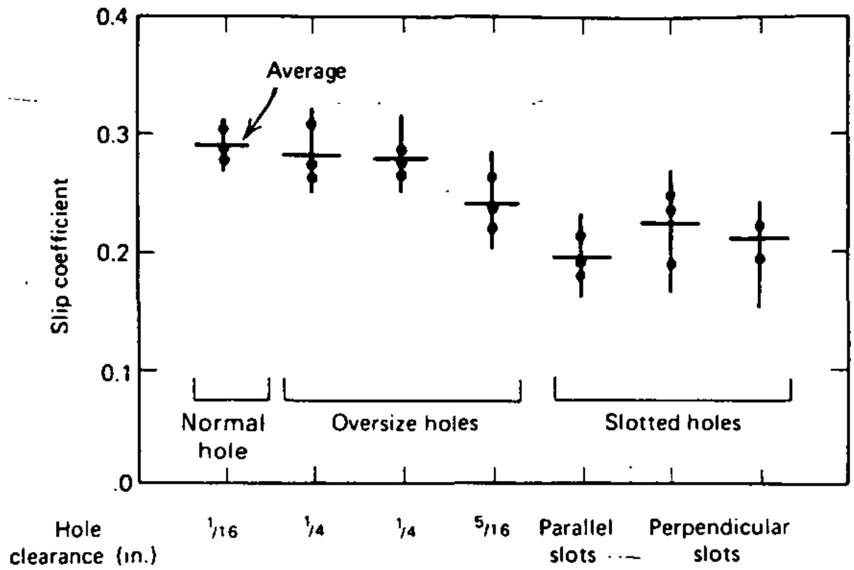


Fig. 9.6. Comparison of average slip coefficients.

concluded that the average slip coefficient for joints with up to $\frac{1}{4}$-in. hole clearance did not change with varying oversize. The joints with $\frac{5}{16}$-in. clearance holes showed a 17% decrease in the slip coefficient for clean mill scale faying surfaces. The slip coefficient for joints with slotted holes showed a 22 to 33% decrease when compared with test specimens with a hole clearance of $\frac{1}{16}$ in. A decrease in slip resistance with the removal of plate material from around the bolt was expected because of the resulting high contact pressures in the area around the bolt. Removal of the plate causes extremely high contact pressures adjacent to the bolt holes that tends to flatten the surface irregularities and thereby reduces the slip resistance of the joint.

The slip resistance is also affected by the decreased clamping force that has been observed in joints with oversize and slotted holes. The combined effects of the change in slip coefficient and the reduction in the clamping force on the slip resistance is estimated to cause a 15% reduction in slip resistance for oversize holes and a 30% reduction for parallel and transverse slotted holes.[4.26]

Major slip of the connection is terminated when one or more bolts come into bearing against the plates. The amount of slip exhibited before bearing occurs depends on the available clearance and fabrication tolerances. Joints with oversize holes or parallel slotted holes may undergo substantial displacements if the slip resistance of the joint is exceeded.

Studies have also been carried out to evaluate the influence of oversize holes upon the slip resistance of blast-cleaned and coated surfaces.[9.3] This work showed that, for holes up to $\frac{1}{4}$-in. greater in diameter than the bolt diameter, there was no significant effect of hole oversize on the slip coefficient. (Further work with sandblasted surfaces showed that the surface roughness of the A572 steel surfaces did not significantly affect the slip coefficient, and that sandblasting time did not affect the slip coefficient for A36, A572, and A514 steels tested. These tests were carried out using joints with holes of normal clearance.)

The painted surfaces examined included organic zinc primer, with or without an epoxy topcoat, and inorganic zinc primer with a vinyl topcoat. The specified primer thickness was 6 mils and that of the topcoat was 3 mils. This part of the study again found that holes up to $\frac{1}{4}$ in. greater in diameter than the $\frac{7}{8}$-in. diameter bolts did not affect the slip resistance of the joints.

Although joints with slotted holes were not examined in this study, it is reasonable to expect that their slip behavior would be similar to that displayed by the coated or blast-cleaned surfaces containing oversize holes.

9.3.2 Ultimate Strength

The ultimate strength of a connection is governed by either the shear capacity of the bolts or the tensile capacity of the plates. The effect of oversize holes or slotted holes on the ultimate strength can be evaluated by examination of the limiting case, transverse slotted holes. Tests have shown that the presence of transverse slotted holes does not result in a reduction of the tensile strength of the plates or of the shear strength of the fasteners.[4.26] Hence, the ultimate strength of a joint can be assumed to be unaffected by either oversize or slotted holes.

9.4 DESIGN RECOMMENDATIONS

Since the ultimate strength of a joint with oversize or slotted holes is the same as the ultimate strength of a similar standard type connection with identical bolt and plate areas, the design recommendations given in Chapter 5 are applicable. The provisions given there for both plate material and bolts of bearing-type shear splices are applicable also to joints with oversize or slotted holes. Care must be exercised when using oversize or slotted holes to ensure that excessive deformation will not occur at working loads. The slots should be oriented so that large displacements cannot result. Transverse slotted holes are preferable, since they limit the slip to the same magnitude that can be experienced with standard hole clearances.

Design recommendations for slip-resistant joints with oversize or slotted holes must reflect the reduced slip resistance. Hole diameters that do not exceed those given in Table 9.1 do not significantly alter the slip coefficient. However, the clamping force is reduced by about 15%, and this must be reflected in the slip resistance and design conditions. A factor 0.85 can be used to provide for the reduced clamping force and its effect on the slip resistance. For slip-resistant joints with slotted holes, a reduction factor of 0.70 will account for the loss in slip resistance caused by either parallel or slotted holes.

To prevent the use of extremely large slotted holes, present specifications limit the length of slotted holes to 2 $\frac{1}{2}$ times the bolt diameter. (These are defined as long slotted holes.) The width of the hole should not exceed the bolt diameter by more than $\frac{1}{16}$ in. Short slotted holes are also used. Short slotted holes are $\frac{1}{16}$ in. wider than the bolt diameter and have a length that does not exceed the allowable oversize diameter for that bolt size by more than $\frac{1}{16}$ in. Joints with short slotted holes will develop the same slip resistance as joints with oversize holes. Therefore, the design of joints with oversized or short slotted holes is the same.

To achieve an adequate clamping force in the bolts, washers should be used under both the bolt head and the nut when oversize or slotted holes occur in the outside plates of a joint. Special requirements are necessary for large diameter A490 bolts.

DESIGN RECOMMENDATIONS FOR OVERSIZE AND SLOTTED HOLES

Hardened washers are to be inserted under both the head and the nut if oversize or slotted holes are placed in the outside plies of a connection. A490 bolts with diameters greater than 1 in. should have at least $\frac{5}{16}$-in. thickness material under both the head and the nut in order to bridge over a slotted or oversize hole. (Use of multiple washers to make up the thickness will not be satisfactory.) If this additional material is hardened, no washers will be necessary. However, if ordinary structural steel plate is used, standard hardened washers should be added under both the nut and bolt head.

Slip-Resistant Joints

$P_s' = 0.85\ P_s$ for oversize and short slotted holes not exceeding the dimensions given in Table 9.2

$P_s' = 0.70\ P_s$ for long slotted holes not exceeding the dimensions given in Table 9.2

where P_s is the slip load described in Subsection 5.4.2 for joints using holes of normal clearance.

For coated surfaces, the design recommendations given in Section 12.5 should be similarly modified if slotted or oversize holes are present.

Table 9.2 Standard, Oversize, and Slotted Hole Dimensions

	Hole Dimensions			
Bolt Diam	Standard (Diam)	Oversize (Diam)	Short Slot (Width × Length)	Long Slot (Width × Length)
$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{9}{16} \times \frac{11}{16}$	$\frac{9}{16} \times 1\frac{1}{4}$
$\frac{5}{8}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d+\frac{1}{16}$	$d+\frac{5}{16}$	$(d+\frac{1}{16})\times(d+\frac{3}{8})$	$(d+\frac{1}{16})\times(2.5\times d)$

Chapter Four

Bolts

4.1 BOLT TYPES

The types of bolts used in connecting structural steel components in buildings and bridges can be categorized as follows (see Section 1.3):

1. Low carbon steel bolts and other fasteners, ASTM A307, grade A
2. High-strength medium carbon steel bolts, ASTM A325, plain finish, weathering steel finish, or galvanized finish
3. Alloy steel bolts, ASTM A490
4. Special types of high strength bolts such as interference body bolts, swedge bolts, and other externally threaded fasteners or nuts with special locking devices, ASTM A449 and ASTM A354 grade BD bolts.

The only marking requirement for ASTM A307 bolts is that the manufacturer's symbol appear on top of the head of the bolt.[1.10] A307 bolts are manufactured with a hexagonal head and nut and either a regular or heavy head, depending on the bolt diameter. Nuts do not need to be marked. The bolts are produced in diameters ranging from $\frac{1}{4}$ to 4 in., have a specified minimum tensile strength of 60 ksi, and may be galvanized.

In application, A307 bolts and nuts are tightened so that some axial force is present that will prevent movement of the connected members in the axial direction of the bolt. Proper tightening also prevents loosening of the nut. The actual force in the bolt is not closely controlled and may vary substantially from bolt to bolt. Because of the small axial forces, little frictional resistance is developed, and in most situations the bolt will slip into bearing. This results in shear stresses in the bolts and contact stresses at the points of bearing.

High-strength bolts are heat treated by quenching and tempering. The most widely used are A325 high-strength carbon steel bolts[1.3] and A490 alloy steel bolts.[1.9]

The A325 bolt is manufactured in diameters ranging from $\frac{1}{2}$ to $1\frac{1}{2}$ in. and is provided as Type 1 (made of medium carbon steel), Type 2 (low-carbon martensite steel), or Type 3 (atmospheric corrosion-resistant steel). Types 1 and 2 can be galvanized. The specified minimum tensile strength for all three types is 120 ksi for bolt diameters up to and including 1 in. and 105 ksi for diameters from $1\frac{1}{8}$ to $1\frac{1}{2}$ in. The bolt heads of all types must be marked A325 and shall also have the manufacturer's symbol. Additional markings distinguish among the three bolt types (see Fig. 4.1). Nut and washer markings are shown in Fig. 4.1. A metric specification is also available for ASTM A325 bolts.[4.30]

Bolts manufactured to ASTM Specification A490 can also be one of three types. Type 1 bolts are made from alloy steel, Type 2 are of low-carbon martensite steel, and Type 3 are of atmospheric corrosion-resistant steel. The bolts are manufactured in diameters ranging from $\frac{1}{2}$ to $1\frac{1}{2}$ in. for all three types, and the specified minimum tensile strength is 150 ksi for all bolts made under this specification. A490 bolts should not be galvanized since they become susceptible to stress corrosion cracking and hydrogen embrittlement.

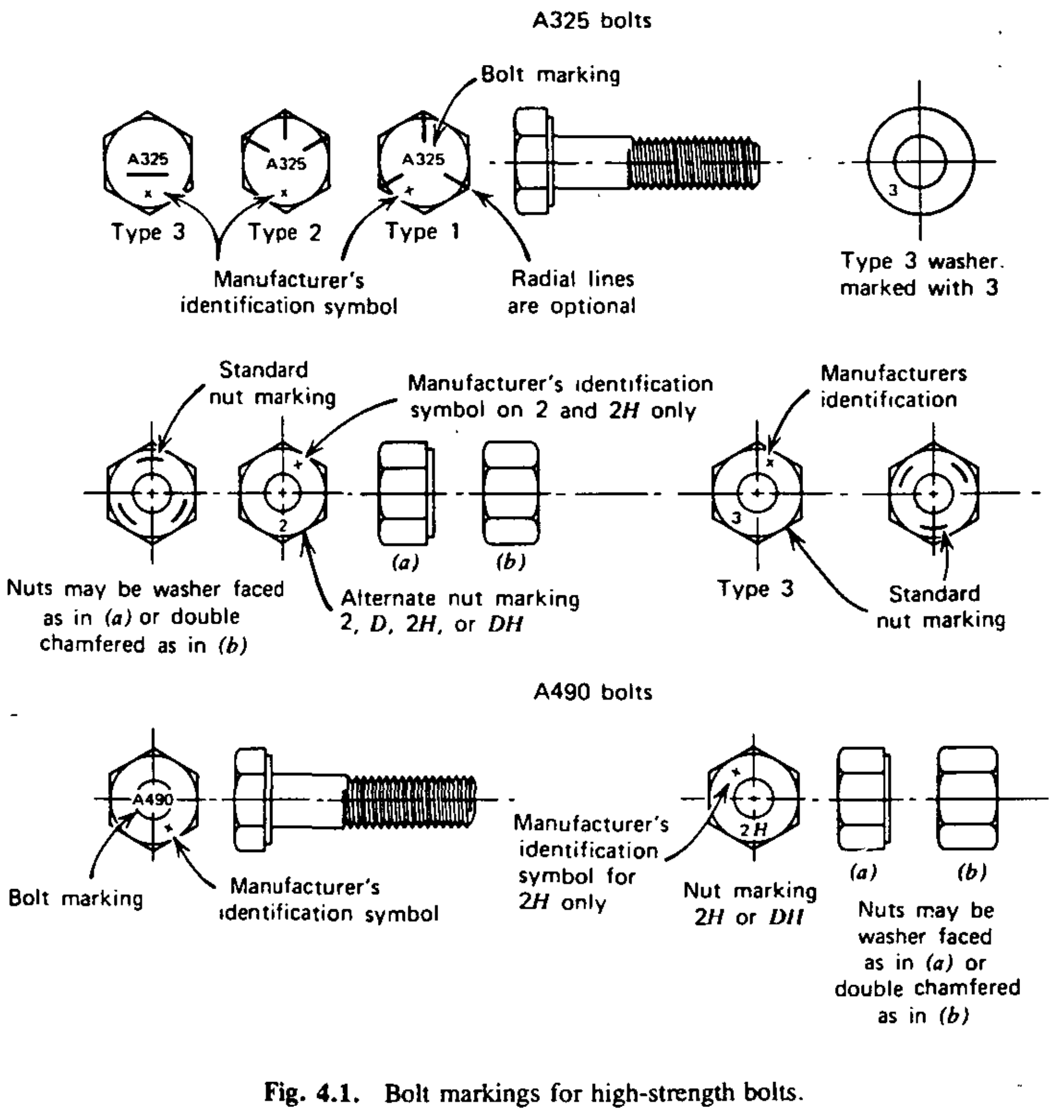


Fig. 4.1. Bolt markings for high-strength bolts.

The markings for A490 bolts are also shown in Fig. 4.1. Bolt heads must be marked with both A490 and the manufacturer's symbol. Other marks, dependent on the bolt type, also appear.

Nuts for A325 bolts must be heavy hex and are required to meet ASTM Specification A563. For bolt Types 1 and 2, plain (uncoated) nut grade C, plain finish, should be used. For bolt Types 1 and 2, galvanized, nut grade DH, galvanized, is required. Nut grade C3 is to be used with bolt Type 3. Grades 2 and 2H nuts, as specified in ASTM A194, and grades D and DH nuts, as specified in ASTM A563, are acceptable alternatives for grade C nuts. Grade 2H nuts (ASTM A194) are an acceptable alternative for grade DH nuts, and type DH3 nuts can be used in place of C3 nuts.

Heavy hex nuts are also required for A490 bolts. Grade DH heavy hex nuts shall be furnished for use with Type 1 and 2 bolts, but grade 2H heavy hex nuts (ASTM A194) are also acceptable. Type 3 A490 bolts require grade DH3 (ASTM A563) heavy hex nuts.

Nuts are marked in various ways, as shown in Fig. 4.1. It should also be noted that both ASTM specifications for high-strength bolts, A325 and A490, stipulate that they are intended for use in structural connections that conform to the RCSC Specification.[1.4]

In addition to the standard A325 and A490 bolts $\frac{1}{2}$ through $1\frac{1}{2}$ in. diameter, short thread heavy head structural bolts above $1\frac{1}{2}$ in. diameter and other types of fasteners and fastener components are available. These are covered by the general bolting specifications A449 and A354. Specification A449 covers externally threaded fastener products with mechanical properties similar to A325. The A354 grade BD covers externally threaded fastener parts that exhibit mechanical properties similar to A490.

Among the special types of fasteners or fastener components are the interference body bolts, swedge bolts, tension-control bolts, and bolt and nut combinations in which the nuts have special locking devices. The interference body bolt (see Fig. 4.2) meets the strength requirements of the A325 bolt and has an axially ribbed shank that develops an interference fit in the hole and prevents excessive slip. A swedge bolt, shown in Fig. 4.3 consists of a fastener pin made from medium carbon steel and a locking collar of low carbon steel. The pin has a series of annular locking grooves, a breakneck groove, and pull grooves. The collar is cylindrical in shape and is swaged into the locking grooves in the tensioned pin by a hydraulically operated driving tool that engages the pull grooves on the pin and applies a tensile force to the fastener. After the collar is fully swaged into the locking grooves, the pin tail section breaks at the breakneck groove when its preload capacity is reached.

Like the swedge bolt, the tension-control bolt is installed by working from one side only, and only one person is required to install the bolt. A special wrench contains a two-part socket that both turns the nut and holds the bolt by means of a splined bolt end. The spline is present toward the end of an extension of the bolt shaft beyond the nut end. This extension also contains a circular notch ("torque control groove") that is calibrated to shear at a torque that will ensure that the

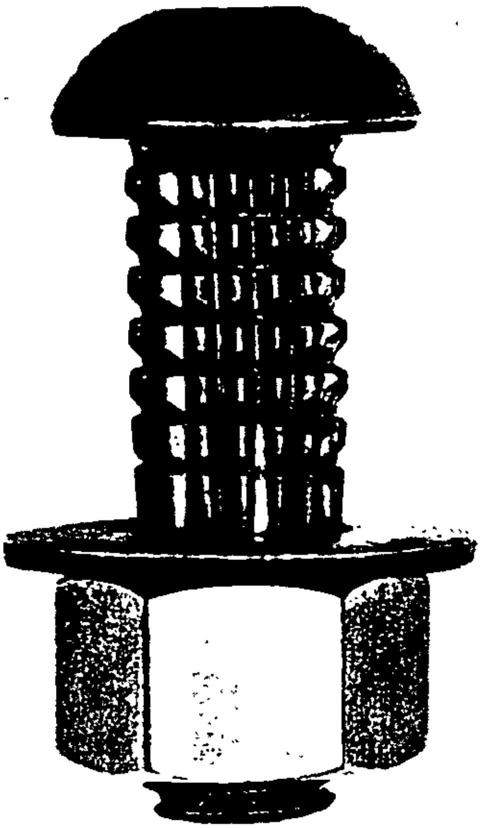


Fig. 4.2. Interference body bolt. (Courtesy of Bethlehem Steel Corp.)

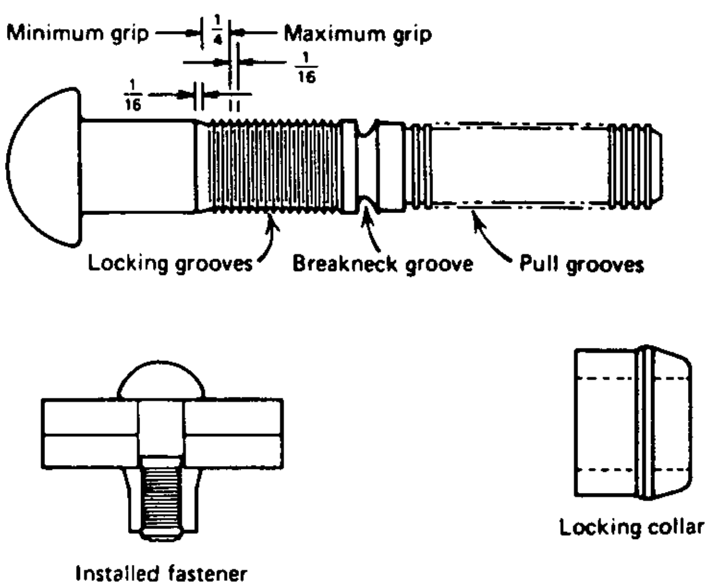


Fig. 4.3. High tensile swedge bolt.

required bolt tension is reached. Installation is quieter than that for a conventional bolt (electric wrenches rather than air-operated impact wrenches are used). Inspection is visual and is simply an observation that the tips have been sheared off. Bolt costs are higher than for conventional high-strength bolts, however, and disposal of the sheared tips may present safety problems.

It should be noted that both swedge bolts and tension-control bolts could be difficult to remove in situations where a structure was being altered or dismantled because they use a rounded head on the bolt.

4.2 BEHAVIOR OF INDIVIDUAL FASTENERS

Connections are generally classified according to the manner of stressing the fastener (see Section 2.2), that is, tension, shear or combined tension and shear. Typical examples of connections subjecting fasteners to shear are splices and gusset plates in trusses. Bolts in tension are common in hanger connections and in beam-to-column connections. Some beam-to-column connections may also subject the bolts to combined tension and shear. It is apparent that, before a connection can be analyzed, the behavior of the component parts of the connection must be known. Therefore, the behavior of a single bolt subjected to the typical loading conditions of tension, shear, or combined tension and shear is discussed in this section.

4.2.1 Bolts Subjected to Tension

Since the behavior of a bolt subjected to an axial load is governed by the performance of its threaded part, load versus elongation characteristics of a bolt are more significant than the stress versus strain relationship of the fastener metal itself.

In the 1985 ASTM specifications for high-strength bolts, both the minimum tensile strength and proof load are specified.[1.3,1.9] The proof load is about equivalent to the yield strength of the bolt or the load causing 0.2% offset. To determine the actual mechanical properties of a bolt, ASTM requires a direct tension test of most sizes and lengths of full-size bolts. In practice, the bolt preload force is usually introduced by tightening the nut against the resistance of the connected material. As this torque is applied to the nut, the portion not resisted by friction between the nut and the gripped material is transmitted to the bolt and, due to friction between bolt and nut threading, induces torsional stresses into the shank. This tightening procedure results in a combined tension-torsional stress condition in the bolt. Therefore, the load versus elongation relationship observed in a torqued tension test differs from the relationship obtained from a direct tension test. Specifically, torquing a bolt until failure results in a reduction in both ultimate load and ultimate deformation as compared with the corresponding values determined from a direct tension test. Typical load versus elongation curves for direct tension as well as torqued tension tests are shown in Fig. 4.4 for A325 bolts and A490 bolts. In torquing a bolt to failure, a reduction in ultimate strength of between 5 and 25% was experienced in tests on both A325 and A490 bolts.[4.1–4.3] The average reduction

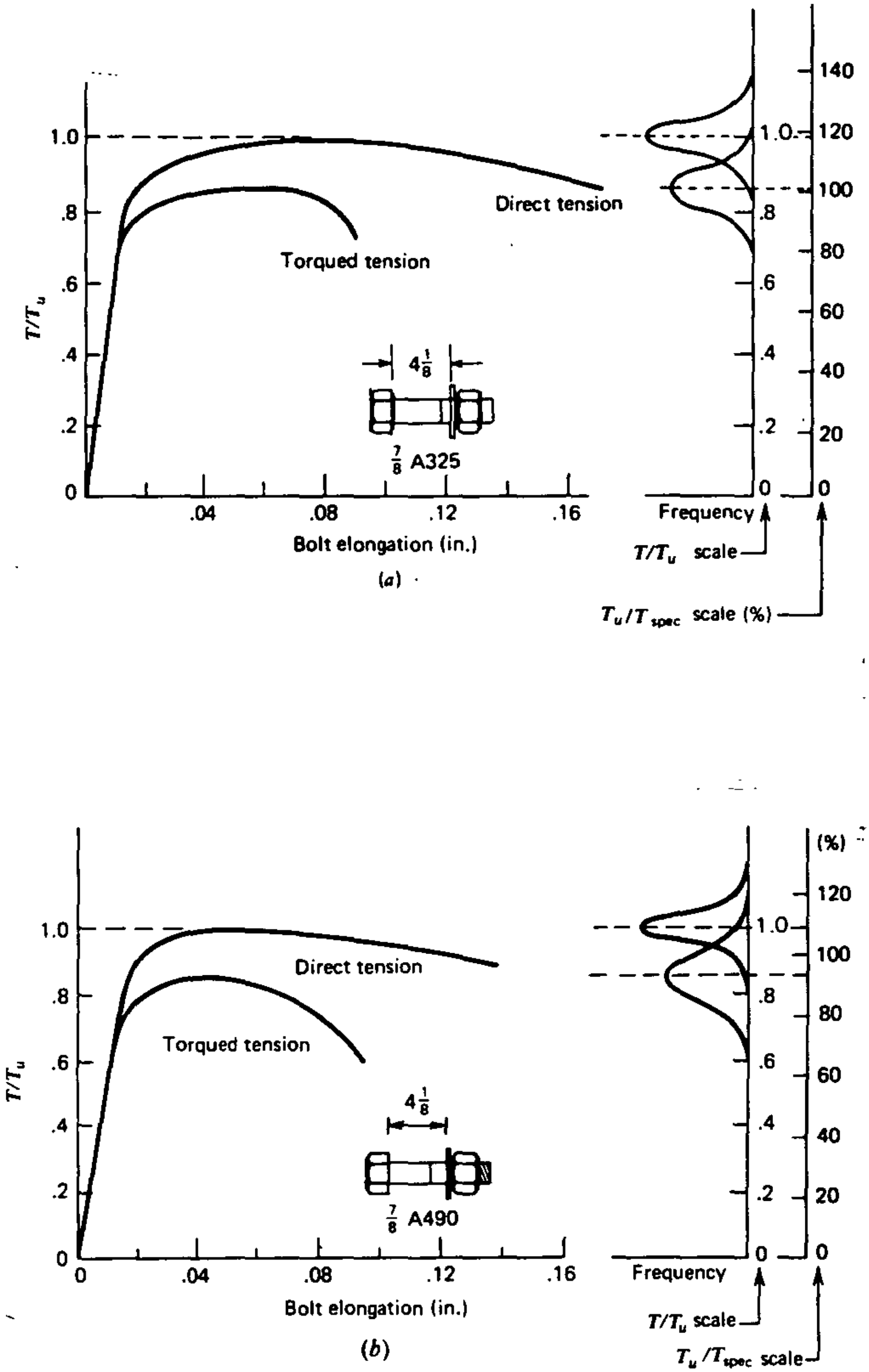


Fig. 4.4. (*a*) Load versus elongation relationship and frequency distribution of A325 bolts tested in torqued tension and direct tension; (*b*) A490 bolts.

is equal to 15%. Frequency distributions of the ratio T/T_u for both A325 and A490 bolts are also shown in Fig. 4.4.

As well as having a higher load, a bolt loaded to failure in direct tension also has more deformation capacity than one that is failed in torqued tension.[4.1–4.3] This is visible in the two specimens shown in Fig. 4.5. The differences in thread deformation and necking of the critical section in the threaded part of the bolts are readily apparent.

To determine whether specified minimum tensile requirements are met, specifications require direct tension tests on full-size bolts if the bolts are longer than three diameters or if the bolt diameter is less than $1\frac{1}{4}$ in. for A325 bolts or 1 in. for A490 bolts. Bolts larger in diameter or shorter in length shall preferably be tested in full size; however, on long bolts tension tests on specimens machined from such bolts are allowed. Bolts shorter than three diameters need only meet minimum and maximum hardness requirements. Tests have illustrated that the actual tensile strength of production bolts exceeds the minimum requirements considerably. An analysis of data obtained from tensile tests on bolts shows that A325 bolts in sizes $\frac{1}{2}$ through 1 in. exceed the minimum tensile strength required by 18%. The standard deviation is equal to 4.5%. For larger diameter A325 bolts (1 to $1\frac{1}{2}$ in.), the range of actual tensile strength exceeds the minimum by an even greater margin. A similar analysis of data obtained from tensile tests on A490 bolts shows an average actual strength 10% greater than the minimum prescribed. The standard variation is equal to 3.5%. Frequency distribution curves of the ratio T_u/T_{spec} are shown in Fig. 4.4a for A325 and in Fig. 4.4b for A490 bolts. Compared with the A325, the A490 bolts show a smaller margin beyond the specified tensile strength because specifications require the actual strength of A490 bolts to be within the range of 150 to 170 ksi, whereas for A325 only a minimum strength is specified.

Loading a bolt in direct tension after having preloaded it by tightening the nut (torqued tension) does not significantly decrease the ultimate tensile strength of the bolt, as illustrated in Figs. 4.6 and 4.7. The torsional stresses induced by torquing the bolt apparently have a negligible effect on the tensile strength of the bolt. This means that bolts installed by torquing can sustain direct tension loads without any apparent reduction in their ultimate tensile strength.[4.1,4.2]

Mean load versus elongation curves for 15 regular head, $\frac{7}{8}$-in. dia. A325 bolts of various grips are plotted in Fig. 4.8.[4.2] The thickness of the gripped material varied from approximately $4\frac{3}{4}$ to $6\frac{3}{4}$ in., and the length of thread under the nut varied from $\frac{3}{4}$ to 1 in. No systematic variation existed among the load versus elongation relationships for the different grip conditions. Most of the deformation occurs in the threaded portion between the underside of the nut and the unthreaded part of the bolt. Because this length is relatively constant, the grip length has no appreciable effect on the load versus elongation response. The behavior shown in Fig. 4.8 for the direct tension test was also observed during torqued tension tests. With shorter grip lengths, the effect of bolt length is more pronounced.

Figure 4.8 also shows that, within the elastic range, the elongation increases

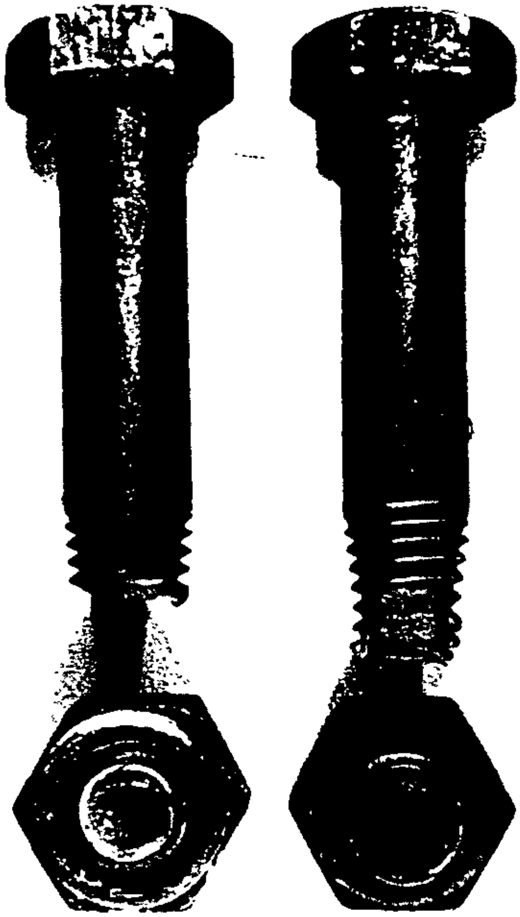


Fig. 4.5. Comparison of torqued tension and direct tension failures.

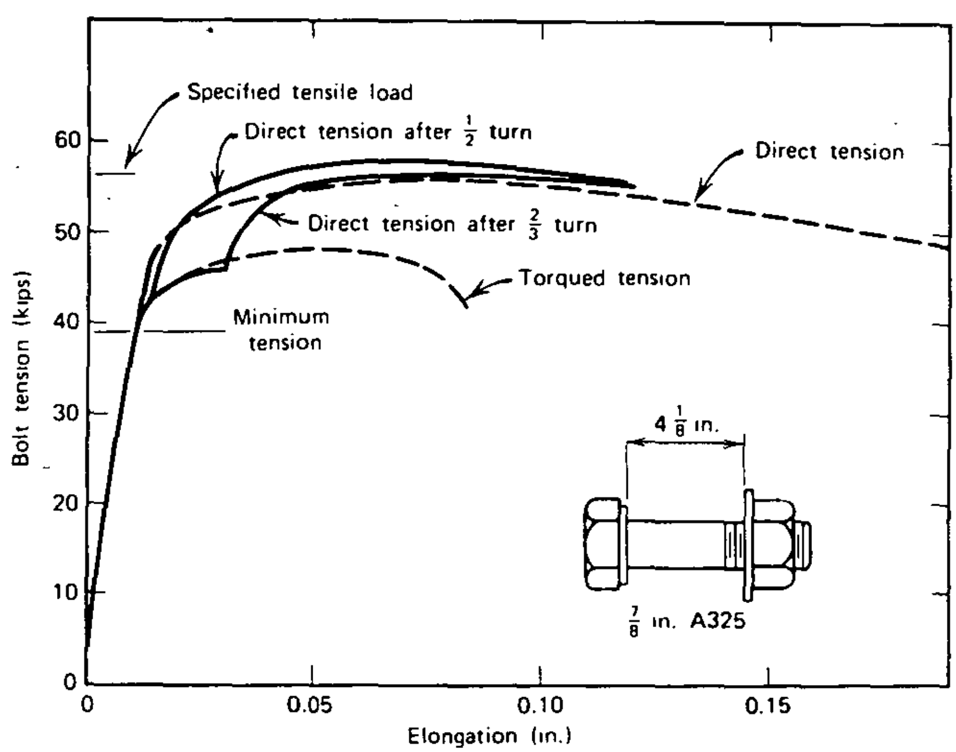


Fig. 4.6. Reserve tensile strength of torqued A325 bolts.

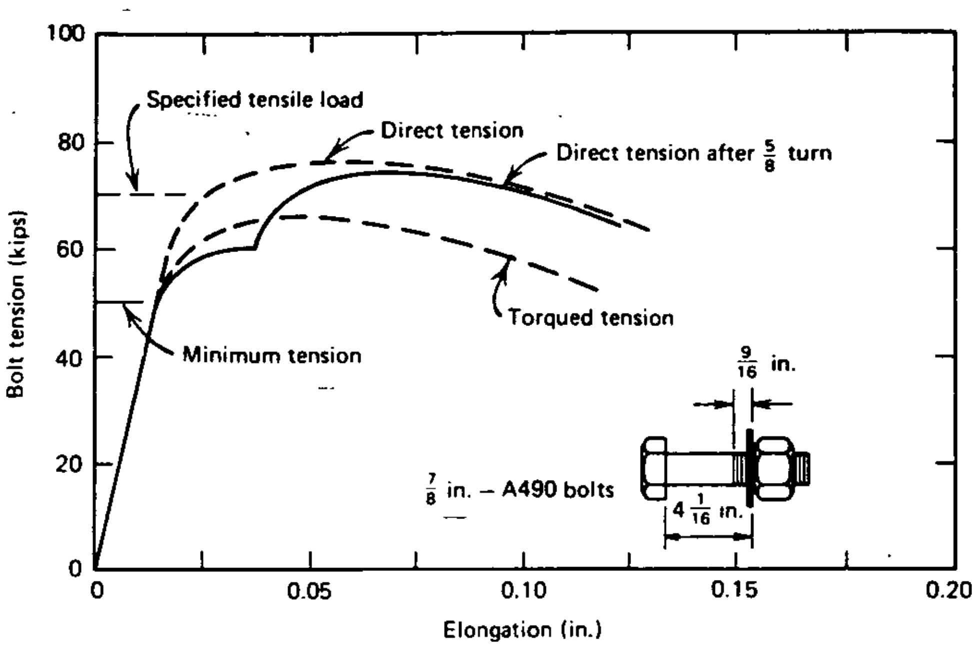


Fig. 4.7. Reserve tensile strength of torqued A490 bolts.

slightly with an increase in grip. As the load is increased beyond the elastic limit, the threaded part, which is approximately of uniform length, behaves plastically, while the shank remains essentially elastic. Hence, when there is a specific amount of thread under the nut, grip length has little effect on the load versus elongation relationship beyond the proportional limit. For short bolts, nearly all deformation occurs in the threaded length, with a resultant decrease in rotational capacity.

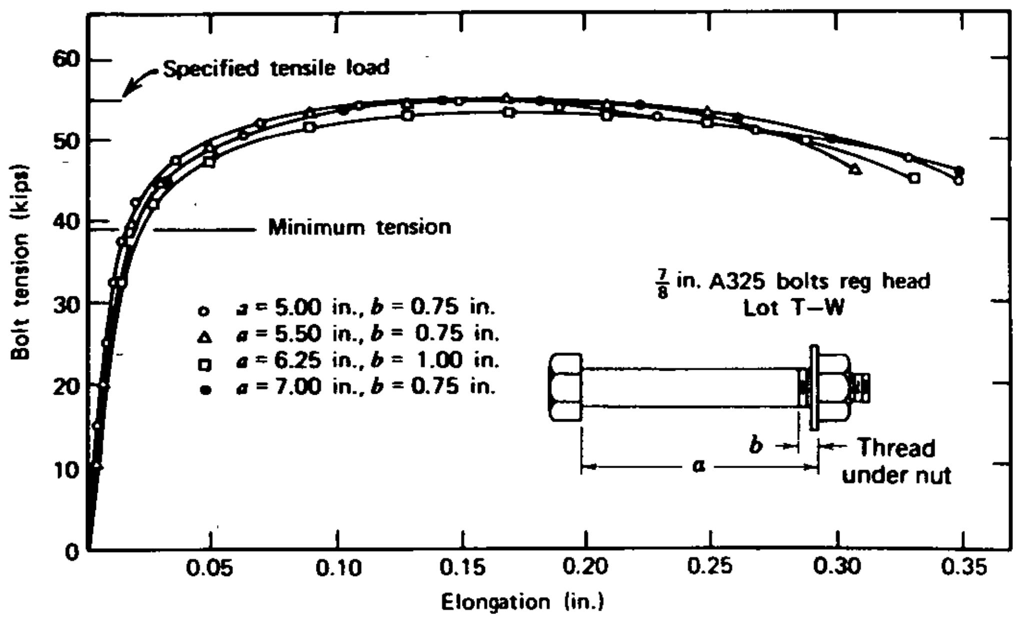


Fig. 4.8. Effect of grip length, direct tension.

A325 bolts with heavy hex heads demonstrate behavior similar to that of bolts with regular heads for grips ranging from 4 to 8 in. and with thread lengths under the nut ranging from $\frac{1}{8}$ to $\frac{3}{4}$ in. Similar observations have also been made about A490 bolts.[4.1,4.3] (Both A325 and A490 bolts are customarily furnished with heavy hexagonal heads unless other dimensional requirements have been agreed on.)

Since most of the elongation occurs in the threads, the length of thread between the thread run-out and the face of the nut will affect the load versus elongation relationship. The heavy head bolt has a short thread length, whereas the regular head bolt has the normal ASA thread length specified by ANSI standards. As a result, for a given thickness of gripped material, the heavy head bolt shows a decrease in deformation capacity, as illustrated in Fig. 4.9.[4.2]

4.2.2 Bolts Subjected to Shear

Shear load versus deformation relationships have been obtained by subjecting fasteners to shear induced by plates either in tension or compression. Typical results of shear tests on A325 and A490 bolts are shown in Fig. 4.10. As expected, the increased tensile strength of A490 bolts as compared with A325 bolts results in an increased shear strength for that fastener. A slight decrease in deformation capacity is evident as the strength of the bolt increases.[4.4]

The shear strength is influenced by the type of test. The fastener can be subjected to shear by plates in tension or compression, as illustrated in Fig. 4.11. The influence of the type of test on the bolt shear and deformation capacity is illustrated in Fig. 4.12, where typical shear stress versus deformation curves are compared for bolts from the same lot that were tested in both tension and compression jigs.[4.4]

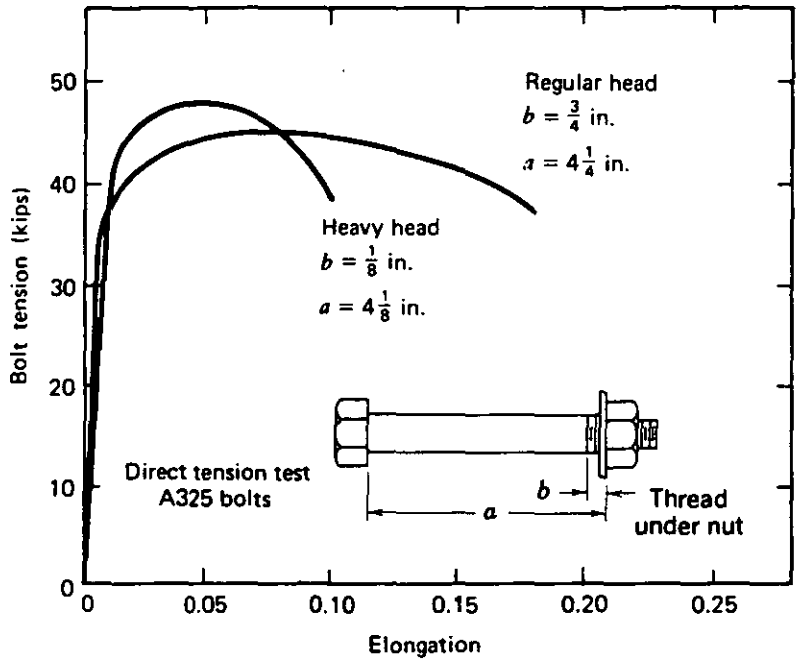


Fig. 4.9. Comparison of regular and heavy head A325 bolts.

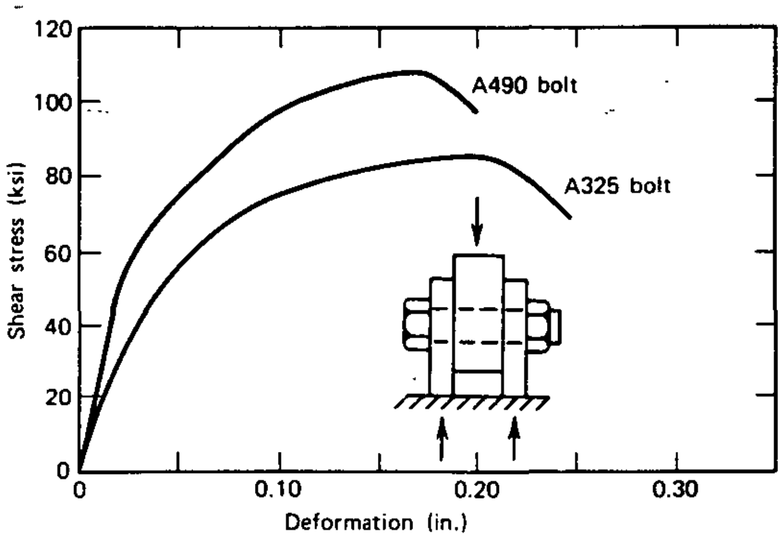


Fig. 4.10. Typical shear load versus deformation curves for A325 and A490 bolts.

Test results show that the shear strength of bolts tested in A440 steel tension jigs is 6 to 13% lower than bolts tested in A440 steel compression jigs.[4.4] The same trend was observed in constructional alloy steel jigs where the reduction in shear strength of similar bolts varied from 8 to 13%. The average shear strengths for A325 and A490* bolts tested in tension jigs were 80.1 and 101.1 ksi, respectively. These shear strengths correspond to about 62% of the respective actual tensile strengths of single bolts. The same bolt grades tested in compression jigs yielded shear strengths of 86.5 and 113.7 ksi, respectively (68% of the bolt tensile strength).[4.4]

The lower shear strength of a bolt observed in a tension type shear test as compared with a compression type test (see Fig. 4.12) is the result of lap plate prying action, a phenomenon that tends to bend the lap plates of the tension jig outward.[4.4,4.25] Because of the uneven bearing deformations of the test bolt, the resisting force does not act at the centerline of the lap plate. This produces a moment that tends to bend the lap plates away from the main plate and thereby causes tensile forces in the bolt.

Catenary action, resulting from bending in bolts, may also contribute to the increase in bolt tension near ultimate load. However, it is believed that this effect is small in comparison with the tension induced by lap plate prying action.[4.25] In any case, the catenary action is present in both the tension and compression jigs.

The tension jig is recommended as the preferred testing device because it produces a lower bound shear strength. Bolts in tension splices are subjected to shear in a similar manner. The tension jig shear test also yields the most consistent test results.

An examination of available test data indicates that the ratio of the shear strength

*Actually, A354 grade BD bolts were used instead of A490 bolts because of their similarity in mechanical properties. At the time of the tests, the A490 bolt was still under development.

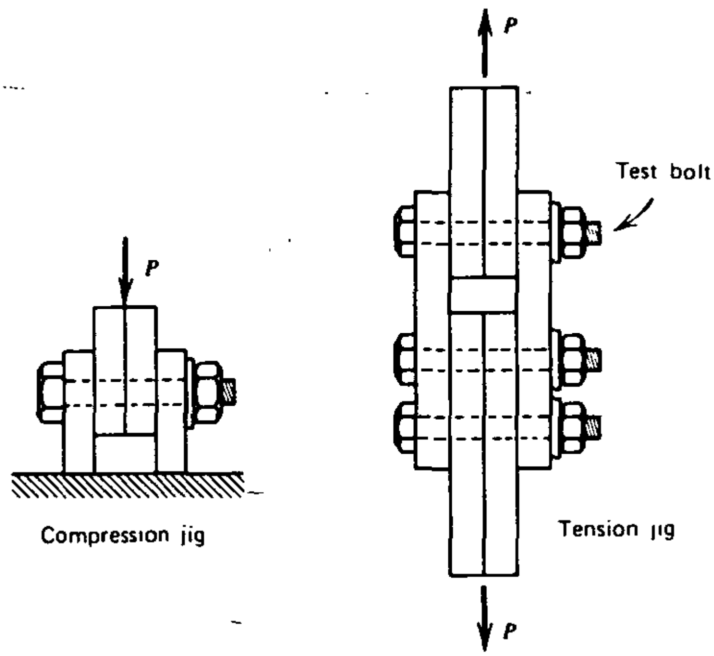


Fig. 4.11. Schematic of testing jigs for single bolts.

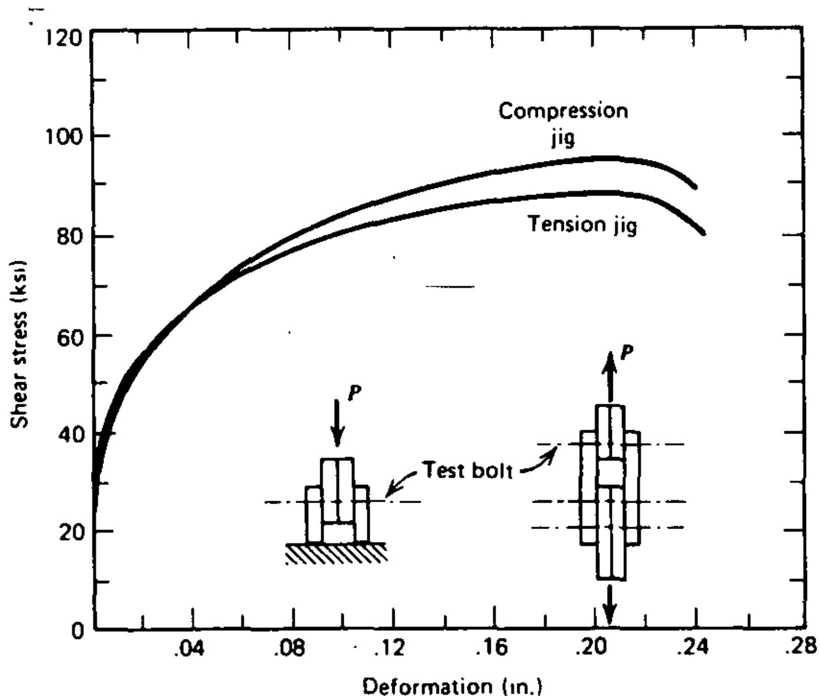


Fig. 4.12. Typical shear load versus deformation curves; A354 BC bolts tested in A440 steel tension and compression jigs.

to the tensile strength is independent of the bolt grade, as illustrated in Fig. 4.13. The shear strength is plotted versus the tensile strength for various lots of A325 and A490 bolts. The average shear strength is approximately 62% of the tensile strength.

The variance of the ratio of the shear strength to tensile strength, as obtained from single bolt tension shear jigs, is shown in Fig. 4.14. A frequency curve of the ratio of shear strength to tensile strength was developed from test data acquired at the University of Illinois and Lehigh University. The average value is equal to 0.62, with a standard deviation of 0.03.

Tests on bolted joints indicated that the initial clamping force had no significant effect on the ultimate shear strength.[4.5–4.7] A number of tests were performed on A325 and A490 bolts torqued to various degrees of tightness and then tested to failure in double shear. The results of tests with A490 bolts are shown in Fig.

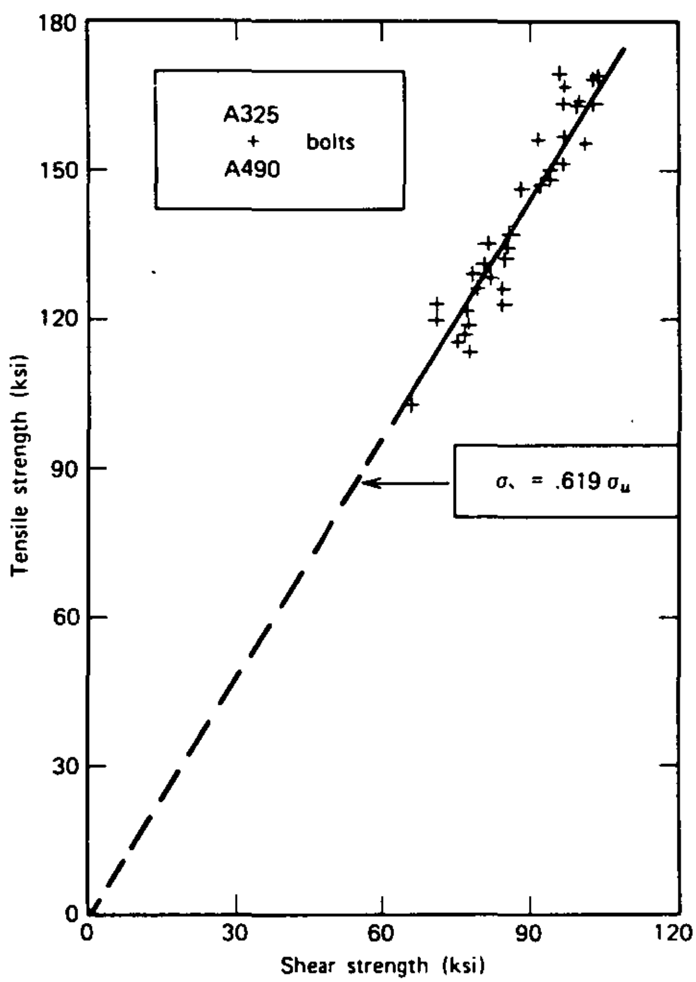


Fig. 4.13. Shear strength versus tensile strength. *Note.* Each point represents the average values of a specific bolt lot. The shear strength is computed on the relevant area, depending on the location of the shear plane.

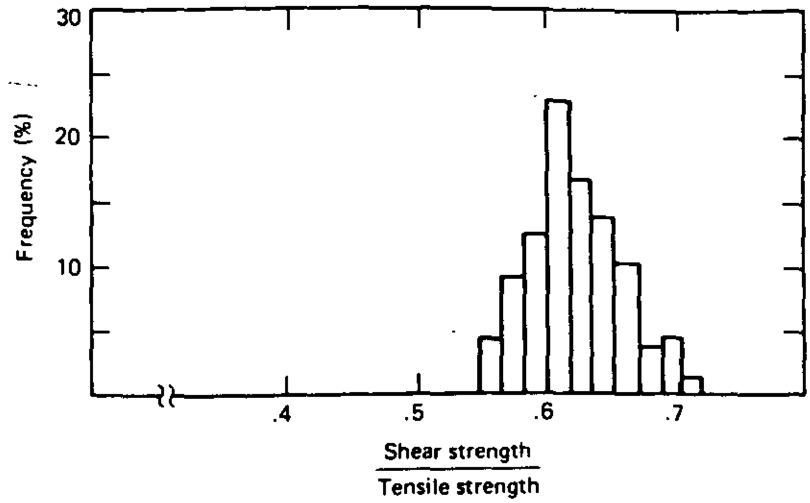


Fig. 4.14. Frequency distribution of ratio of shear strength to tensile strength for A325 and A490 bolts. Number of tests, 142; average value, 0.62; standard deviation, 0.03.

4.15.[4.4] The lower portion shows the relationship between bolt shear strength and the initial bolt elongation after installation. The bolt preload was determined from measured elongations and the torqued tension relationship given in the upper portion of Fig. 4.15. The results confirm that no significant variation of shear strength occurred when the initial bolt preload was varied.

There are two sources of tensile load in the bolt that should, theoretically, interact with the shear load and result in a failure load that is less than that from shear alone. These are (1) the bolt preload induced during the installation procedure, and (2) bolt tension resulting from prying action in the plates.

Measurements of the internal tension in bolts in joints have shown that at ultimate load there is little preload left in the bolt.[4.6,4.7,4.25] The shearing deformations that have taken place in the bolt prior to its failure have the effect of releasing the rather small amount of axial deformation that was used to induce the bolt preload during installation.

At any level of load producing shear in the bolts, prying action of the plates can also produce an axial tensile load in the bolts. In most practical situations, however, the tensile stress induced by prying action will be considerably below the yield stress of the bolt; therefore, it has only a minor influence. Studies of bolts under combined tension and shear have shown that tensile stresses equal to 20 to 30% of the tensile strength do not significantly affect the shear strength of the bolt.[4.8]

The shear resistance of high-strength bolts is directly proportional to the available shear area. The available shear area in the threaded part of a bolt is equal to the root area and is less than the area of the bolt shank. For most commonly used bolts, the root area is about 70% of the nominal area. The influence of the shear plane location on the load versus deformation characteristics of A325 and A490 bolts is reported in Ref. 4.4. Figure 4.16 shows the influence of the shear plane

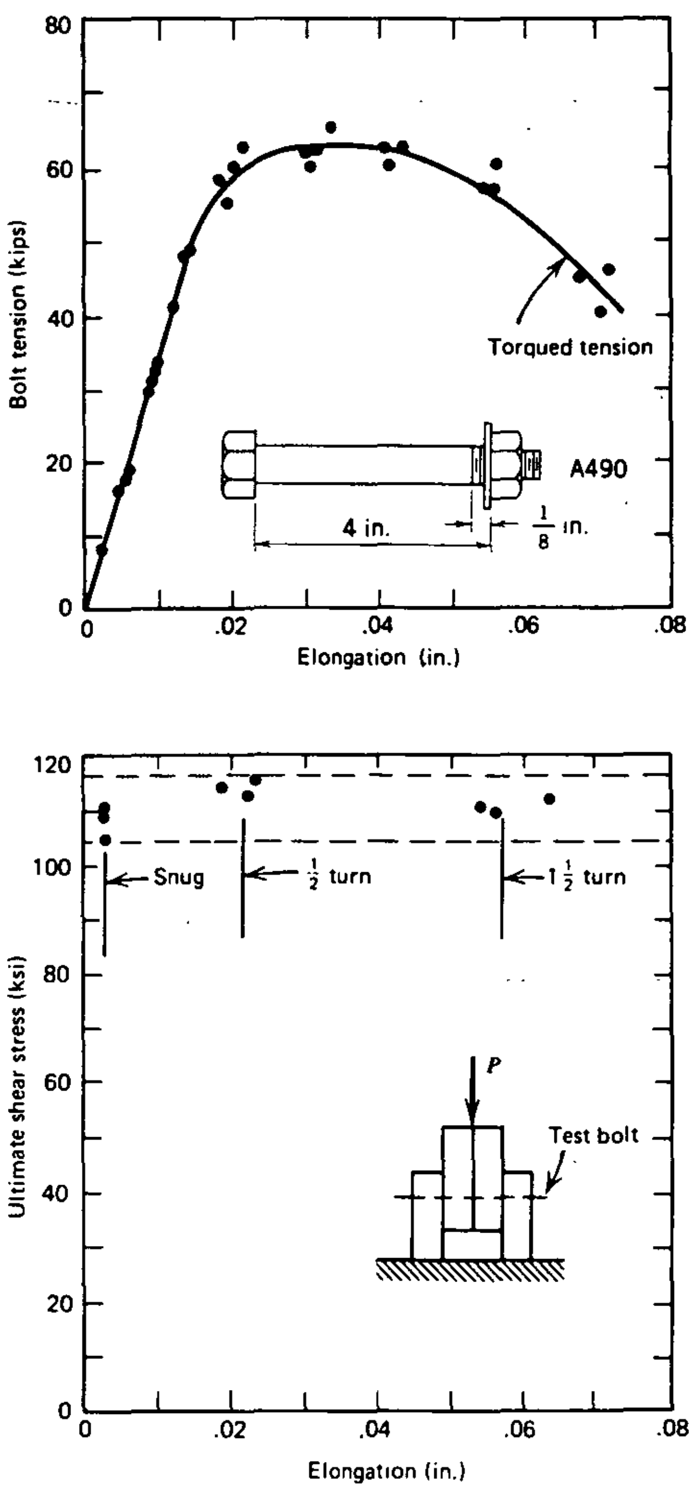


Fig. 4.15. Effect of bolt preload on shear strength of A490 bolts.

location on the load versus displacement behavior of A325 bolts. When both shear planes passed through the bolt shank, the shear load and deformation capacity were maximized. When both shear planes passed through the threaded portion, the lowest shear load and deformation capacity were obtained. All available tests indicate that the shear resistance of both A325 and A490 bolts is governed by the available

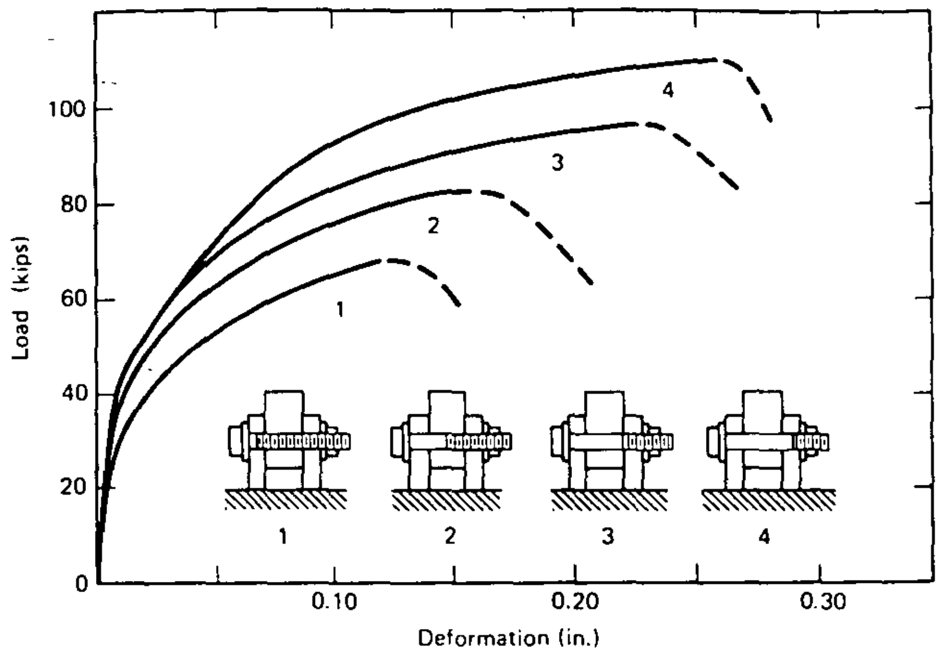


Fig. 4.16. Shear load versus deformation curves for different failure planes.

shear area. The unit shear strength was unaffected by the shear plane location, however.

4.2.3 Bolts Subjected to Combined Tension and Shear

To provide information regarding the strength and behavior characteristics of single high-strength bolts subjected to various combinations of tension and shear, tests were performed at the University of Illinois.[4.8] Two types of high-strength bolts, A325 and A354 grade BD, were used in the investigation. Since the mechanical properties of A354 grade BD and A490 bolts are nearly identical, the data are also directly applicable to A490 bolts.

Certain other factors that might influence the performance of high-strength bolts under combined loadings of tension and shear were also examined in the test program. These included (1) bolt grip length, (2) bolt diameter, (3) type of bolt, and (4) type of material gripped by the bolt. In addition, the influence of the location of the shear planes was examined.

The Illinois tests indicated that an increase in bolt grip tends to increase the ultimate load of a bolt subjected to combined tension and shear. This increase in resistance is mainly caused by the greater bending that can develop in a long bolt as compared with a short grip bolt. At high loads the short grip bolt presented a circular shear area, whereas the long grip bolt, because of bending, presented an elliptical cross-section with a larger shear area.

It was concluded, however, that neither the test block material nor the bolt diameter had a significant effect on the ultimate load capacity of the bolt.

Figure 4.17 summarizes test results of bolts subjected to combined tension and

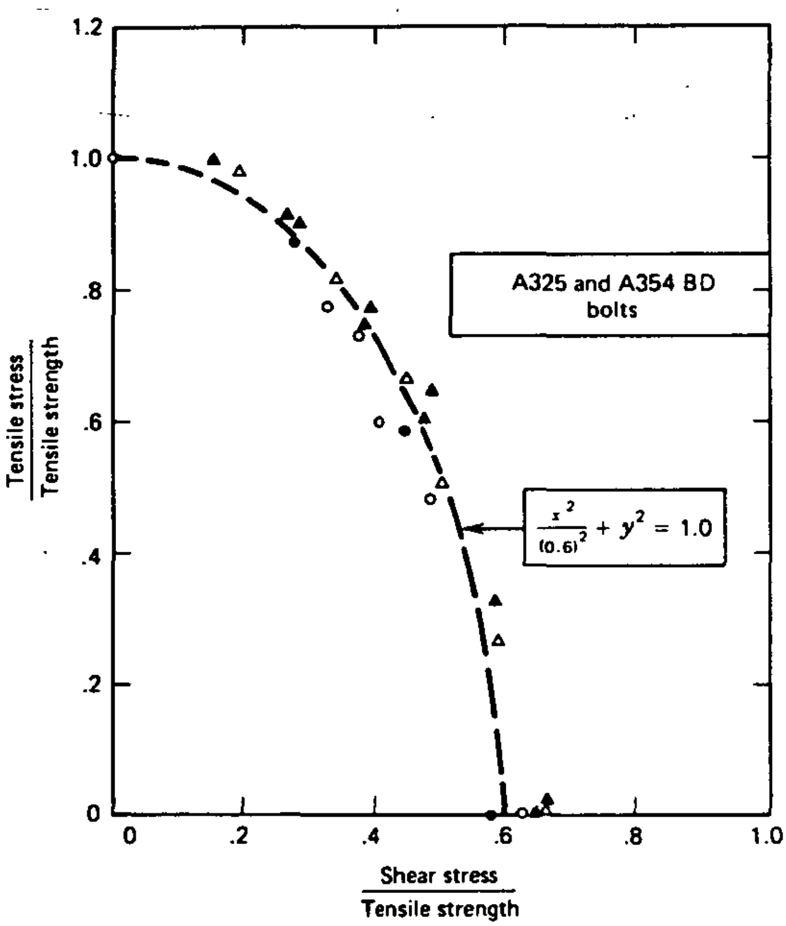


Fig. 4.17. Interaction curve for high-strength bolts under combined tension and shear.

	A325	A354BD
Threads in shear plane	△	○
Shank in shear plane	▲	●

shear.[4.8] The tensile strength (in kilopounds per square inch) was used to nondimensionalize the shear and tensile stresses due to the shear and tensile components of the load. The tensile stress was computed on the basis of the stress area, whereas the shear stress is dependent on the location of the shear plane. An elliptical interaction curve can be used to provide a good representation of the behavior of high-strength bolts under combined tension and shear; namely,

$$\frac{x^2}{(0.62)^2} + y^2 = 1.0 \tag{4.1}$$

where x is the ratio of the shear stress on the shear plane to the tensile strength and y is the ratio of the tensile stress to the tensile strength (both computed on the stress area). Figure 4.17 also indicates that neither the bolt grade nor the location of the shear plane influence the ultimate x/y ratio. This is compatible with the behavior of bolts in pure shear.

4.3 INSTALLATION OF HIGH-STRENGTH BOLTS

North American practice prior to 1985 had been to require that all high-strength bolts be installed so as to provide a high level of preload, regardless of whether it was needed (bolts in a slip-resistant connection or in a connection subject to tension) or not needed (bolts in a bearing-type connection). The advantages in such an arrangement were that a standard bolt installation procedure was provided for all types of connections and that a slightly stiffer structure probably resulted. The disadvantages were economic: the cost of installation of bolts that do not have to be preloaded was increased and the inspection of these installed bolts was unnecessarily complicated.

As was noted in Subsection 4.2.2, the ultimate shear strength of high-strength bolts is not dependent upon the amount of preload in the bolts. There have been a number of specifications that have recognized this in the past,[4.31–4.33] particularly in Europe but also including the International Standards Organization draft specifications for steel structures.[4.34] These specifications permit the use of non-preloaded high-strength bolts in bearing-type connections when load reversals are not present. In 1985, the RCSC introduced a significant relaxation of the rule that had been in previous editions of the specification, namely, that all high-strength fasteners be installed so as to provide a preload equal to 70% of the minimum specified tensile strength of the bolt. The requirement now is that only fasteners that are to be used in slip-critical connections or in connections subject to direct tension need to be preloaded to this level. Bolts to be used in bearing-type connections need only be tightened to the snug-tight condition.

To provide the desired level of preload for bolts used in slip-critical connections or in connections subjected to tension, the RCSC Specification[1.4] continues to require that in these cases the high-strength bolts be tightened such that the resulting bolt tension (preload) is at least 70% of the minimum specified tensile strength of the bolt. The resulting required minimum bolt tension, for various bolt diameters, is given for both A325 and A490 bolts in Table 4.1.

When the high-strength bolt was first introduced, installation was primarily by methods of torque control. Approximate torque values were suggested for use in obtaining the specified minimum bolt tension. For example, early versions of the council specification provided a value of torque that was supposed to produce the required bolt tension (0.0167 lb-ft per inch of bolt diameter per pound tension for standard water-soluble lubricated bolts and nuts). However, tests performed by Maney,[4.12] and later by Pauw and Howard,[4.13] showed the great variability of the torque-tension relationship. Bolts from the same lot yielded extreme values of bolt tension $\pm 30\%$ from the mean tension desired. The average variation was in general $\pm 10\%$. This variance is caused mainly be the variability of the thread conditions, surface conditions under the nut, lubrication, and other factors that cause energy

Table 4.1. Fastener Tension

Bolt Size (in.)	Minimum Fastener Tension[a] in Thousands of Pounds (kips) A325 Bolts	A490 Bolts
$\frac{1}{2}$	12	15
$\frac{5}{8}$	19	24
$\frac{3}{4}$	28	35
$\frac{7}{8}$	39	49
1	51	64
$1\frac{1}{8}$	56	80
$1\frac{1}{4}$	71	102
$1\frac{3}{8}$	85	121
$1\frac{1}{2}$	103	148

[a]Equal to 70% of specified minimum tensile strengths of bolts, rounded off to the nearest kip.

dissipation without inducing tension in the bolt. Experience in field use of high-strength bolts confirmed the erratic nature of the torque versus tension relationship.

RCSC specifications prior to 1980 permitted high-strength bolts to be tightened by using calibrated wrenches, by the turn-of-nut method, or by use of direct tension indicators.[1.4] The last two procedures depend on strain or displacement control, as contrasted to the torque control of the calibrated wrench method. The 1980 edition of the RCSC specification removed approval for the use of calibrated wrenches, however. (No doubt, many installations were still made using this method. In 1979 it was estimated that about 36% of the bolt installations in the United States were made using calibrated wrenches, but that it was scarcely used at all in Canada.[4.35]) In 1985 the RCSC specification again permitted use of the calibrated wrench method of installation, but with a clearer statement of the requirements of the method and its limitations.

In the calibrated wrench method the wrench is calibrated or adjusted to shut off when the desired torque is reached. In practice, several bolts of the lot to be installed are tightened in a calibrating device that directly reads the tension in the bolt. The wrench is adjusted to shut off at bolt tensions that are a minimum of 5% greater than the required preload. To minimize the variation in friction between the underside of the turned surface and the gripped material, hardened washers must be placed under the element turned in tightening. A minimum of three bolts of each diameter must be tightened at least once each working day in a calibrating device capable of indicating actual bolt tensions. This check must also be performed each time significant changes are made in the equipment or when a significant difference is noted in the surface conditions of the bolts, nuts, or washers.

The calibrated wrench method has a number of drawbacks. Because the method is essentially one of torque control, factors such as friction between the nut and the bolt threads and between the nut and washer are of major importance. The water-soluble lubricant supplied on the bolts can be degraded by rain or moisture or threads can become contaminated with dirt or grease. The result is an erratic torque-tension relationship, and this is not reflected in the calibration procedure. This method of installation also presents field problems when more than one bolt length is used in a given joint because the wrench must be calibrated for each length. (In Japan, the nuts and washers of so-called Quality A high-strength bolt sets are generally treated with a chemical coating in order to overcome some of these problems. The coating reduces the frictional resistance between nut and bolt threads and between nut and washer. However, this coating is sometimes affected by time or temperature.[4.35])

To overcome the variability of torque control, early efforts were made to develop a more reliable tightening procedure. The American Association of Railroads (AAR), faced with the problem of tightening bolts in remote areas without power tools, conducted a large number of tests to determine if the turn-of-nut could be used as a means of controlling bolt tension.[4.14,4.15] These tests led to the conclusion that one turn from a finger-tight position produced the desired bolt tension. In 1955 the RCRBSJ adopted one turn of the nut from hand-tight position as an alternative method to installation.

Experience with the one full turn method indicated that it was impractical to use finger or hand tightness as a reliable point for starting the one turn. Because of out-of-flatness, thread imperfections, and dirt accumulation, it was difficult and time consuming to determine the hand-tight position.

Bethlehem Steel Corporation developed a modified "turn-of-nut" method, using the AAR studies and additional tests of their own.[4.16,4.17] This method called for running the nut up to a snug position using an impact wrench rather than the finger-tight condition. From the snug position the nut was given an additional $\frac{1}{2}$ or $\frac{3}{4}$ turn, depending on the length of the bolt. The snug condition was defined as the point at which the wrench started to impact. This occurred when the turning of the nut was resisted by friction between the face of the nut and the surface of the steel. Snug-tightening the bolts induces small clamping forces in the bolts. In general, at the snug-tight condition the bolt clamping forces can vary considerably because elongations are still within the elastic range. This is illustrated in Fig. 4.18 where the range of bolt clamping force and bolt elongation at the snug tight condition is shown for $\frac{7}{8}$ in. dia. A325 bolts installed in an A440 steel test joint. The average clamping force at the snug-tight condition was equal to about 26 kip. The bolts in this test joint were snug tightened by means of an impact wrench. This modified turn-of-nut method was eventually incorporated into the 1960 specification of the council.

For bolts equal to or greater than about $\frac{3}{4}$ in. dia., snug position provided by an impact wrench is approximately equal to the tightness attained by the full effort of a man using an ordinary spud wrench. For longer or larger diameter bolts, the force produced by this snug load will be less than that for the "standard" case, and for shorter or smaller diameter bolts it will be more. These differences are accommodated in the specification by prescribing the same definition of snug tight

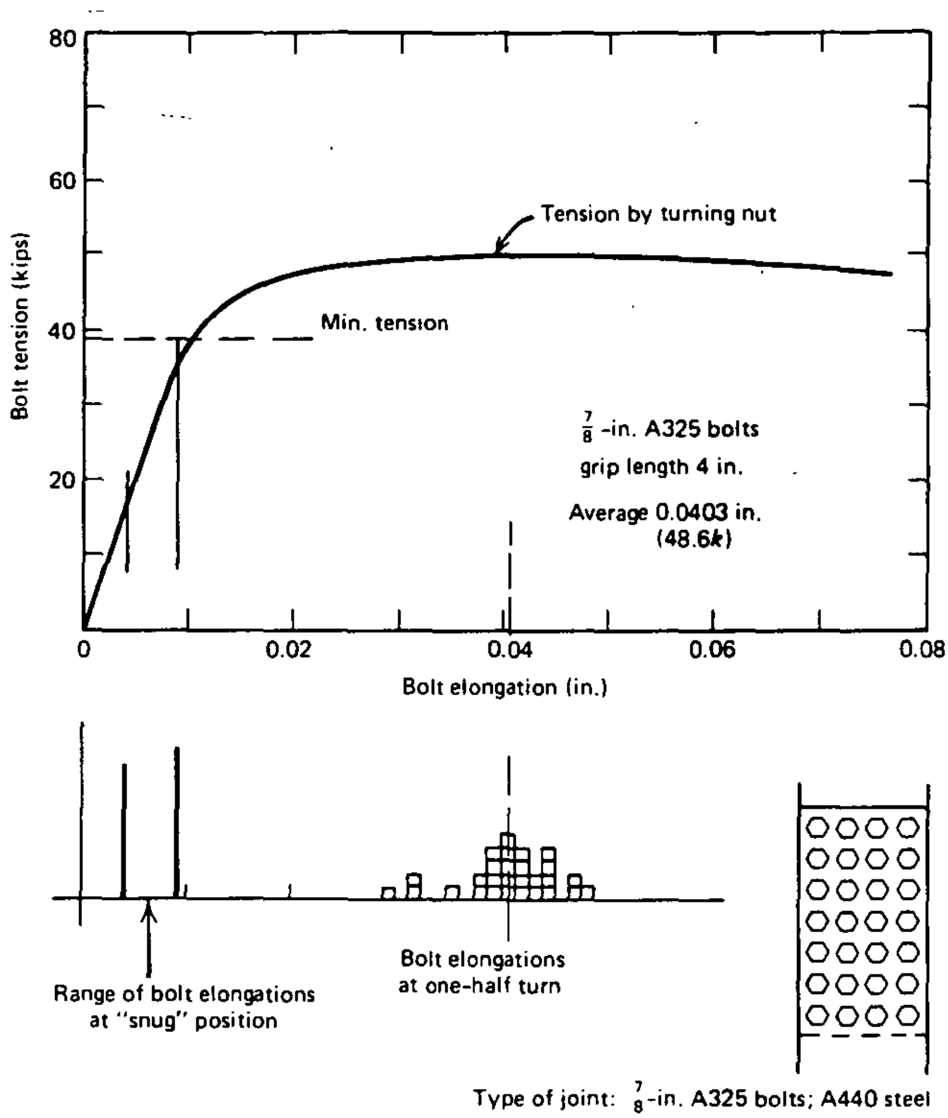


Fig. 4.18. Bolt elongation "snug" and after additional one-half turn of nut. Type of joint: $\frac{7}{8}$ in. dia. A325 bolts; A440 steel.

for all cases but varying the degree of rotation required beyond snug for different situations. As seen in Table 4.2, the current RCSC specification requires one-half turn from snug for bolts whose length from the underside of the head to the extreme end of the bolt is over four but less that eight bolt diameters. If this dimension is less than four bolts diameters, only one-third turn is required, and if it is greater than eight diameters, two-thirds turn is required. Test results are not available for bolts longer than 12 diameters, and so an upper limit is noted in the table. (The definition of bolt length as given previously and in Table 4.2 should not be abused. It is assumed that only a modest projection of bolt beyond the top of the nut will be present. If, for some reason, a large projection is present, the use of Table 4.2 should be based on an adjusted bolt length rather than on the actual bolt length. The length between the underside of the bolt head and the top of the nut would be

Table 4.2. Nut Rotation[a] from Snug-Tight Condition

	Disposition of Outer Faces of Bolted Parts		
Bolt Length (as measured from underside of head to extreme end of point)	Both Faces Normal to Bolt Axis	One Face Normal to Bolt Axis and Other Face Sloped Not More Than 1:20 (bevel washer not used)	Both Faces Sloped Not More Than 1:20 from Normal to Bolt Axis (bevel washers not used)
Up to and including 4 diameters	$\frac{1}{3}$ turn	$\frac{1}{2}$ turn	$\frac{2}{3}$ turn
Over 4 diameters but not exceeding 8 diameters	$\frac{1}{2}$ turn	$\frac{2}{3}$ turn	$\frac{5}{6}$ turn
Over 8 diameters but not exceeding 12 diameters[b]	$\frac{2}{3}$ turn	$\frac{5}{6}$ turn	1 turn

[a] Nut rotation is relative to bolt, regardless of the element (nut or bolt) being turned. For bolts installed by $\frac{1}{2}$ turn and less, the tolerance should be $\pm 30°$; for bolts installed by $\frac{2}{3}$ turn and more, the tolerance should be $\pm 45°$. All material within the grip of the bolt must be steel.

[b] No research work has been performed by the council to establish the turn-of-nut procedure when bolt lengths exceed 12 diameters. Therefore, the required rotation must be determined by actual tests in a suitable tension device simulating the actual conditions.

a suitable choice.) In all cases, care must be exercised to ensure that the nut does not encounter the thread run-out.

Controlling tension by the turn-of-nut method is primarily a strain control. If the elongation of the bolt remains within the elastic range, both the starting point (i.e., snug tight) and the amount and accuracy of the nut rotation beyond snug tight will be influential in determining the preload. However, in the inelastic region the load versus elongation curve is relatively flat, with the consequence that variations in the snug-tight condition result in only minor variations in the preload of the installed bolt. This inelastic behavior will be a characteristic of practically all installed bolts. It results from local yielding of the short length of thread between the underside of the nut and the gripped material. It has no undesirable effect on the subsequent structural performance of the bolt. Figure 4.18 illustrates these points.

Research in the 1960s indicated that one-half turn of the nut from the snug-tight condition was adequate for all lengths of A325 bolts that were then commonly used.[4.2,4.5–4.7,4.9] Based on this experience, the 1962 edition of the council specification required only one-half turn, regardless of bolt length.

In 1964 the council incorporated the A490 bolt into its specification. In order to make the specification applicable to both the A325 and the A490 bolts, the turn-of-nut method was modified again. Tests of A490 bolts had indicated that when the grip length was increased to about eight times the bolt diameter, a somewhat greater nut rotation (two-thirds turn) was needed to reach the required minimum bolt tension. Although the additional rotation was not needed for A325 bolts, the two-thirds turn provision has been applied to the A325 bolts as well in the interest of uniformity in field practice.

Calibration tests of A325 bolts with grips more than 4 diameters or 4 in. showed that the one-half turn of the nut rotation produced consistent bolt tensions in the inelastic range.[4.2] These tests also showed a sufficient margin of safety against fracture by excessive nut rotation. Bolts with grips of more than 4 in. or 4 diameters and short thread length under the nut can be given a one-half turn of the nut and have sufficient deformation capacity to sustain two additional half turns before failure. Bolts with long thread lengths in the grip can sustain three to five additional half turns, as illustrated in Fig. 4.19. Similar tests conducted on A490 bolts allow the comparison with A325 bolts shown in Fig. 4.20. A325 and A490 bolts gave substantially the same load versus nut rotation relationships up to the elastic limit.[4.1,4.3,4.9] At one-half turn from the snug position, the A490 bolts provided approximately 20% greater load than A325 bolts because of the increased strength of the A490 bolt. However, the higher strength of the A490 bolts results in a small decrease in nut rotation capacity as compared with the A325 bolt. These studies show that the factor of safety against twist-off for a bolt installed to one-half turn from snug is about three and one-half for A325 bolts and about two and one-half for A490 bolts. Moreover, it must be recognized that the only source of additional rotation after a bolt is installed would have to be vandalism. Because of the high torque required to produce additional rotation, even this source is unlikely.

Studies on short grip bolts (length less than or equal to four bolt diameters) have shown that their factor of safety against twist-off was less than two when one-half

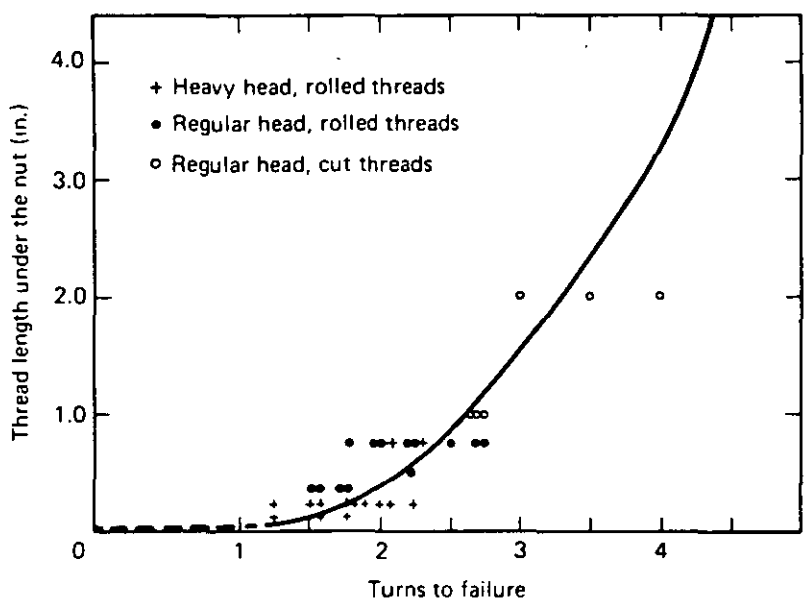


Fig. 4.19. Effect of thread length on rotation capacity of A325 bolts.

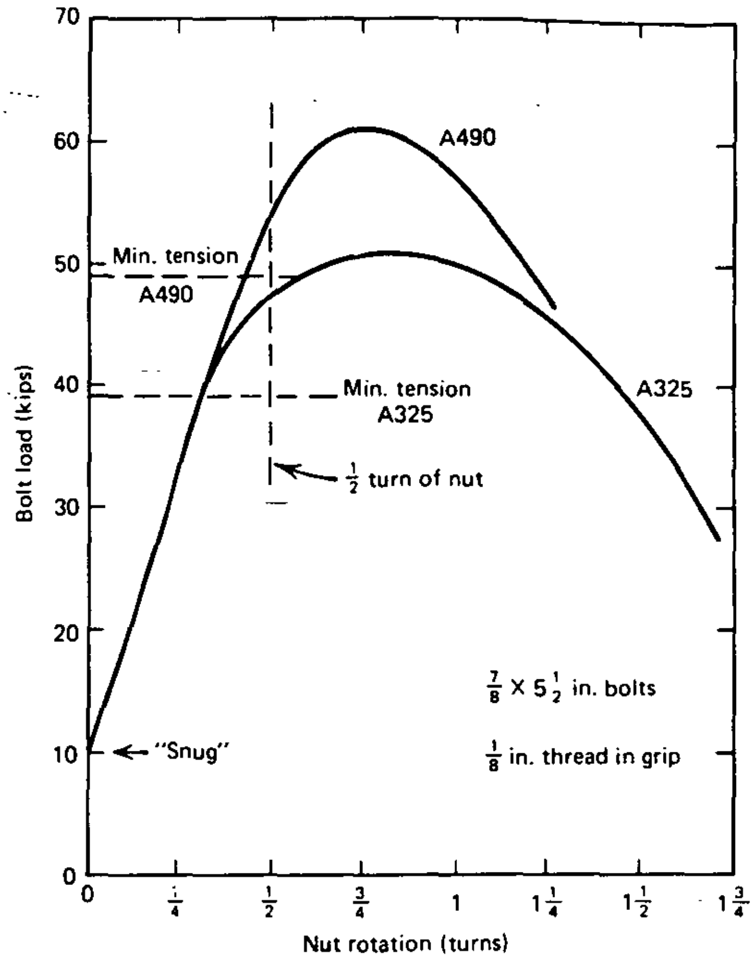


Fig. 4.20. Comparison of bolt load versus nut rotation relationships of A490 and A325 bolts.

turn was used. This resulted in the adoption in 1974 of one-third turn for bolts whose length was less than four diameters. More care needs to be taken in their installation in order to avoid twist-off.

Figure 4.21 shows load versus elongation curves for $\frac{7}{8}$ in. diameter A325 bolts $2\frac{1}{4}$ in. long.[4.36] Some tests were done on low hardness bolts and some on high hardness bolts, and there were either $1\frac{1}{2}$ or $2\frac{1}{2}$ threads unengaged below the nut. It is clear that both parameters had an influence on the ductility of these bolts. High hardness means high strength and reduced ductility. Because most of the bolt elongation is occurring in the threaded portion below the nut, an increase in this length also increased ductility. However, it can be noted that in all cases the specification requirement of one-third turn beyond snug produced a preload greater than the specified minimum value.

It should be apparent that short grip A490 bolts will be potentially less ductile than A325 bolts. Large diameter, short grip bolts will also be of concern because the ratio of tensile stress area to gross area decreases as bolt diameter increases. Figure 4.22 shows unpublished test results on large diameter, short grip A490

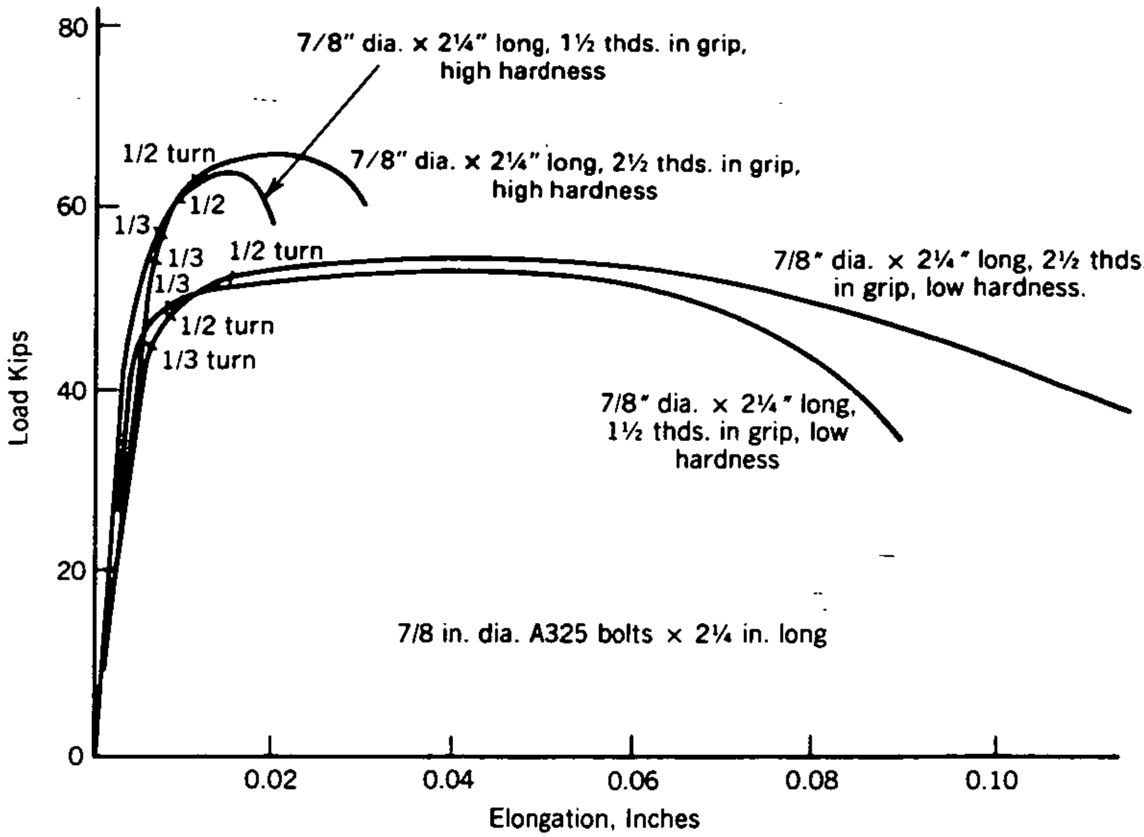


Fig. 4.21. Bolt load versus elongation for short grip A325 bolts.

bolts.[4.37] Because of the relatively large length of unengaged thread below the nut ($\frac{7}{8}$ in.), these bolts showed reasonable ductility for both low hardness and high hardness cases. However, for the same reason, one-third turn beyond snug was not sufficient to produce the specified preload in the bolts. Users of large diameter high-strength bolts, especially A490 bolts, should be aware that the RCSC specification requirement for installation of short grip bolts may not produce the required preload. If such bolts are to be used in a slip-resistant joint, calibration tests in a load-indicating device are advisable.

The 1985 specification of the RCSC permits alternate design bolts to be used and the installation of standard high-strength bolts by means of load-indicating washers. Alternate design bolts include the swedge bolts and tension-control bolts described in Section 4.1. The most common direct tension indicator used is a special washer with projections arranged circumferentially on one flat face. The gap created between the projections and the surface of the steel being clamped will be closed as the preload is introduced into the bolt. Measurements of the size of the gap can be related to the preload. Because of the time required to measure the gap, only spot measurements are usually taken, and care must be exercised to ensure that the protrusions bear against a hardened surface and do not turn as the nut is turned onto the bolt.

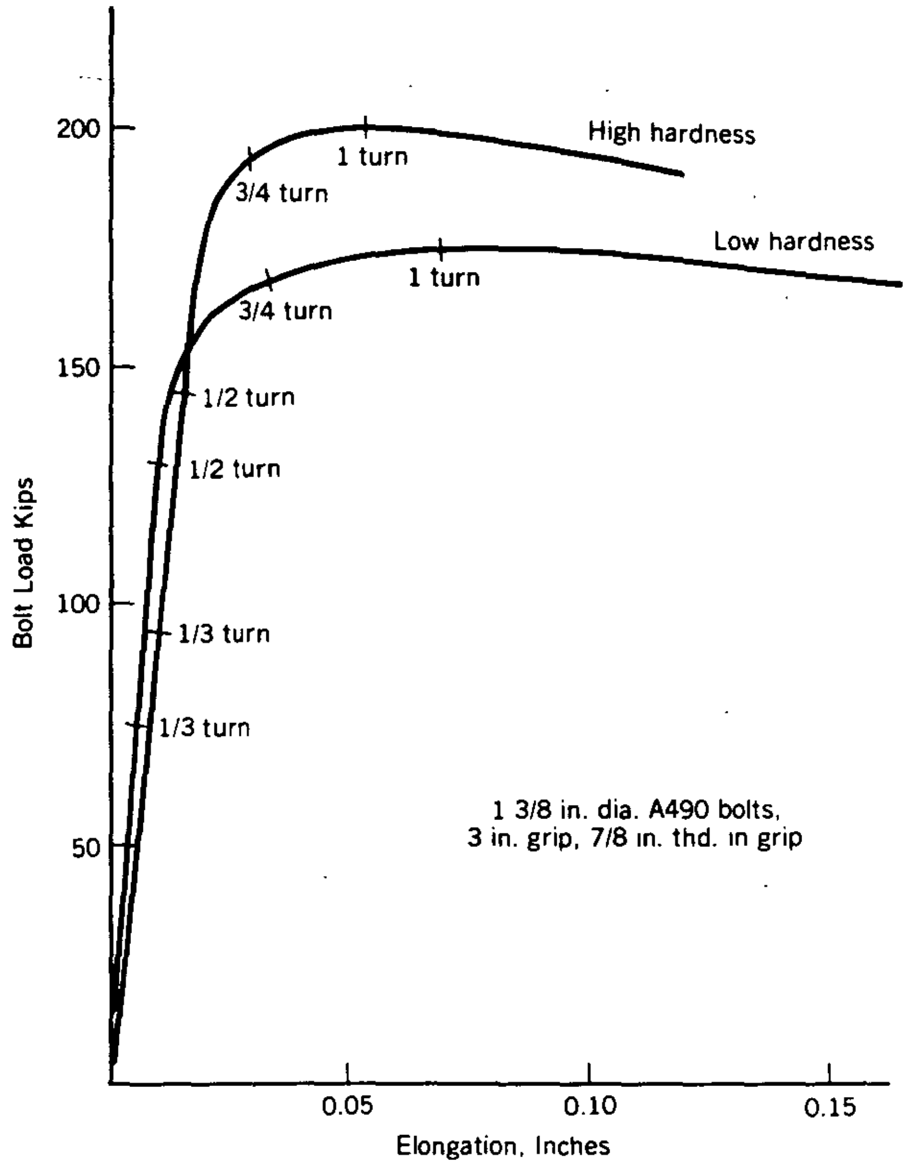


Fig. 4.22. Bolt load versus elongation for short grip A490 bolts.

The council specification contains requirements for verification of load if either alternate design bolts or direct tension indicators are used. Tension-control bolts (see Section 4.1) should be included in this category.

The calibrated wrench method of bolt installation is the most common method used in Japan.[4.35] As an alternative, some installations are made by a method that detects first yield of the bolt threads. An electrically operated wrench using a direct current motor is used. Because the torque in a direct current motor is directly related to the current drawn, it can be used to monitor the bolt tension. A monitor in the wrench is used to detect the first nonlinearity of the operation, and further tightening is prevented. Since the specified minimum tension is below the onset of first yield, this method is satisfactory.

Specifications require that the slope of surfaces of bolted parts in contact with the bolt head or nut shall not exceed 1:20 with respect to a plane normal to the bolt axis. Research carried out at the University of Illinois determined the influence of beveled surfaces (1:20 slope) when bevel washers were omitted.[4.9] A325 bolts are ductile enough to deform to this slope. Greater slopes are undesirable since they affect both strength and ductility.

From these tests it was concluded that the inclusion of bolted connections with a 1:20 slope in the grip and without beveled washers requires additional nut rotation to ensure that tightening will achieve the required minimum tension.[4.9] If one face is normal to the longitudinal axis of the bolt but the other has a bevel of up to 1:20, the usual one-half turn should be increased to two-thirds turn. If both faces are sloped at 1:20, five-sixths turn should be used. Table 4.2 shows the amount of nut rotation for shorter and longer grips when bevelled surfaces are present. Of course, bevel washers can be used to eliminate the slopes and thereby also eliminate the need for additional turns above the standard cases.

4.4 RELAXATION

Because of the high stress level in the threaded part of an installed bolt, some relaxation will occur that could affect the bolt performance. To evaluate the influence of this relaxation, studies were performed on assemblies of A325 and A354 grade BD bolts in A7 steel.[4.9] The bolts were tightened by turning the nut against the gripped material. The bolt tension versus time was registered throughout the study.

From these tests it was evident that immediately upon completion of the torquing there was a 2 to 11% drop in load. The average loss was 5% of the maximum registered bolt tension. This drop in bolt tension is believed to result from the elastic recovery that takes place when the wrench is removed. Creep and yielding in the bolt due to the high stress level at the root of the threads might result in a minor relaxation as well.

The grip length as well as the number of plies are believed to be among the factors that influence the amount of bolt relaxation. Although no experimental data are available, it seems reasonable to expect an increase in bolt force relaxation as the grip length is decreased. Similarly, increasing the number of plies for a constant grip length might lead to an increase in bolt relaxation. Relatively large losses in bolt preload have been reported for very short grip (i.e., $\frac{1}{2}$ to 1 in. grip) galvanized bolts.

Relaxation tests on A325 and A354 BD bolts showed an additional 4% loss in bolt tension after 21 days as compared with the bolt tension measured 1 min after torquing.[4.9] Ninety percent of this loss occurred during the first day. During the remaining 20 days the rate of change in bolt load decreased in an exponential manner.

Relaxation studies on assemblies with high-strength bolts were performed in

Japan and showed similar results.[4.10] By extrapolating the test data, it was concluded that the relaxation after 100,000 hr (11.4 years) could be estimated at about 6% of the bolt load immediately after tightening.

The relaxation characteristics of assemblies of galvanized plates and bolts were found to be about twice as great as plain bolts and connected material.[4.19] The amount of relaxation appeared to be related to the thickness of the galvanized coating. It was concluded that the increased bolt relaxation occurred because of the creep or flow of the zinc coating under sustained high clamping pressures. As with plain ungalvanized bolts, the galvanized bolts experienced most of the creep and relaxation immediately upon completion of the tightening process.

Based on tests performed at Lehigh University, it was concluded that, within certain limits, oversize or slotted holes do not significantly affect the losses in bolt tension with time following installation.[4.26] The loss in tension was about 8% of the initial preload. A more detailed discussion on this is given in Chapter 9.

4.5 REUSE OF HIGH-STRENGTH BOLTS

Since the turn-of-nut method is likely to induce a bolt tension that exceeds the elastic limit of the threaded portion, repeated tightening of high-strength bolts may be undesirable. Tests were performed to examine the behavior of high-strength bolts after torquing one-half turn, loosening, and then retorquing.[4.1,4.2] The record of one such test on a A325 bolt is summarized in Fig. 4.23. It is apparent that the cumulative plastic deformations caused a decrease in the A325 bolt deformation capacity after each succeeding one-half turn. However, A325 bolts can be reused

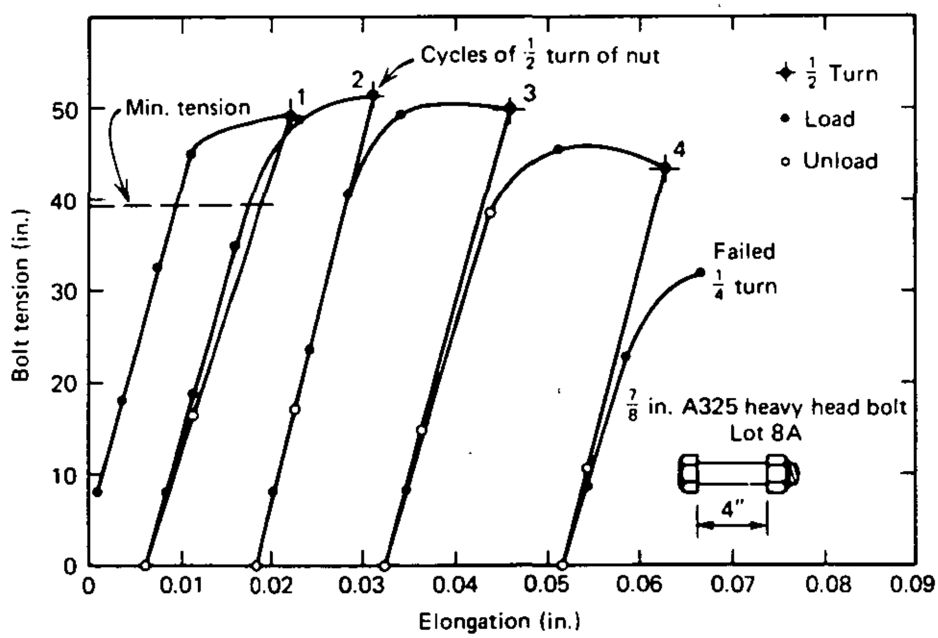


Fig. 4.23. Repeated installation of A325 bolts.

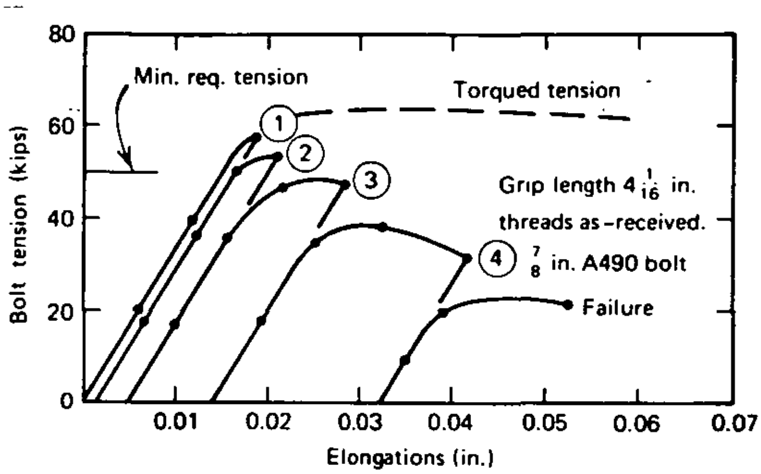


Fig. 4.24. Repeated installation of A490 bolts.

once or twice, providing that proper control on the number of reuses can be established.

As-received high-strength bolts have a light residual coating of oil from the manufacturing process. This coating is not harmful, and it should not be removed. Such as-received A325 bolts generally do have adequate nut rotation capacity to allow for a limited reuse provided either that the original lubricant is still on the bolt or oil, grease, wax, and so on is applied subsequently. Reuse of coated A325 bolts is not recommended, however. Tests have indicated that the nut rotation capacity of a bolt is generally reduced by providing a coating (see Section 4.6).[4.19,4.27] Therefore, unless experimental data indicate otherwise, reuse of coated A325 bolts should not be permitted.

Figure 4.24 shows typical results of one lot of A490 bolts repeatedly installed with threads as-received. Note that the minimum required tension was achieved only during the first and second cycle. Subsequent cycles showed a sharp decrease in induced bolt tension. Test results have indicated that bolts from the same lot when waxed had considerably improved characteristics.[4.1] However, whether the threads were waxed or as-received, a marked increase in installation time was noted for successive cycles. The behavior of A490 bolts under repeated torquing seems to be more critical than A325 bolts. Therefore, reuse of A490 bolts is not recommended.

4.6 GALVANIZED BOLTS AND NUTS

At the present time, a wide range of structures are being treated with a protective surface coating to prevent corrosion and reduce maintenance costs. Galvanizing is a widely used procedure and provides an excellent corrosion-resistant protection.

The behavior of galvanized bolts may differ from the behavior of normal, uncoated high-strength bolts.[4.18,4.19] This difference in behavior is caused primarily by the zinc layer on the bolt threads. Galling of this zinc layer may take place, and the nut may seize when the bolt is tightened. Occasionally this makes it difficult to reach the desired bolt tension without experiencing a premature torsional failure of the bolt.

The zinc coating on the surface of a bolt does not affect the bolt static strength properties. Calibration studies showed that neither the tensile strength, as determined from a direct tension test, nor the shear strength of the bolt were affected by the galvanizing process.[4.18,4.19] However, if bolt tension is induced by turning the nut against the gripped material, because of seizure unlubricated galvanized bolts experienced a greater reduction in the maximum bolt tension as compared with torqued ungalvanized bolts or properly lubricated galvanized bolts. This reduction was up to 25% more than that for plain black bolts, depending on the thread conditions and thickness of the zinc layer.

Besides this reduction in torqued tension strength, the added frictional resistance on the threads of the galvanized bolts caused a considerable decrease in ductility, as illustrated in Fig. 4.25. This effect of high frictional resistance can be reduced substantially by employing lubricants on the threads of galvanized bolts. Tests indicated no appreciable difference in the torqued tensile strength of plain bolts as-received and galvanized bolts lubricated with either beeswax, cetyl alcohol, or commercial wax.[4.11,4.27] Some reduction in ductility of the galvanized bolts was observed. Calibration tests performed on galvanized A490 bolts showed results similar to the results of A325 bolts.[4.18]

A high tendency for stripping-type failures was observed in torqued tension tests of galvanized high-strength bolts.[4.19] This can be attributed to several factors. As

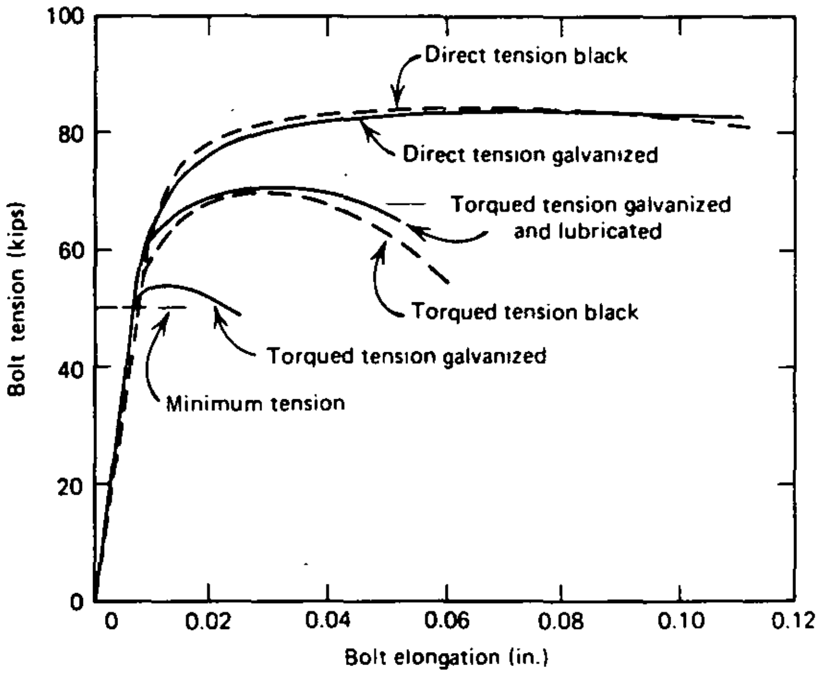


Fig. 4.25. Comparison of bolt load versus elongation relationships between 1-in. black and galvanized A325 bolts.

the bolt is torqued, the threaded section within the grip necks down and the nut spreads. This, along with the overtapping of the nut that is necessary when galvanizing, may cause a disengagement of some of the threads in the nut and increase the chance for stripping failures. To reduce the possibility of an undesirable stripping failure, harder nuts should be used for galvanized bolts (nuts of quality DH or 2H). In order to ensure that the galvanized bolt and nut and the lubricant provided on the nut or bolt threads will provide bolt preload in excess of the specified minimum tension with rotational reserve, special tests are required by the ASTM A325 specification. It must be demonstrated that the galvanized assembly can be subjected to 360° rotation from snug without failure. This test requirement ensures that the tolerances and lubricant are adequate.

Although galvanizing does provide an excellent protection against corrosion of the bolt, it may increase its susceptibility to stress corrosion and hydrogen stress cracking. This applies especially to galvanized A490 bolts. Therefore, it was concluded that galvanized A490 bolts should not be used in structures.[4.23,4.24]

4.7 USE OF WASHERS

Originally the high-strength structural bolt assembly included a bolt with a nut and two hardened washers. The washers were thought necessary to serve the following purposes:

1. To protect the outer surface of the connected material from damage or galling as the bolt or nut was torqued or turned
2. To assist in maintaining a high clamping force in the bolt assembly
3. To provide surfaces of consistent hardness so that the variation in the torque-tension relationship could be minimized

When the turn-of-nut method for tightening high-strength bolts was adopted, a procedure was introduced that provided a means of obtaining the required bolt tension without reliance upon torque-tension control. Hence, it was desirable to determine whether hardened washers were needed in the bolt assembly. Tests showed that a hardened washer was not needed to prevent minor the bolt relaxation resulting from the high stress concentration under the bolt head or nut of A325 bolts.[4.9] It was also concluded that any galling that may take place when nuts for A325 bolts are tightened directly against the connected parts is not detrimental to the static or fatigue strength of the joint.

As a result of these findings, the council specifications in general do not require the use of washers when A325 bolts are installed by the turn-of-nut method. If bolts are tightened by a calibrated wrench method, that is, by torque control, a washer should be used under the turned element, the nut or the bolt head. Washers are required under both the head and nut of A490 bolts when they are used to connect material with a yield point of less than 40 ksi. This prevents galling and brinelling of the connected parts. In high-strength steel they are only required to prevent galling of the turned element.

When bolts pass through a beam or channel flange that has a sloping interface, a bevel washer is often used to compensate for the lack of parallelism. Specifications require the use of beveled washers when an outer face has a slope greater than 1:20. A325 bolts are ductile enough to deform to this slope.[4.9] Greater slopes are undesirable as they affect both strength and ductility.

As noted in Section 4.3, when slopes of up to 1:20 are present in the gripped material, bolts require additional nut rotation to ensure that tightening will achieve the required minimum preload.

There are special requirements for washers when oversize or slotted holes are present; these are described in Chapter 9.

4.8 CORROSION AND EMBRITTLEMENT

Under certain conditions, corrosive environments may be detrimental to the serviceability of coated high-strength bolts subjected to sustained stresses. Hydrogen stress cracking as well as stress corrosion may cause delayed, "brittle" fractures of high-strength bolts. Although both processes have been studied extensively, no completely acceptable mechanism for explaining either phenomenon has been developed.

In many respects the two fracture mechanisms have a number of similarities. Both may cause delayed, brittle-type fractures of bolts. However, there appear to be significant differences. For example, stress corrosion at least in part involves electrochemical dissolution of metal along active sites under the influence of tensile stress. Hydrogen stress cracking occurs as the result of a combination of hydrogen in the metal lattice and tensile stress. The hydrogen produces a hard martensite structure that is susceptible to cracking. Atomic hydrogen absorption by the steel is necessary for this type of failure to occur. Since corrosion frequently is accompanied by the liberation of atomic hydrogen, hydrogen-stress cracking may occur in corrosive environments. However, in many situations a combination of both fracture patterns develops.

The crack surface of a failed bolt that experienced stress corrosion cracking is shown in Figure 4.26. The thumbnail-shaped markings at the bottom of the photograph corresponds to corrosion bands after crack extension and exposure to water. The microscopic appearance of the crack surface near the origin is shown in Figure 4.27 at 2000×. This shows intergranular cracking that is characteristic of stress corrosion cracking.

Laboratory tests have shown that both phenomena influence the life of high-strength bolts.[4.22–4.24] The behavior of A325 as well as A490 bolts under different environmental conditions was studied. From these test results, it became apparent that the higher the strength of the steel, the more sensitive the material becomes to both stress corrosion and hydrogen stress cracking. The study indicated a high

Fig. 4.26. Macroscopic appearance of the crack surface.

susceptibility of galvanized A490 bolts to hydrogen stress cracking. It was concluded that this was caused by a break in the zinc film, which promoted the entry of atomic hydrogen into the metal. If there were no breaks in the coating, failures were not likely to occur. The study also indicated the desirability of limiting the hardness of A490 bolts. Several uncoated bolts were observed to fail when high hardness and strength were present. Because of this observation, the maximum permissible tensile strength was decreased by the ASTM.

On the basis of these tests, it was concluded that properly processed black and galvanized A325 bolts, heat treated within presently specified hardness limits, will behave satisfactorily with regard to hydrogen stress and stress corrosion cracking in most corrosive environments.[4.23] Particular attention should be given to the preparation of the bolts for galvanizing. Improper pickling procedures could induce hydrogen embrittlement. It was further concluded that galvanized A490 bolts should not be used in structures. The tests did indicate that black A490 bolts can be used without problems from "brittle" failures in most environments.

A basic study of the effects of electroplated and hot-dip zinc coatings on the fracture of low-alloy steel AISI 4140 bars in hardness ranges of R_c 33 to 49 was

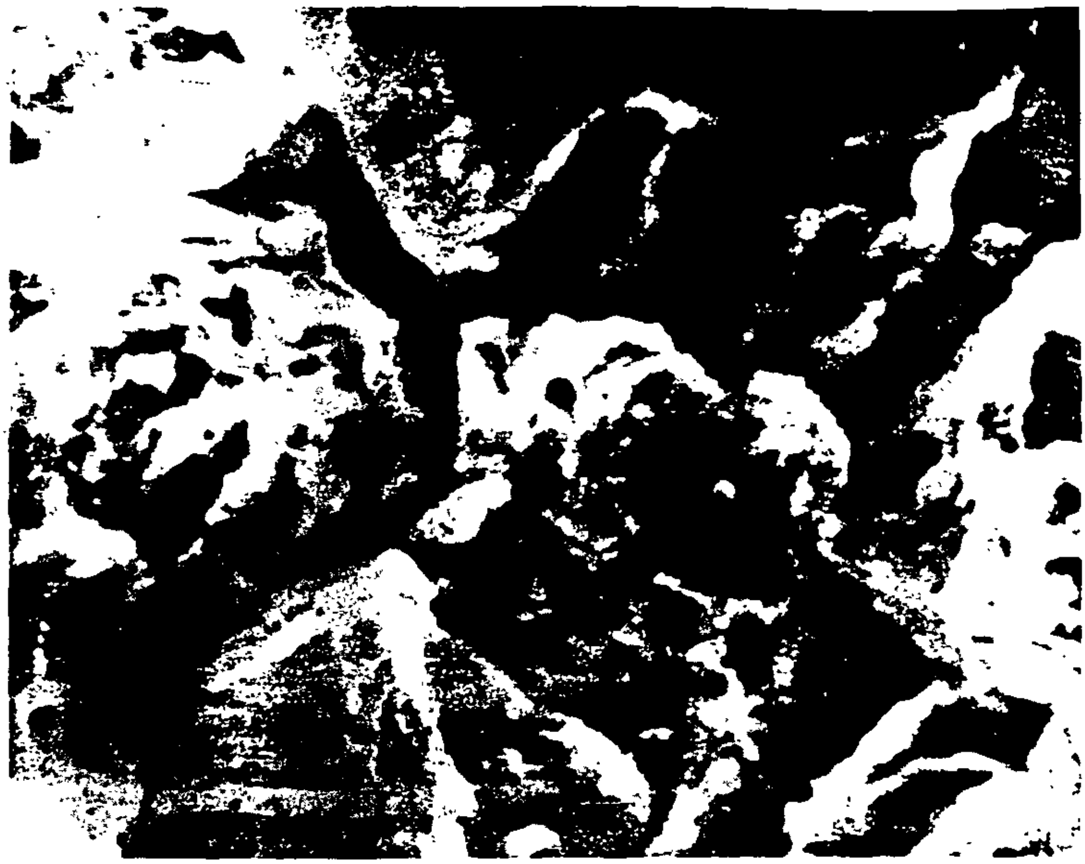


Fig. 4.27. Intergranular fracture surface at the crack origin. 2000×. (Courtesy of Bethlehem Steel Corp.)

conducted by Townsend.[4.38] Electroplated and hot-dip zinc coatings decreased the resistance to stress corrosion cracking directly in relation to the threshold stress intensity, K_{sc}. This effect was attributed to an increased equilibrium hydrogen activity at the crack-tip surface caused by the galvanic effect of the sacrificial coatings. Figure 4.28 shows the measured critical stress intensity as a function of R_c hardness. Although all hardness levels showed stress corrosion susceptibility, the higher hardness levels showed an increased susceptibility.

It was suggested that the condition in bolt threads was directly comparable to the stress intensity for a notched bar, that is,

$$K = \sigma\sqrt{\pi D}\, f\left(\frac{d}{D}\right)$$

For bolts, $f(d/D)$ varies from 0.25 to 0.23 as the shank diameter D varies from $\frac{1}{2}$ to 2 in. and the minor thread diameter d varies from 0.40 to 1.71 in. This approximate fracture mechanics analysis predicts overly conservative results. No failures were observed in actual bolts studied by Boyd and Hyler[4.23] at the lower hardness levels predicted by this study.

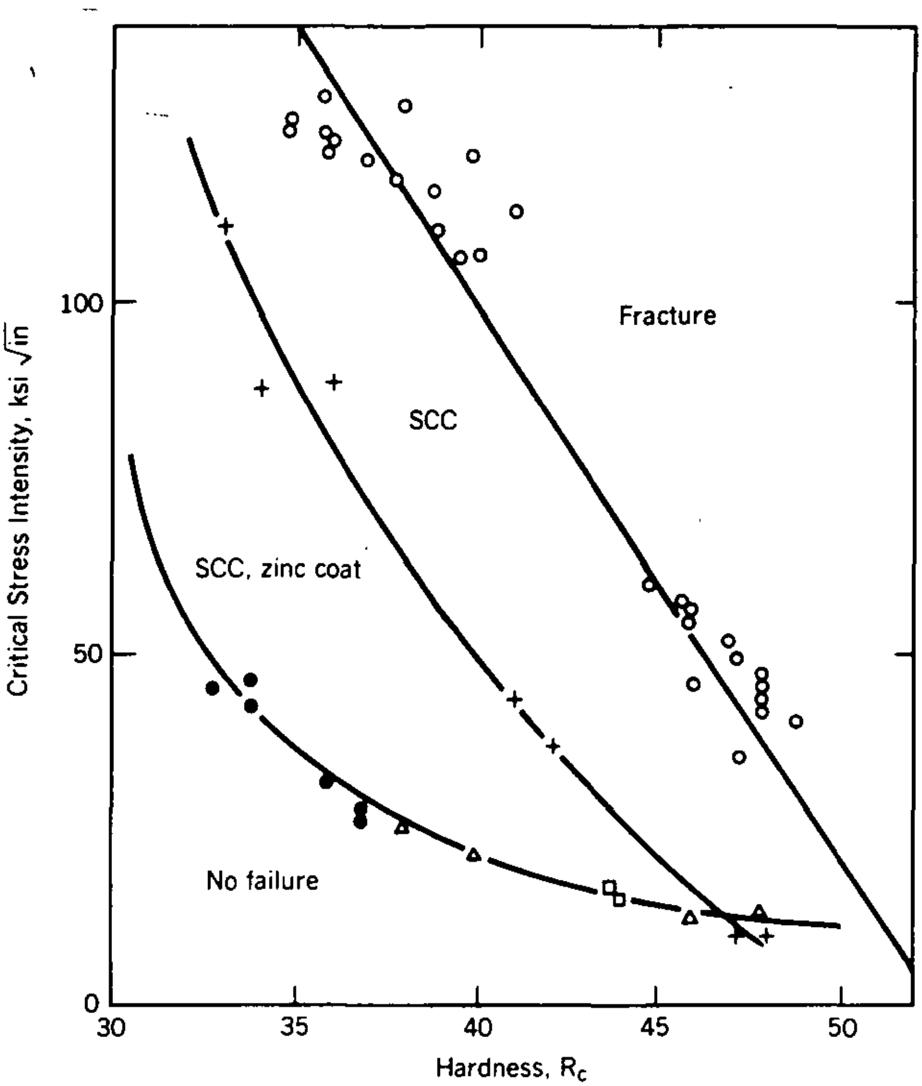


Fig. 4.28. Critical stress intensities in presence of corrosive environment: ○ K_x, all surfaces; + K_{sc}, uncoated; ● K_{sc}, hot-dip Al-Zn; ▲ K_{sc}, electroplated zinc; ■ K_{sc}, hot-dip zinc. (From Ref. 4.38)

4.9 EFFECT OF NUT STRENGTH

The behavior of bolt assemblies may vary when tightened to failure. In some cases, failure is in tension through the bolt threads; in other instances, the threads of the nut and/or bolt strip. A tensile failure of the bolt is easily detected; however, a stripping failure develops with imperceptible reduction in torque and is difficult to identify, since some tension remains in the bolt. Therefore, when failure by overtightening occurs or is imminent, a tensile failure of the bolt is preferable. To provide for this, nuts are specified to have a somewhat higher proof load than the bolts with which they are to be used.

As a nut is tightened against the resistance of the gripped material, the bolt lengthens within the grip. If the gripped material and the threads were completely rigid, one turn of the nut would cause the bolt to elongate one pitch. This does not happen, because some thread deformations occur in the bolt and nut. This diminishes the theoretical bolt elongation in the threaded portion.

Since the deformations of the threads are directly affected by the hardness of the nut or the bolt and the number of threads within the depth of the nut, calibration tests were performed on A325 high-strength bolts with minimum and maximum strength levels and assembled with hex nuts and with the thicker heavy hex nuts having various hardness values.[4.20] These tests showed that, with increasing nut hardness, the stripping strength of the connection also increases until the mode of failure changes to a tensile failure in the bolt thread. The bolt tension at one-half turn from a snug-tight condition also increased with an increase in nut hardness, and higher bolt loads were observed in assemblies using high hardness bolts. For all bolt and nut combinations used in this study, the average bolt tension at snug-tight plus one-half turn was considerably above the required minimum tension.

On the basis of these tests,[4.21] as well as other information, the council specification in 1972 started to require the use of heavy hexagonal nuts for A325 and A490 bolts. Because heavy hex nuts have the same dimension across the flats as the bolt head, their use has the additional advantage that a single wrench is applicable to both nut and head.

4.10 BASIS FOR DESIGN RECOMMENDATIONS

The behavior of individual fasteners subjected to different types of loading forms a basis for developing design recommendations. This section summarizes the individual fastener strengths that are used in subsequent chapters to develop design recommendations.

4.10.1 Bolts Subjected to Tension

The tensile capacity of a fastener is equal to the product of the stress area A_s and its tensile strength σ_u. However, it is convenient for design purposes to specify permissible forces and stresses on the basis of the nominal area of the bolt A_b rather than on the stress area A_s. Such a transformation is readily performed because the ratio of the stress area to the nominal bolt area only varies from 0.75 for $\frac{3}{4}$-in. diameter bolts to 0.79 for $1\frac{1}{8}$-in. diameter bolts. The maximum tensile load B_u of a fastener is given as

$$B_u = A_s \sigma_u \tag{4.2}$$

Expressed in terms of the nominal bolt area and using the lower bound,

$$B_u = 0.75 A_b \sigma_u \tag{4.3}$$

For most bolt diameters, Eq. 4.3 yields a slightly conservative estimate of the tensile capacity of a bolt.

4.10.2 Bolts Subjected to Shear

The tension-type shear test was observed to provide a lower bound shear strength. The shear strength (in kilopounds per square inch) of a fastener was found to be independent of the bolt grade and equal to 62% of the tensile strength of the bolt material; hence

$$\tau_u = 0.62\sigma_u \tag{4.4}$$

The shear resistance of a bolt is directly proportional to the available shear and the number of shear planes. If a total of m shear planes pass through the bolt shank, the maximum shear resistance S_u of the bolt is equal to

$$S_u = mA_b(0.62)\sigma_u \tag{4.5}$$

When shear planes pass through the threaded portion of the bolt, the shear area is equal to the root area of the bolt, which is about 70 to 75% of the nominal bolt area. A lower bound to the maximum shear capacity of the bolt can be expressed as

$$S_u = (0.70)\ mA_b(0.62)\sigma_u \tag{4.6}$$

or

$$S_u = (0.43)\ mA_b\sigma_u \tag{4.7}$$

If one shear plane passes through the shank of the bolt and one passes through the threads, the total shear area is equal to the sum of the individual components.

4.10.3 Bolts Subjected to Combined Tension and Shear

An elliptical interaction curve was found to represent adequately the behavior of high-strength bolts under combined tension and shear. The equation (4.1) was given in Section 4.2 as

$$\frac{x^2}{(0.62)^2} + y^2 = 1.0$$

where x is the ratio of the shear stress on the shear plane to the tensile strength and y is the ratio of the tensile stress to the tensile strength (both computed on the stress area). Equation 4.1 relates the shear stress component to the critical tensile stress component. The product of ultimate stress and the appropriate area yields the critical shear and tensile load components.

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MÓDULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA

EQUIPOS Y PROCEDIMIENTOS DE DIBUJO Y TRAZADO DE ESTRUCTURAS METÁLICAS

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PALACIO DE MINERÍA
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CHAPTER 1

Drafting Equipment and Drafting Procedures

EQUIPMENT AND SUPPLIES

The vast majority of drawings made today for the fabrication and erection of structural steel are done in pencil on tracing paper, pencil tracing cloth or drafting film. Equipment requirements and techniques described in this book will be largely oriented toward pencil work. Purchasers who require ink tracings will usually accept satisfactory cloth or film reproductions of the original pencil work.

The equipment and supplies used by the structural detailer are shown in List No. 1. Some of the items will be his personal property; others, as noted, are usually furnished by his employer. List No. 2 includes equipment which, while not essential in the preparation of a typical shop drawing, will add to the detailer's convenience and efficiency. Here, too, some of the items are standard drawing room equipment furnished by the employer. Items noted with an asterisk (*) are usually furnished by the employer.

List No. 1

*(1) A drawing table, at least 36 in. x 72 in.
*(2) A drafting machine
(3) A 45° transparent triangle, about 8 in. long on each of its two equal legs
(4) A 30–60° transparent triangle, about 10 in. long on the longer perpendicular leg
(5) An architect's scale, three-sided, 12 in. long, preferably with the divisions on a white plastic background
(6) A pencil bow compass with center wheel
(7) A circle template which can be used to draw circles up to about 1½-in. diameter
(8) An irregular (French) curve, transparent, preferably 8 to 10 in. long
*(9) Drawing pencils: F, H, 2H and 4H grades, either wood or refill type. Similar grades of plastic pencils are recommended, though not essential, for use on drafting film
*(10) Pencil sharpener
*(11) A sandpaper pad or pencil pointer
*(12) Erasers, soft, for pencil work
(13) Erasing shield, preferably steel
*(14) Tracing paper, pencil tracing cloth or drafting film
*(15) Drafting (masking) tape or draftsman's stapler
*(16) Scratch pads
*(17) A dusting brush
(18) A copy of *Manual of Steel Construction*, current edition, available from book stores or the American Institute of Steel Construction, 400 North Michigan Avenue, Chicago, IL 60611.
(19) Hand calculator, preferably an instrument with engineering functions
(20) Various templates for welding and other frequently used symbols

List No. 2

(1) Special drawing instruments, such as a larger pencil compass and a pair of dividers
(2) Additional triangles, larger or smaller than those recommended in List No. 1
(3) An adjustable triangle that combines the functions of a protractor and triangle
(4) A "Modern Joint Detailer" for the graphic solution of beveled joints, procurable from Fabricators Service, 1100 Wood St., Pittsburgh, PA 15221
(5) A protractor, fixed or adjustable, at least 6-in. diameter
(6) A civil engineer's scale, three sided, 12 in. long
*(7) A beam compass for large circles and circle arcs

It is suggested that the draftsman trainee who is purchasing equipment be guided by the following recommendations:

(1) Observe the equipment owned and used by the more experienced men in the drafting room and ask their advice before making purchases.
(2) Purchase items suggested in List No. 2 only as the need arises.
(3) Purchase the best quality equipment you can afford; professional grade, if possible.
(4) Delay purchase of a full set of drawing instruments, including ruling pens, compasses, dividers, etc., until actual needs are determined. Large drawing sets may have a number of in-

struments which will never be used, particularly if only occasional ink work is to be done. Consult manufacturers' catalogs and compare the cost of available sets with that of instruments bought separately, as needed. It is recommended that "bow" instruments, such as pencil compasses, dividers, etc., be the center-wheel type. These can be adjusted quickly and are generally more durable than other types.

Drafting Paper, Cloth and Film

Although the draftsman may have little to say about the choice of tracing media he is expected to use, he should be familiar with the characteristics of the various types he may encounter. Most pencil drawings are made on tracing paper, of which there are two distinct kinds: treated vellum and plain white rag paper. Of the two, vellum is the more transparent, thickness for thickness, and may be printed at greater speeds than plain rag paper. There appears to be little difference between these papers insofar as acceptance of pencil marks and erasability is concerned. Both papers, in the better qualities, are composed of 100% rag stock which toughens the paper and resists discoloration and brittleness due to age.

Past objections to the use of vellum on the ground that pencil lines cut through the treated surface, were difficult to erase and consequently produced "ghost marks" on prints, have been largely surmounted by modern manufacturing methods, which treat the entire thickness of the paper, and not the surface alone.

Pencil tracing cloth and drafting film have common characteristics in that the matte surface takes pencil extremely well and produces excellent prints and tracing reproductions. Both materials are far superior to all others in resistance to wear occasioned by repeated handling and printing. However, since pencil cloth is vulnerable to moisture, the draftman's hands must be dry, and other moisture kept away to prevent spotting and destroying the surface. Drafting film, with a polyester film base and applied matte surface, is impervious to water and practically indestructible. Cloth is furnished with a dull and a glossy side; the former is the working surface.

Prints and Reproductions

For use in the fabricating shop and elsewhere, prints of original tracings are made by one of several processes. All processes require a coated paper, exposure to light through a tracing, and development by chemicals in water or by ammonia gas. Blueprints show white lines on a blue background; other prints may have brown, black or blue lines on a white or light background, depending on which of several available processes is employed. Blueprints are subject to distortion due to wetting and drying during processing. Dry process prints are dimensionally more stable, because they are subjected only to atmospheric humidity.

Reproducible prints are frequently made from the original tracing. These may be on translucent (printable) paper, cloth or drafting film. One use of reproducibles is to save redrawing a member when two members are similar, but the differences are too great to note on a single detail. In this case, a reproducible is made of the detail drawing of the first member and then altered as required for the second member. Obviously, this is much quicker than drawing the member over again. In recent years most shops prefer separate drawings to a complex combination of details.

Reproducibles may be altered with liquid chemicals or an ordinary eraser, depending on the type of reproduction. Certain types of cloth and film reproductions may be very easily altered with a moistened eraser or damp cloth. Reproducibles have a matte surface which permits penciled corrections or additions.

Drawing Boards, T-Squares and Triangles

Most drafting rooms are furnished either with fully equipped drawing tables or with drawing boards and T-squares. Drawing tables have wooden or metal tops, which may have a tilting adjustment. Sometimes, drawing tables are equipped with parallel ruling attachments which eliminate the need for a T-square, or with drafting machines with scales which supplant both the T-square and triangles. Drafting machines have scales positioned at 90° to each other and attached to a movable protractor head which can be rotated and locked in position to permit measuring and drawing lines at any angle.

The working surface of the drawing board or table top must be provided with a backing sheet. This sheet should have a cushioning effect, yet be resistant to the pressure from the pencil which tends to "engrave" and roughen it. Several highly resilient materials are available for table tops. The metal tops are often furnished with integral resilient surfaces. For ordinary wooden boards, one or more layers of heavy paper suffice. The tracing paper, cloth or film is fastened to this backing with tape or staples.

The 30-60 and 45° triangles recommended in List No. 1 are used in conjunction with the T-square to produce vertical lines and inclined lines limited to 15°-intervals. When used with a swivel head T-square, almost any desired line inclination can be drawn, although not so readily as with a drafting machine. Adjustable triangles incorporate in one instrument a right angle and a movable edge, which may be adjusted to any desired angle.

Drafting Scales

Due to the size of most structural members, it is invariably necessary to picture them on shop drawings less than full size, using an appropriate scale for the desired reduction. One such scale, commonly used, is 1″ = 1′-0″. At this reduction, the view of an object which is actually one foot long will have a length of one inch on the drawing. All other dimensions of the object will be shown reduced in the same proportion, except as discussed later in this chapter. Other scales of reduction are also used in structural drafting. When and why each one is used will become evident in later chapters.

The architect's scale, with edges divided and identified in terms of full-size values, permits the draftsman to lay out full-size measurements at the desired reduction. The reducing scales found on a typical three-sided (triangular) architect's scale are 10 in number and range from 3/32″ = 1′-0″ to 3″ = 1′-0″. Only the foot intercepts are shown and numbered in the body of the scale, and the units at the ends are subdivided into inches and fractions of inches, depending upon the scale.

In using the architect's scale, the draftsman must recognize the value of the least division of a foot on each scale. For example, the smallest interval on the 1/8″ scale represents 2 in.; on the 3″ scale, the smallest interval represents 1/8-in. Also on the triangular scale is a full scale, having each inch divided into halves, quarters, eighths and sixteenths.

As the draftsman gains experience, he may find it convenient to own other scales than the one described in List No. 1. Available to him are triangular scales 6 in. long, flat scales containing two or four of the most frequently used scales, and civil engineer's scales on which the individual scales may be divided decimally into 10, 20, 30, 40, 50 or 60 units per inch.

Drafting Pencils

The selection of pencils with the degree of hardness (grade) needed to produce satisfactory tracings and prints is determined by the type of paper, cloth or film used, the individual draftsman's "touch," and, to some extent, the humidity. For lettering on tracing paper, draftsmen commonly use F or H pencils; for border and object lines, HB or F; and for dimension lines, center lines, etc., 2H or 3H. Pencil tracing cloth and drafting film will accept pencils about one grade harder, with comparable results. Pencils of other grades, both harder and softer, are available.

Manufacturers recommend plastic pencils for use on drafting film. The advantages cited include a dead black nonreflecting line and strong resistance to smearing. The plastic leads, available in wooden pencils or as leads for refill pencils, may be obtained in six numbered grades, from soft to hard. Their adaptability to various weights of lines and the technique of their use are matters for experimentation. Plastic pencils have been used with success on pencil tracing cloth, but they are too hard for use on ordinary tracing paper. One disadvantage is that, unless erasures are made lightly, the "tooth" of the matte surface of pencil cloth or film is diminished and it is difficult to reproduce the original weight or blackness of line over the erased area. This handicap may be minimized by use of special erasers recommended by pencil manufacturers for use on drafting film.

It should be emphasized that linework and lettering must be uniformly black and distinct. Fuzzy and uncertain lines, barely readable on the tracing, may disappear on the print or be misinterpreted by the shopman using the drawing. Because properly sharpened pencils are needed for linework of professional quality, many draftsmen favor the refill pencil, which is readily pointed and kept sharp by a desk-type pencil pointer or sandpaper pad. Wooden case pencils of the same grade are frequently kept on hand in lots of a half-dozen or so, used in rotation, and all sharpened at once. A draftsman's pencil sharpener, which removes the wood only, used in conjunction with a pencil pointer, has proved to be a time saver in some drafting rooms.

LINEWORK AND LETTERING

The good appearance of any pencil drawing is largely a matter of uniformity in making lines and letters. The utility of a pencil drawing depends on the strength and contrast of the various line symbols and the legibility of freehand work. It follows that the most desirable drawing is one that combines both uniformity and utility. No matter how uniform in appearance, fine, delicate linework and tiny lettering must be avoided, since oil and grease on prints in the shop or field can render such a drawing useless or misleading.

Simple details can be sketched freehand. As a matter of fact, the structural draftsman often finds it convenient to make such sketches on a scratch pad. These sketches enable him to collect and relate data from various sources and to plan the arrangement of the particular drawing he is to make. However, in the interest of clarity and uniformity, and because all draftsmen are not equally proficient in freehand sketching, the linework on a structural drawing is invariably drawn with a drafting machine, T-square, triangles or the other instruments commonly used in technical drafting.

To aid the trainee in selecting the proper line weights, Fig. 1-1 shows recommended line conventions generally used in structural drafting. Figure 1-2 illustrates the appearance of such lines as they relate to one another on a drawing.

Border lines, when not preprinted on the sheet, should be made heavy and black, to contrast strongly with all other lines. To draw these, it is recommended that the pencil point not be too sharp and that lines be retraced until the necessary density is achieved. In the interest of speed, all other lines should be made with a single stroke, using pencils of the necessary hardness and sharpness to obtain the density required. The technique of pencil linework can best be learned through practice. The cone point, resulting from the pencil sharpener or pencil pointer, requires a slow rotation of the pencil to maintain uniform thickness from end to end of the line.

The dashes and spaces of hidden object lines should be proportioned by eye, uniform, and slightly less pronounced in weight than visible object lines. The dashes should be firm, not feathered, and about twice the length of the intervening spaces (see Fig. 1-1). A dashed line should start with a solid line, not a space. Corners should be formed with intersecting dashes. A good average proportion is 1/8-in. dashes and 1/16-in. spaces.

The starting and stopping of all lines should be positive; overriding of object and dimension lines beyond their proper terminations must be avoided.

Most circles and circle arcs are executed with a compass. The pencil bow compass, suggested in List No. 1, should be satisfactory for the greater part of the work of the structural draftsman. Small circles are conveniently made with a circle template; extremely large ones require a beam compass or railroad curves. Because the pressure that can be applied is limited, leads for use in pencil compasses should be of grade F or H, which are softer than used for hand-drawn linework. Leads for compass work should be sanded to a wedge point to assure line uniformity.

In pencil work, certain small circles and other symbols representing holes and bolts on shop and erection drawings may be conveniently drawn freehand. Freehand execution speeds up the work considerably, and, when done carefully with a well-sharpened pencil, is entirely satisfactory.

ORTHOGRAPHIC PROJECTION

Although pictorial drawings (see Fig. 1-3) have some application in developing and communicating ideas, they do not lend themselves readily to making structural shop drawings. A multiview system known as *orthographic projection* is used for shop details throughout the industry. The basis of this method is to show the characteristics of an object by using as many dimensioned views as necessary to describe it fully. The views show the shape of the object, as observed from several directions, and are related to each other by location on the drawing and by their dimensioning.

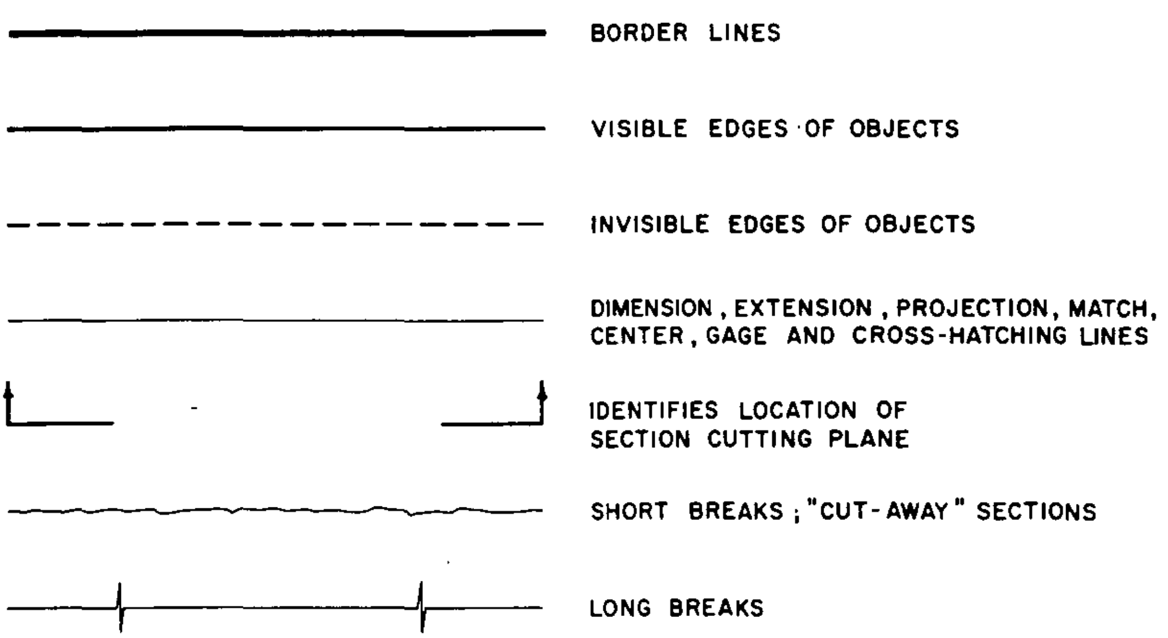


Fig. 1-1. Conventional lines for structural detailing

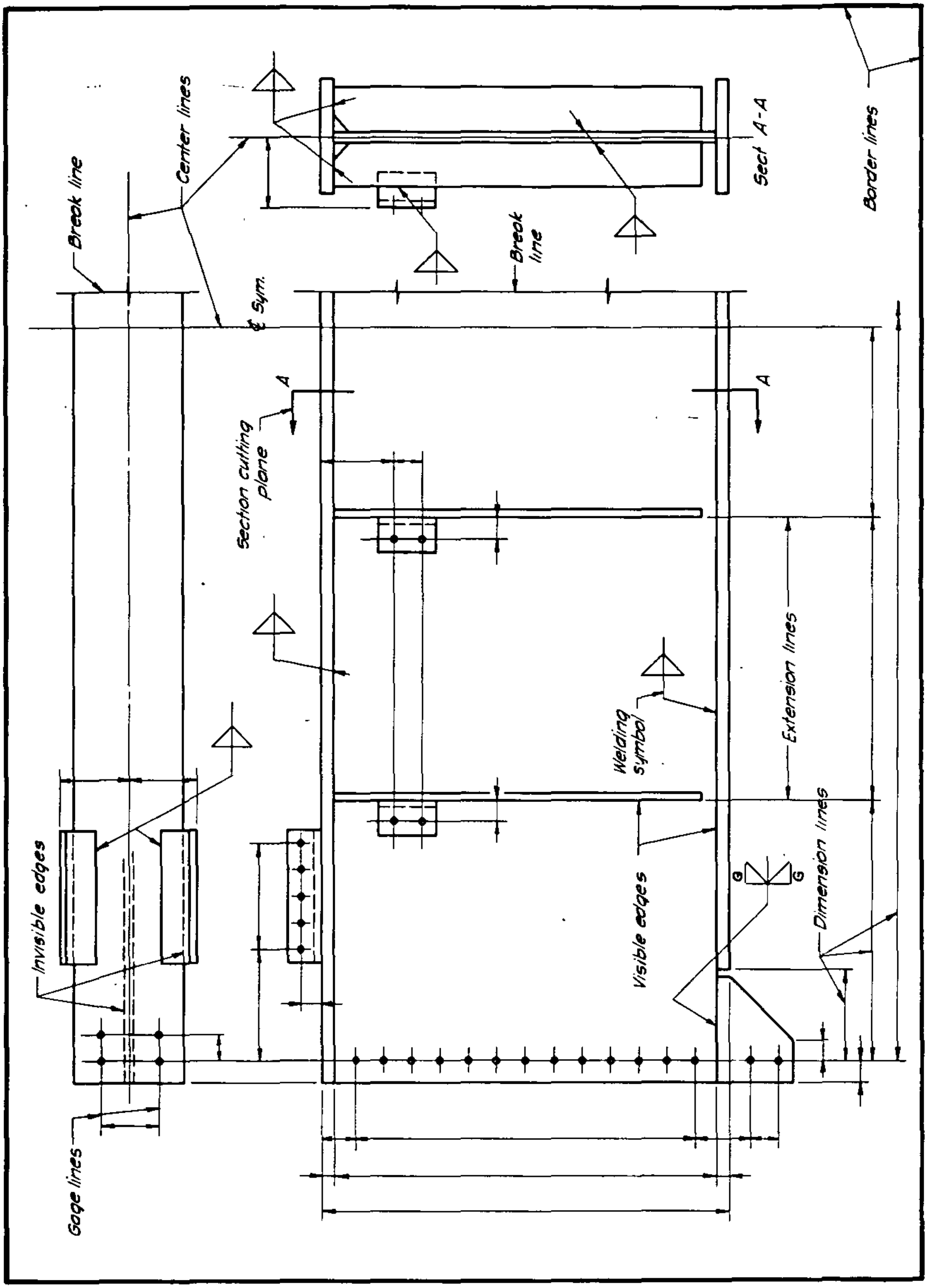


Fig. 1-2. Partially dimensioned detail of girder showing conventional lines and their relative weights

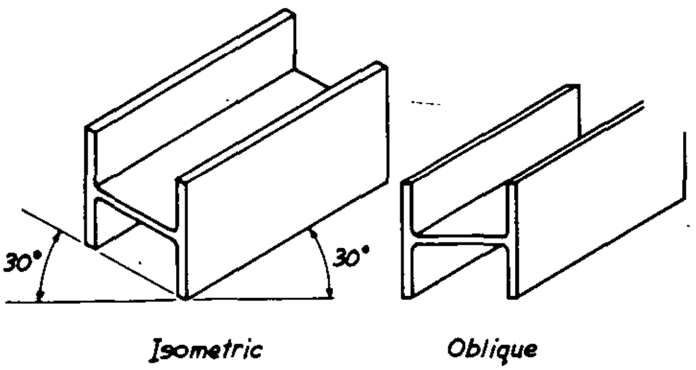


Fig. 1-3. Types of pictorial drawing

Figure 1-4 is an *isometric* drawing of a short W column in which the four faces **A**, **B**, **C** and **D** and the top are labeled. The arrows indicate the directions in which the surfaces are viewed in orthographic projection.

Figure 1-5 shows the same W column of Fig. 1-4 with the indicated views shown separately and arranged in orthographic projection. Note that this system required selection of a principal view, in this case face **B**, from which the other views are projected. With respect to face **B**, faces **A** and **C** are left and right views, respectively, projected laterally and in alignment with face **B**. In a similar manner, the top view is projected vertically from face **B**, and represents the column as seen from above.

Note that no bottom view is given in Fig. 1-5. In the illustration, the top view gives the shape description, and a bottom view, being identical, would serve no purpose. Had a bottom view been required to show detail attachments, it would have been drawn as though observed looking down the column shaft, rather than up from underneath, as would be the case in true orthographic projection. Note also that face **D** is omitted. Structural members with a single web rarely require more than one web view, as detail attachments on the far side of a web can be shown readily by hidden (dashed) lines. Had a box-shaped member required a separate view from the rear (face **D**), it would have been located to the right of face **C**.

The usual procedure in preparing an orthographic projection is to draw the principal view to scale, then project the scaled dimensions to the other views. In Fig. 1-5, with face **B** drawn to scale, the depth and flange thickness can be projected to the top view and the length to the side views **A** and **C**. The only additional scaling required is that necessary to show flange width; thicknesses should not be to scale. Proportion for clarity and location of the web and holes. The advantage of projecting as many lines as possible will be more apparent when drawing complicated built-up sections, or members with attached detail material which must appear in more than one view.

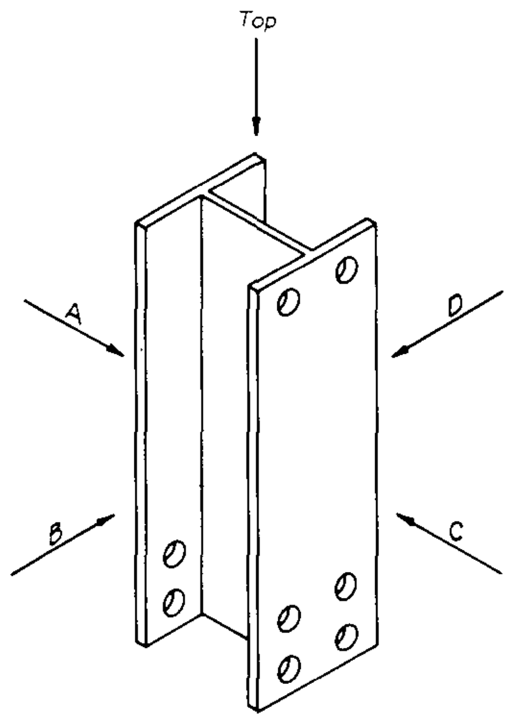


Fig. 1-4. Isometric drawing of a short W column

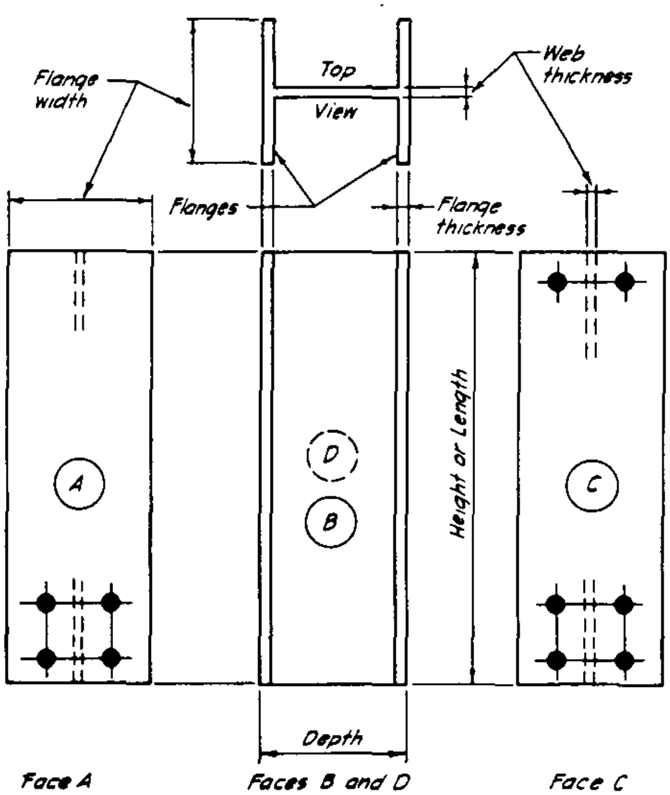


Figure 1-5

Figure 1-5 is intended primarily as an example of the relationship of views in an orthographic projection. In preparing a working drawing, the number of views needed is determined by the amount and kind of fabrication required and the attached detail material. The spacing of the views must permit adequate dimensioning and the addition of any notes that may be required. More information covering the preparation of working drawings will be found in later chapters.

The application of orthographic projection to structural drafting is governed by certain rules; however, modifications, special devices and shortcuts which have been developed by common usage through the years are generally accepted in all structural drafting rooms. Although much of this information will be absorbed gradually by the trainee as he follows the methods and examples in later chapters of this book, some of the more basic concepts will be discussed here.

Selection of Views

The principal view of a structural member should be the one which contains the most information concerning cuts, copes, hole punching and drilling, and attached fittings. In the case of beams, channels, columns and girders, which comprise the bulk of structural work, this usually means the web view. If at all possible, the principal view should show the majority of detail fittings on the side toward the observer, to minimize the need for hidden lines. An exception to this general rule relates to single channels, which are generally shown with their backs toward the observer because shop layout can be performed more easily on the flat back surface (see Fig. 1-6a). In showing hidden lines, it is accepted practice to indicate hidden edges and surfaces only partially, to the extent necessary to clarify the sketch. Long runs of hidden lines should be avoided. See Figs. 1-6a and 1-6c for permissible omissions.

Projected views should be shown only if they contribute to clarity and understanding. If top, bottom or end views are not required to show fabrication, the location of fittings, or an unusual cross-sectional configuration, they should be omitted. Since the shape descriptions of standard rolled sections such as W, S, C, and L are understood by all concerned from the written billing, the end views which merely illustrate the appearance of such a shape are unnecessary (see Figs. 1-6d and 1-6e). The projected views of most structural members follow the general arrangement shown in Fig. 1-6b. Where the bottom flanges of beams, channels and girders must be shown, they are observed from above and drawn in plan. These are, in effect, sectional views with the section cutting plane and section labeling omitted (see Fig. 1-6b).

Orientation of Views on the Drawing

Generally speaking, the principal view should be placed on a drawing in the same position the member will assume in the final structure. A general rule is to detail the member reading the erection plan from the bottom of the drawing or from the right end of the drawing. For example, beams and girders are placed horizontally on the sheet. Columns, unless the length and amount of detail require horizontal placement, are shown vertically on the sheet with lower ends at the bottom. Diagonal truss and bracing members, when not shown in place, are located horizontally with their left ends at the left side of the sheet.

Sectional Views

It is conceivable that a top or end view could be drawn to include all the fabrication and detail fittings appearing throughout the length or depth of a member—but the view, with its innumerable visible and hidden lines, would become so complicated that it would be difficult to interpret. This problem is readily solved by use of separate sectional views.

When it is necessary to show or dimension an interior detail which is not visible in the usual top or end views, it is customary to use a sectional view, located conveniently near the detail. The position of the section is established by a line representing the imaginary cutting plane. Directional arrows are added to indicate which way the cut surface is being viewed (see Fig. 1-6e). The corresponding sectional view, constructed by projection and scaling, permits picturing the detail for whatever treatment is required (see Fig. 1-2).

Sectional views are usually projected from the principal view in the same manner as end or top views. The accepted practice is to observe sections looking downward or to the left. If, due to lack of space on the drawing, the sectional view must be displaced from its normal projected position, it should retain its proper orientation and in no case be turned 90° (see Fig. 1-6e). Sometimes it may be advisable to use an offset cutting plane, so that one sectional view serves in place of two (see Fig. 1-6d). As elsewhere, linework should be used sparingly in drawing sectional views. For example, it is usually unnecessary to show the cut section of the main member when the only interest is in the detail fitting. References to center lines or other points projected from the member suffice to locate such a detail (see Fig. 1-6e).

The labeling of section cutting planes and sectional views for identification, illustrated in Fig. 1-2, is sometimes abbreviated to a single letter or number. In girder work, where a multiplicity of sections occurs fre-

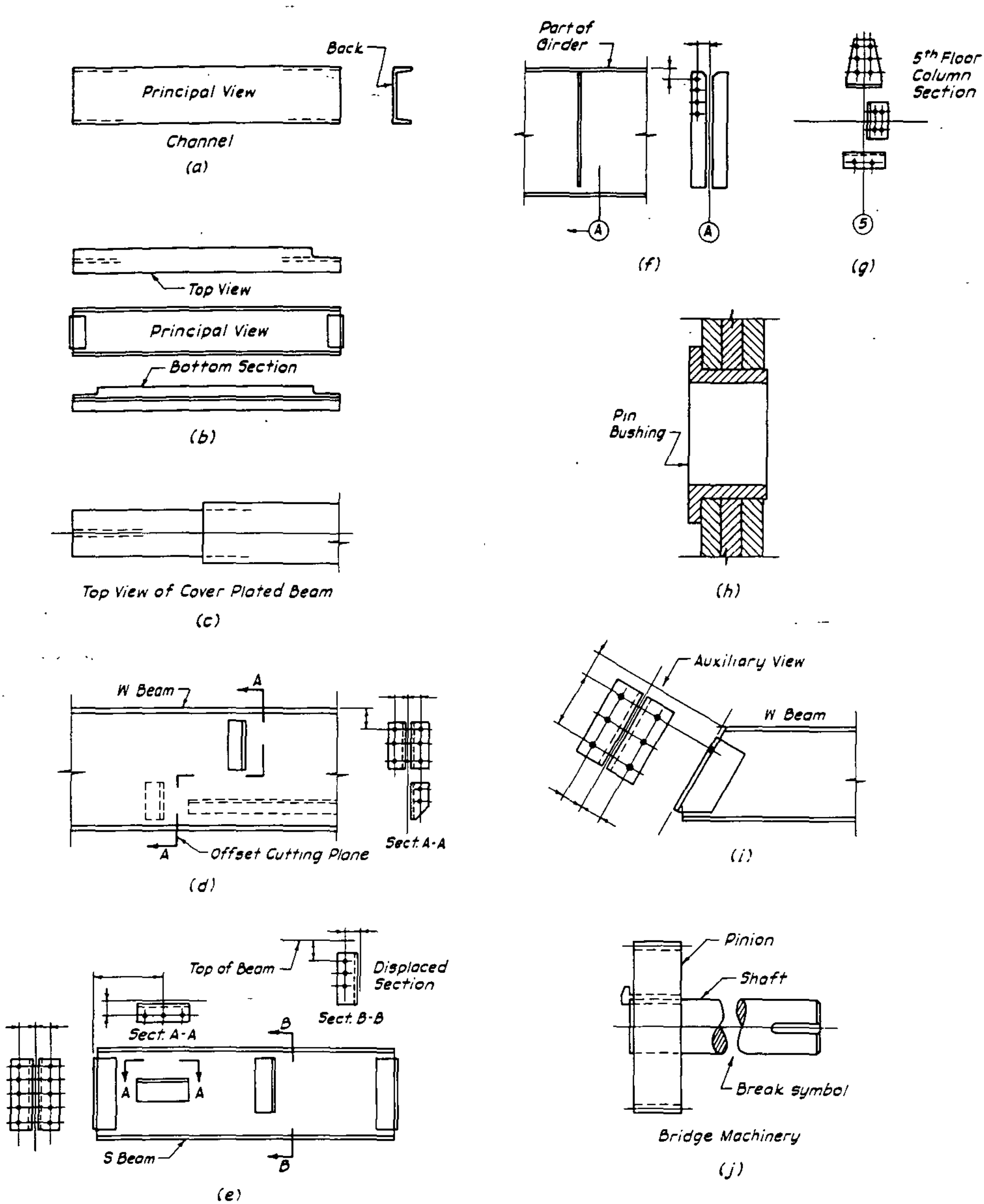


Fig. 1-6 Conventions in orthographic projection

quently, the simplified method of Fig. 1-6f is favored in many drawing rooms. In column work, where detail connection material appears largely at floor levels, the cutting plane is omitted and the sectional view, understood to be taken looking down the shaft, is identified by the floor number (see Fig. 1-6g).

The cross-hatching of the cut surfaces of sectional views is generally omitted in structural work as a time-consuming embellishment. However, it may be used to a limited extent when the adjacent parts in a complicated assembly must be identified, or when the section involves machine parts. When used, cross-hatching lines should be spaced evenly at an angle of 45°. They are sloped in opposite directions on adjacent pieces to indicate separate plies of material (see Fig. 1-6h). Although short cross-hatching lines may be drawn freehand, large areas will appear better if done with a triangle.

Auxiliary Views

When it is necessary to represent and dimension a surface which slopes with respect to the usual front, top and end views, an auxiliary view must be constructed. An auxiliary view is projected perpendicularly from the sloping surface in such a way as to present a true, undistorted appearance. The skewed detail, shown in Fig. 1-6i, demonstrates the use of auxiliary views.

Break Lines and Symbols

In structural work, *break lines* are used primarily to close the ends of members which are only partially shown on the drawing. Figure 1-2 illustrates break lines as used on girder work. Break lines should not be used to indicate foreshortening the length of a beam or column, nor should they be used to show reduction of the width or depth of any structural member. However, since machine drawing practice sanctions break symbols to show reduction in length of such parts as pipe and shafting, this custom may be retained where machinery is involved on structural details (see Fig. 1-6j). It should be emphasized that dimensions, whether on a sketch or in written billing, control all aspects of structural work. Break lines are not required as a caution against out-of-scale drawings.

Match Lines

On long members, particularly bridge girders, where the detail requires more than one sheet, it is customary to draw as much of the drawing as convenient on the first sheet of a series, and to continue the drawing on succeeding sheets until the member is complete. The

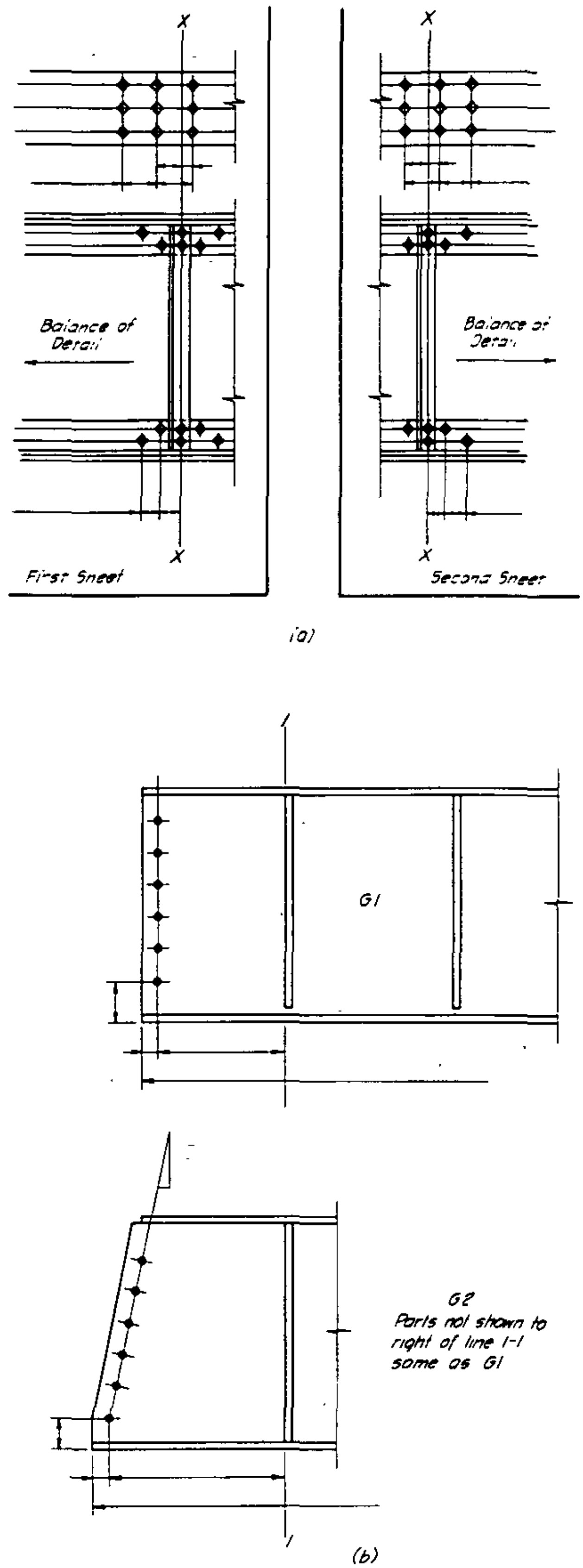


Fig. 1-7. Use of match lines

several sections of such a member are related to one another by *match lines*. Match lines are usually established at a readily identifiable point, such as a stiffener gage line, or, for welded work, the face of a bar stiffener. Match lines are tied by dimensions to the closest dimensioned feature of all the views they cross. The ends of each pair of match lines carry identical letters or numbers, as X-X, Y-Y, 1-1, 2-2, etc. (see Fig. 1-7a).

Another important use of match lines is to provide reference lines for the partial omission of drawings. In the event two members are identical except for unlike end connections, it is convenient to detail one member completely, and to detail the second showing only the points of difference. Match lines, bounding the identical portions of both members, with notes such as, "**G2**—portion between lines X-X and Y-Y, same as **G1**," or "**G2**—parts not shown to right of line 1-1, same as **G1**," provide the shop with sufficient information to fabricate both members (see Fig. 1-7b).

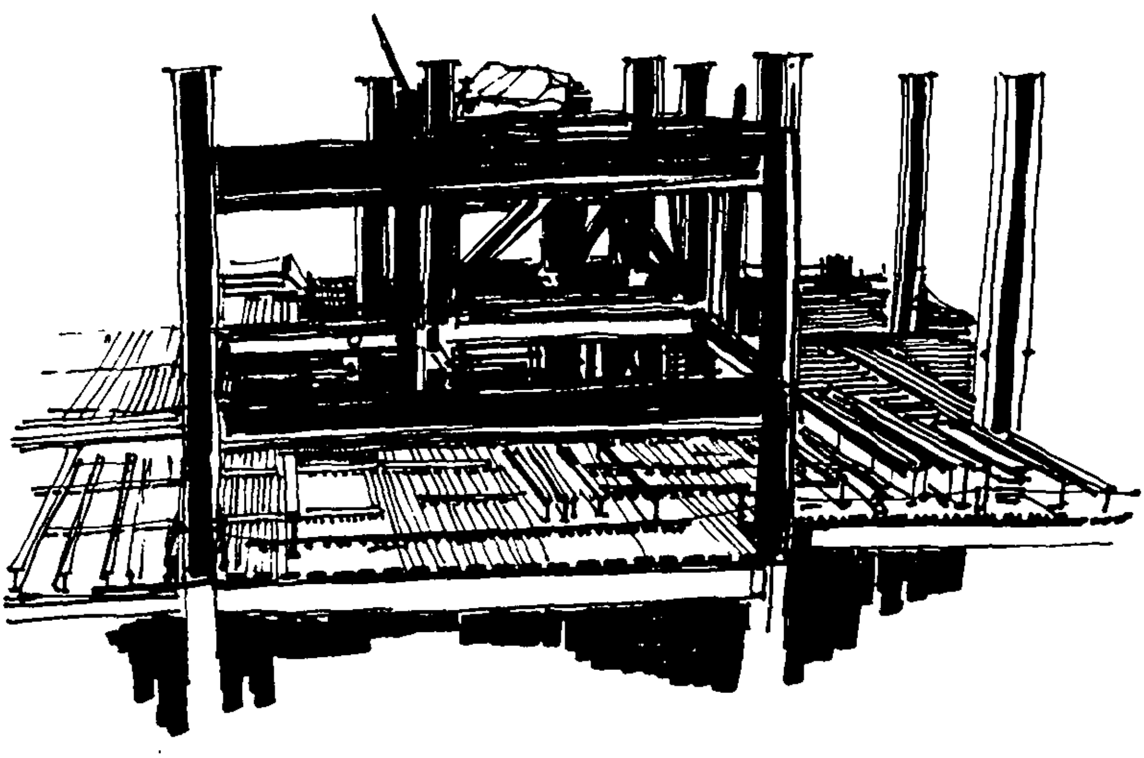
Scaling Details

It has been previously stated that structural details should be drawn to scale. Although this is generally true, there are some permissible, even inescapable, deviations from the rule. It is obvious that the length of most members makes strict adherence to longitudinal scaling impractical. As a consequence, it is acceptable to foreshorten lengths and sometimes widths or depths, to permit long or outsize members to fit on a drawing or in the space allotted. This foreshortening may be accomplished as follows: (1) by reducing the scale of all dimensions, (2) by using a smaller longitudinal than transverse scale, and (3) by reducing dimensions arbitrarily, to no particular scale, as the complexity of the detail will permit. Frequently, in detailing assembled trusses, the work line diagram is made to a smaller scale than the member detail scale. By this means the proper angular relationship between members is preserved, an adequate detail scale is possible, and the entire sketch can be contained conveniently on the sheet.

The separation between object lines which are close together is generally estimated, rather than scaled. The space is made wide enough so that the lines will not blur together on printing. This applies to the edge views of relatively thin plates, beam webs, beam flanges or angle legs. Need for this separation is apparent when it is realized that the scaled thickness of a ¼-in. plate at 1" = 1'-0" reduction is approximately 1/50-in., somewhat less than the width of a single object line!

Structural Members Billed Only

Certain types of structural material can be described adequately without benefit of any drawing. "Plain" material, not assembled to any other member in the fabricating shop, and on which no fabrication other than cutting to length is performed, may be completely described by written billing alone. This applies to all rolled shapes with square cut ends and to rectangular plates and bars. Other items, such as standard bolts, nuts and washers, and similar "purchased finished" or "shelf" items, may also be billed only, with no drawing required on the shop detail drawing—unless they must be shown assembled to another member.



CHAPTER 2

Structural Steel

The shop, where structural steel is fabricated, does not produce the steel. The steel is produced at rolling mills and shipped to the fabricating shop in a wide variety of shapes and forms. At this stage the steel is referred to as *plain material.*

PLAIN MATERIAL

The great bulk of plain material can be classified into the following basic groups:

(1) *American Standard Beams* (S).
(2) *American Standard Channels* (C).
(3) *Miscellaneous Channels* (MC). These are special purpose channel shapes other than the standard C shapes.
(4) *Wide-Flange Shapes* (W), used as both beams and columns.
(5) *Miscellaneous Shapes* (M). These lightweight shapes are similar in cross-sectional profile to W shapes.
(6) *Structural Tees* (ST, WT, MT). These are made by splitting S, W and M shapes, usually along the mid-depth of their webs. The Tee shapes thus formed are furnished as such only by the producers making the original shapes from which they are cut. However, fabricators frequently cut beam sections to form Tees in their own plants.
(7) *Angles* (L), consist of two legs, of equal or unequal widths. The legs are set at right angles to each other.
(8) *Plates* (PL) and *Flat Bars* (FLT) are rectangular in cross section and come in many widths and thicknesses. Bars are limited to maximum widths of 6 or 8 in., depending on thickness; plates range in width from over 8 up to 200 in., subject to thickness and length limitations.

A clear understanding of the various forms and shapes in which structural steel is available is essential before the draftsman can prepare detail drawings. Figure 2-1 shows typical cross sections of plain material. Note that S, C and MC shapes are characterized by tapered flanges, and that W shapes have parallel inner and outer flange surfaces. M shapes may have either parallel or tapered inner surfaces of the flanges, depending on the particular section. For details of this nature, refer to producers' catalogs. The AISC *Manual of Steel Construction* (hereafter referred to as Manual), Part 1, lists all shapes commonly used in construction, including sizes, weights per foot, dimensions and properties, as well as their availability from the principal rolling mill producers.

Table 2-1 has been prepared to show the customary methods of designating or billing individual pieces of structural shapes and plates on shop drawings, the conventional way of picturing these shapes, and the correct names of their component parts.

This system is generally accepted and used in structural drafting rooms, although some minor deviations may occur when trade name or proprietary designations are substituted for certain of the listed "Group Symbols" in billing material. Table 2-1 should be studied carefully, with particular attention given to the drafting instructions under the "Remarks" column.

CHARACTERISTICS

Steel, specifically structural steel, is fundamental to building and bridge construction. It is produced in a wide range of shapes and grades which permit maximum flexibility of design. It is relatively inexpensive to produce, and is the strongest, most versatile and economical material available to the construction industry. Steel is uniform in quality and dimensionally stable; its durability is unaffected by alternate freezing and thawing. By the addition of small amounts of cop-

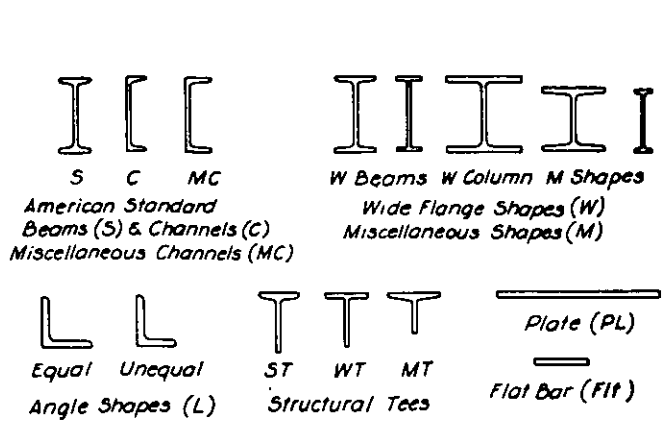


Fig. 2-1. Typical cross sections of plain material

per or other alloying elements, its resistance to atmospheric corrosion can be enhanced markedly.

Steel also has several unique qualities which make it especially adaptable to the demanding requirements of space-age construction. It can be alloyed, or alloyed and heat-treated, to obtain toughness, ductility and great strength, as the service demands, and yet be capable of ready fabrication with conventional shop equipment..

SPECIFICATIONS

Structural steel is composed almost entirely of the element iron. Small proportions of other elements, particularly carbon and manganese, must also be present to provide strength and ductility. Increasing the carbon content makes steel stronger and harder. Decreasing the carbon content makes steel softer or more ductile, but at some sacrifice of strength. The standard grades of steel used for bridges and buildings contain approximately one-fourth of one percent of carbon, with small amounts of several other elements as required or permitted by the particular steel specifications.

All steels are manufactured to specifications which stipulate the chemical and mechanical requirements in detail. Standard specifications for structural steels are established by the American Society for Testing and Materials (ASTM). Committees of this society, made up of representatives of producers, consumers and general interest groups, develop and keep up-to-date material specifications to provide and maintain reliable, acceptable and practical standards. Reference to the latest ASTM specifications is recommended for complete information on all structural steels.

Of equal importance is ASTM A6, *Specification for General Requirements for Standard Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use,* which covers in detail all aspects of mill practice and the allowances or tolerances applicable to rolled steel that must be dealt with in the fabrication process.

The AISC specification for buildings, as well as most bridge specifications, recognizes several grades of steel for structural purposes. The ASTM specifications list the scope and principal properties of these steels. As these specifications indicate, the tensile strength and minimum yield stress levels within a specific grade of steel may vary with the size of shapes and the thickness of plates and bars. Table 1 in the Manual, Part 1, serves as a quick reference to determine the availability of shapes, plates and bars by steel type, ASTM designation and minimum yield stress. For structural shapes, Table 2 in the Manual, Part 1, lists the five size groups referred to in the various ASTM specifications. A brief review of Table 1 shows that:

(a) ASTM A36 is a carbon steel with one minimum yield stress level, 36 ksi*, for all shape groups and plates and bars up to 8 in. thick. Plates and bars over 8 in. thick have a minimum yield stress level of 32 ksi.

(b) ASTM A529, also a carbon steel, has a minimum yield stress level of 42 ksi, but is limited to Group 1 shapes and plates and bars ½-in. thick and less.

(c) ASTM A441 is a high-strength low-alloy steel with three minimum yield stress levels for shapes and four levels for plates and bars. These levels, in ascending order, range from 40 ksi to 50 ksi as shape size, and plate and bar thickness decrease.

(d) ASTM A572 is a high-strength, low-alloy steel with four minimum yield stress levels ranging from 65 ksi to 42 ksi. All shape groups are available in 42 ksi and 50 ksi grades; however, only Groups 1 and 2 are shown in grade 60 and only Group 1 in grade 65. The limits of availability of plates and bars, by thickness, are also given.

(e) ASTM A242 is a corrosion-resistant, high-strength, low-alloy steel with three minimum yield stress levels: 50 ksi, 46 ksi and 42 ksi. Again, the characteristic drop in strength is exhibited as the shapes increase in size, and plates and bars increase in thickness.

(f) ASTM A588 is a corrosion-resistant, high-strength, low-alloy steel with a single minimum yield stress level for shapes and three levels for plates and bars. These levels are 50 ksi, 46 ksi and 42 ksi. This steel is unique, since the highest yield stress level applies to all shapes, plates and bars to 4 in. thick. Plates and bars over 4 in. thick have reduced minimum yield stress.

(g) ASTM A514 is a quenched and tempered alloy steel in the 90 to 100 ksi minimum yield stress range. Note that this specification includes plates and bars only. Special care must be taken in the fabrication of this steel so as not to impair its heat treatment.

Several proprietary steels, so-called because their composition and characteristics are defined by steel producers' specifications, are available for structural purposes. Producers of these proprietary steels use rigid control of melting processes and careful selection of alloys to achieve minimum yield stresses ranging in excess of 100 ksi. The toughness, weldability and cost-to-strength ratios compare favorably with those obtainable from standard steels.

**Kips per sq. in. of cross section. A kip is 1,000 pounds.*

Table 2-1. Usual method of billing and sketching structural steel shapes on shop drawings

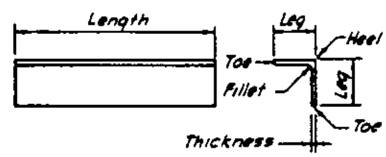
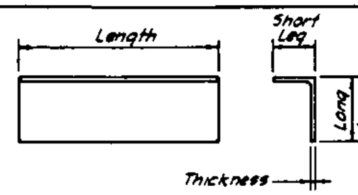
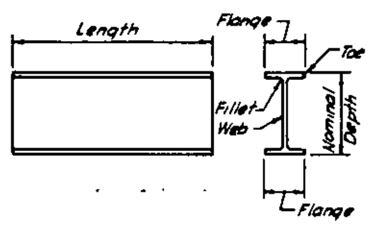
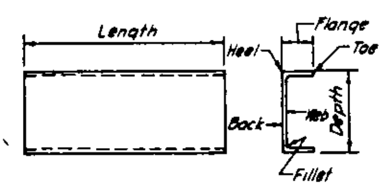
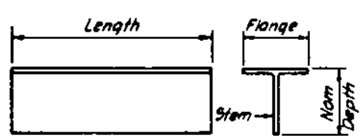
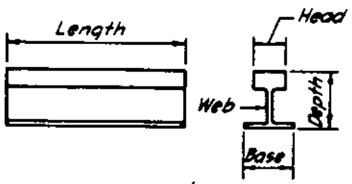
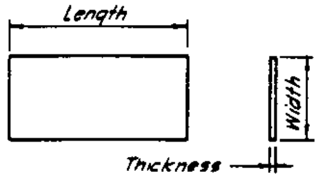
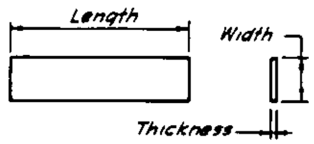
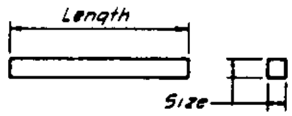
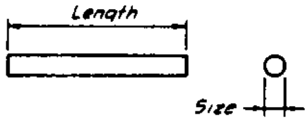
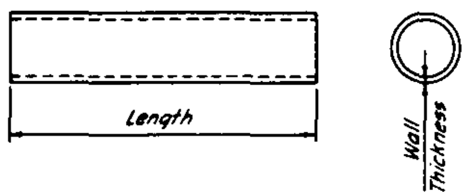
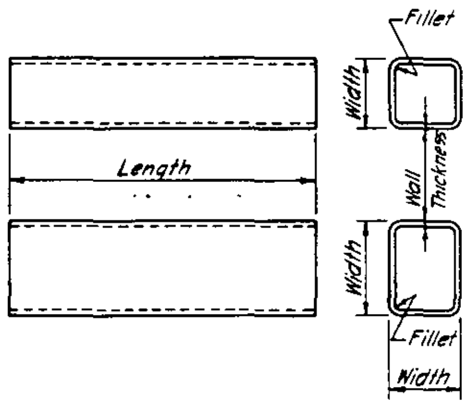
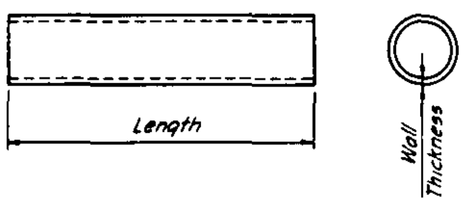
Group	Example of billing on detail drawings	Conventional way of showing on detail drawings and identification of major component parts	Remarks
EQUAL LEG ANGLES	L 3½ x 3½ x ¼ x 5′-6 Group Symbol; Leg Width in inches; Leg Width in inches; Thickness in inches; Length in ft. and in.	 	On details made to scale of 1″ = 1′-0 or smaller, do not show rounded off toes of angles or interior fillet between legs. Bill long leg of unequal leg angles first. Exaggerate leg thickness to suit.
UNEQUAL LEG ANGLES	L 6 x 4 x ⅜ x 10′-3 Group Symbol; Long Leg in inches; Short Leg in inches; Thickness in inches; Length in ft. and in.	 	
WIDE-FLANGE BEAMS & COLS. AM. STD. BEAMS MISC. SHAPES BEARING PILES	W27 X 94 x 26′-10 S15 X 50 x 16′-3½ M 8 X 18.5 x 9′-3 HP12 X 74 x 20′-3 Group Symbol and Nominal Depth in inches; Wgt. per ft. in pounds; Length in ft. and in.	 	On details made to scale of 1″ = 1′-0 or smaller, do not show rounded off toes of flanges or interior fillets between web and flanges. Do not show flange slope for channels or beams with sloping flanges. Exaggerate web and flange thickness to suit.
AM. STD. CHANNELS MISC. CHANNELS	C10 X 15.3 x 18′-8 MC13 X 31.8 x 9′-0 Group Symbol and Nominal Depth in inches; Wgt. per ft. in pounds; Length in ft. and in.	 	
STRUCTURAL TEES: W SHAPES S SHAPES M SHAPES	WT18 X 150 x 34′-6 ST 6 X 17.5 x 8′-3 MT 4 X 3.25 x 9′-0 Group Symbol and Nominal Depth in inches; Wgt. per ft. in pounds; Length in ft. and in.	 	

Table 2-1 (continued). Usual method of billing and sketching structural steel shapes on shop drawings

Group	Example of billing on detail drawings	Conventional way of showing on detail drawings and identification of major component parts	Remarks
RAILS	40# ASCE Rail x 30'-0 104# Beth. Rail x 16'-0 105# USS Rail x 21'-0 ↑ Wgt. per yd. in pounds ↑ Profile Type ↑ Group Symbol ↑ Length in ft. and in.	 	On details made to scale of 1" = 1'-0 or less, show profile about as shown herein, disregarding all rounded corners, fillets, curved and sloping surfaces.
PLATES: PLAIN-SHEARED PLAIN-UNIVERSAL MILL RAISED PATTERN	PL 3/8 x 62½ x 9'-3 UM PL ½ x 14 x 18'-11 (*) PL 5/16 x 31 x 10'-0 ↑ Producer's Designation ↑ Group Symbol ↑ Thickness in inches ↑ Width in inches ↑ Length in ft. and in.	 	On details made to scale of 1" = 1'-0 or less, exaggerate thickness of plates and bars to suit. See Manual Part 6, under "AISI Standard Nomenclature for Flat Rolled Carbon Steel" for size classification of bars & plates; see the discussion in Manual Part 1 on "Bars and Plates Product Availability" for general information.
FLAT BARS	Flt 4½ x ½ x 7'-2 ↑ Group Symbol ↑ Width in inches ↑ Thickness in inches ↑ Length in ft. and in.	 	
SQUARE BARS	Bar 1½ ▨ x 13'-4 ↑ Group Symbol ↑ Size in inches ↑ Convention for "Square" ↑ Length in ft. and in.	 	
ROUND BARS	Bar 1½ ⌀ x 12'-6 ↑ Group Symbol ↑ Size in inches ↑ Convention for "Round" ↑ Length in ft. and in.	 	

* Designations of raised pattern floor plates vary; consult manufacturers' catalogs.

Table 2-1 (continued). Usual method of billing and sketching structural steel shapes on shop drawings

Group	Example of billing on detail drawings	Conventional way of showing on detail drawings and identification of major component parts	Remarks
PIPES	Pipe 3 Std. x 8′-6 Pipe 3 X-Strong x 8′-6 Pipe 3 XX-Strong x 8′-6 ↑ Group Symbol ↑ Nominal Diameter in inches ↑ Weight Type ↑ Length in ft. and in.		See Pipe, Manual Part 1, for actual diameter and wall thickness.
TUBES: SQUARE TUBES: RECTANGULAR	TS 4 x 4 x .375 x 9′-1 TS 5 x 3 x .375 x 9′-1 ↑ Group Symbol ↑ Widths in inches ↑ Wall Thickness in inches ↑ Length in ft. and in.		Wall thickness may be shown as a fractional dimension.
TUBES: CIRCULAR	TS 3 OD x .250 x 9′-1 ↑ Group Symbol ↑ Outside Diameter in inches ↑ Wall Thickness in inches ↑ Length in ft. and in.		

Steel making is in a constant state of progress. Metallurgical research in the industry is continually developing new steels for specific purposes and improving the versatility of older steels. As time passes, and these new products prove themselves, writers of ASTM specifications prepare modifications of present specifications or formulate new ones to recognize technological advances.

PHYSICAL PROPERTIES

The terms *yield stress* and *tensile strength* are used to describe the physical properties of steels and their action when subjected to externally applied forces.

Assume that a bar of structural steel, 1-in. square and any convenient length, is clamped in a testing machine designed to pull the bar apart longitudinally. If this machine is adjusted to pull the bar so that it is resisting a force of 10.0 kips, the bar, with a cross-sectional area of 1 sq. in., is said to be *stressed* in tension at an average intensity of 10.0 kips per sq. in. (ksi). If the force is increased to 20.0 kips, the bar is stressed to 20.0 ksi, and so on.

The bar, loaded as described above, is being pulled and therefore elongated, or *strained*, in direct proportion to the stress being resisted. As the machine load increases, the bar will be stressed and strained proportionally. Within certain limits, the external forces will deform the piece of steel slightly, but on removal of such forces, the steel will return to its original shape. This property of steel is termed *elasticity*. Eventually a point is reached beyond which the elongation will continue with no corresponding increase in stress. This elongation is characteristic of ductile steels.

Mechanical testing of most ductile steels produces a sharp-kneed stress-strain diagram, as shown in Fig. 2-2. The stress at which this knee occurs is termed the *yield point*, and varies numerically for different specifications of steel. High-strength steels may not exhibit such a well-defined knee. For such steels, a *yield strength* is established in conformance with the provisions of ASTM A370 *Standard Methods and Definitions for Mechanical Testing of Steel Products*.

So as not to confuse the issue between these two concepts, the AISC Specification has established the common definition *yield stress*, which is understood to mean either *yield point* (for steels that have a yield point) or *yield strength* (for steels that do not show a sharp knee in the stress-strain relationship). The symbol F_y is used to designate this yield stress and it is expressed in kips per sq. in. (ksi).

It should be noted here that, in the elastic range, the stress-strain relationship is constant at normal temperatures, and is the same for tension or compression loadings. Furthermore, the stress-strain relationship is substantially the same for all steels, regardless of yiel stress. The ratio of stress to strain is called the *modulus of elasticity*, designated by the letter E. Numerically,

$$E = \frac{\text{stress}}{\text{strain}} = \text{approximately } 29{,}000 \text{ ksi}$$

Figure 2-2 is a theoretical diagram of the stress-strain relationship of ASTM A36 steel. The stresses at yield stress and tensile strength, shown on the curve, are minimums permitted by the specification. Often, actual test results exceed the values shown. Strain is plotted horizontally in inches per inch; stresses are plotted on the vertical scale in kips per sq. in. A straight line, representing the elastic range, starts from the point of zero stress and zero strain, and inclines upward to the right. At a stress of 29.0 ksi, for example, the strain is 0.001 in. for each inch of specimen length. At this stress, 10 in. of the 1 in. square bar will be increased in length:

$$10 \times 0.001 = \tfrac{1}{100}\text{-in.}$$

At the upper end of the inclined straight line, the yield stress, $F_y = 36.0$ ksi, is shown graphically by an uneven horizontal line, or plateau, which represents the range of plastic strain. This plastic deformatio tends to cold work the steel, causing it to strain-harden sufficiently to require an additional application of load

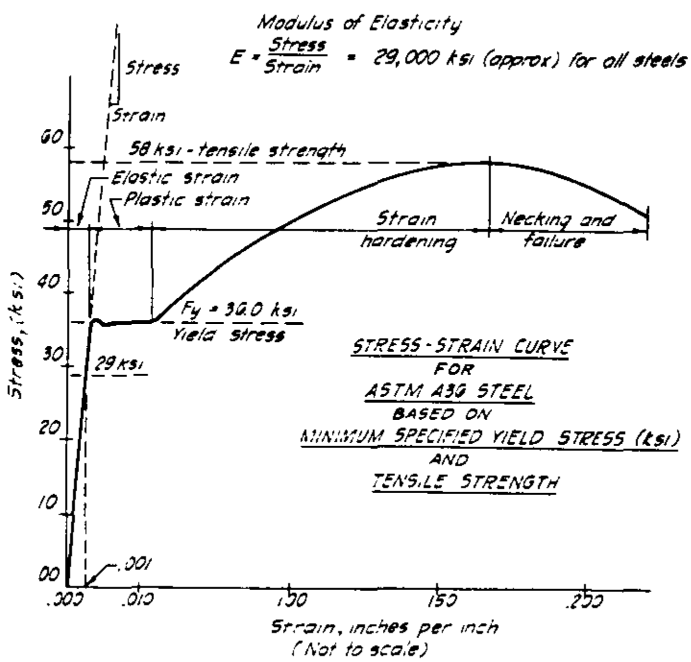


Fig. 2-2. Stress-strain relationship of ASTM A36 steel

for continued elongation. Throughout this strain-hardening range, the curve makes a long upward sweep until the tensile strength of 58.0 ksi is reached. Further elongation, or straining, is accompanied by a perceptible thinning or necking-down of the bar, a drop in the stress needed to continue the elongation, and soon thereafter the rupture of the bar.

That portion of the curve immediately following the yield stress illustrates another important property of structural steel—ductility. In this range the metal is said to be in a state of plastic strain; elongation is no longer in direct proportion to stress. Equal increments of stress are accompanied by greater and greater strains. Permanent distortion occurs and, on load release, the steel bar no longer reverts to its original length. This characteristic, termed *ductility*, provides a considerable reserve of strength, a fact which explains the capacity of structural steel to absorb temporary overloads safely. The ability of steel to support loads throughout large deformations forms the basis for plastic design. Ductility is measured in percent of elongation at rupture. For ASTM A36 steel, this is specified to be at least 20% in a length of 8 in., which means that the steel must have the ability to elongate at least $0.2 \times 8 = 1.6$ in. in 8 in. of specimen length before breaking.

Although the stress-strain diagram in Fig. 2-2 represents ASTM A36 steel, the curves for high-strength steels will have the same general appearance. Specified values of yield stresses (points or strengths), tensile strengths and ductilities may be found in the appropriate ASTM specifications.

STEEL PRODUCTION

Some familiarity with the process by which plain material is produced, and a knowledge of the more important characteristics of steel, will help the draftsman appreciate the reasons for many of the standard practices followed in his everyday work.

The production of all steel begins with the raw materials, principally iron ore, coke and limestone, or pelletized combinations of these ingredients. Predetermined amounts of each are charged into the top of a blast furnace and melted down in a continuous operation to produce iron and slag residue at the bottom. The reactions in this reducing process are highly complex. In brief, the coke, in the presence of superheated air (sometimes enriched with gas, oil or oxygen) introduced under pressure near the bottom of the furnace, burns to form a gas which removes oxides from the ore. The heat of combustion liquefies the limestone, which combines with earthy matter and other impurities in the ore to form slag, and at the same time melts the iron content of the ore. Near the bottom of the furnace the charge becomes viscous and finally, in the 3000° F heat at the hearth in the bottom, the slag and iron become molten. The lighter slag separates and floats on the heavier iron.

At this point in the process, the iron is called *pig iron*. The slag and iron are drawn off separately and the metal is cast into pigs, or transported in a molten state by a hot-ladle to the next operation.

Pig iron is a relatively brittle material, which has absorbed 3½ to 4% of carbon from the blast furnace coke, and is therefore unsuited for most structural applications. The necessary refining of pig iron to make steel is accomplished by either the electric furnace, basic oxygen, or open hearth process.

Structural steel is usually made by either the open hearth or basic oxygen process. In the open hearth process, limestone, scrap steel and molten pig iron are combined in a furnace having a shallow, dish-like hearth. Flames from oil or gas burners play over and heat this mix for 6 to 8 hours at temperatures up to 3000° F. During this time, excess amounts of carbon and certain other elements are either burned out or absorbed by the molten limestone to form slag. In the final stages of the heat, checks of the composition and temperature of the molten mixture are made to control the formulation to meet exacting specification requirements.

In the basic oxygen process, burnt lime, scrap steel and molten pig iron are charged into a barrel-shaped refractory lined furnace, open at the top. High-purity oxygen is introduced through the furnace top into the mixture at supersonic speeds with a water-cooled lance. The refining process is similar to that in the open hearth, but is considerably faster. The oxygen blowing period averages 20 minutes. Checks of the composition and temperature of the molten mixture are made at the end of the blowing cycle to control and direct the steel formulation. The tap-to-tap cycle of the basic oxygen process averages 45 minutes.

At the conclusion of these refining processes, the molten slag and steel are "tapped" separately into ladles. Elements such as manganese, silicon, vanadium and columbium are added to the steel during the tapping operation. The steel is then poured or "teemed" into vertical ingot molds. After the metal has solidified, but is still red hot, the molds are removed, leaving steel castings known as *ingots*. Depending on their end use and on the rolling mill capacity, ingots may vary widely in cross section and height: a representative average might be 2 ft square, tapered slightly from top to bottom, by 7 ft high.

Immediately following removal from the molds, ingots are placed in underground furnaces called *soaking pits*, where they are reheated to a uniform temperature to achieve the proper plasticity required for the rolling operation.

One at a time, these ingots are removed from the soaking pit by overhead cranes and moved to the primary rolling mill, where they are converted into blooms, billets and slabs. Blooms and the somewhat smaller billets are usually square or slightly rectangular in cross section, and are generally used to form structural shapes. Slabs are rectangular and are used to produce plates and sheets. In its action, the primary mill accomplishes a two-fold purpose. First, the repeated squeezing and rolling, while being passed back and forth between large rolls, reduces the cross-sectional dimensions of the ingot; second, the internal structure of the metal is worked and refined to obtain the specified physical properties. The required cross section is achieved by alternately rolling and mechanically rotating the piece so that the dimensions are reduced proportionally as the rolls are brought closer together. During this reduction of cross-sectional area, the length is greatly increased. For example, an ingot 30 in. square by 8 ft long becomes about 72 ft long when rolled into a bloom 10 in. square.

Before roughing and finishing into structural shapes, the blooms, billets and slabs must not only be cropped to remove physically and chemically imperfect material at the ends, but also must be cut into lengths which will not exceed the capacities of the mills used in subsequent rolling operations.

A recent development in steel production eliminates the conventional ingots, soaking pits and blooming or slabbing mills. In continuous casting, molten steel is poured continuously into square, round or rectangular water-cooled open end molds. The resulting product, at first exhibiting skin solidification only, is withdrawn as continuous strands from the molds to maintain a constant level of metal under the pouring nozzles. Water sprays cool the strands to complete solidification, upon which they are straightened and cut to length for further processing on conventional rolling mills.

At this stage, the steel is said to be semifinished, and may either be sent to a cooling bed and stored until needed, or transported directly to another reheating furnace. Here it is again heated to rolling temperature prior to roughing and finishing.

This final work is done on a variety of machines called *mills*. Each consists of one or more stands of rolls suitable to the size and cross-sectional shape of the finished piece. The rolls for each mill must provide a series of roughing, intermediate and finishing passes in which the bloom, billet or slab is brought down step by step from its original size to the final dimensions of a shape or plate. One type of mill is used to produce all sizes of S beams, channels, miscellaneous shapes, angles and other minor shapes, merely by changing the rolls and their spacing to suit the desired contour. A different type of mill is used to roll all sizes of wide-flange beams and columns; again, the various sizes are produced by changing rolls as required. Another type of mill is used for rolling plates, and still another for bars.

It should be noted that, although the usual procedure is to store the semifinished steel until needed for roughing and finishing as described above, it is possible and, if large orders are booked, economical to roll ingots directly in one continuous operation through the bloom, billet or slab stage to the finished product.

Whatever the circumstances, the steel producer must accumulate orders for a sufficient tonnage of a given shape to warrant the expense of shutting down the mill for roll changes. This situation requires advance planning for production of the various shapes according to a rolling schedule. For example, shapes for which there is limited demand may be rolled once every six months, while other more popular items may be scheduled for monthly rollings.

The American Standard beam (S) is rolled with the web in a horizontal position. The final finishing passes are made through rolls so shaped that one-half of the required cross-sectional profile or contour of the beam is formed by each roll. The soft, hot metal is forced or squeezed into deep grooves in the rolls to form the flanges of the beam. This method of forming is limited to beams of relatively narrow flange width.

The rolls for channels, angles and other minor shapes are also grooved to form the contour of the particular shape. These shapes are rolled on the same mill used to produce S beams, all rolls being horizontal.

Wide-flange (W) shapes are also rolled with their webs horizontal. These beam and column shapes receive somewhat more intricate rolling mill processing on a special mill. The characteristic wide-flange profile requires alternate passes in both the roughing and intermediate stands. Work is done first on the edges (toes) of the flanges, and then on the entire cross section except the flange edges. One set of horizontal rolls forms the web and inner surfaces of the flanges; a separate set works on the flange edges to establish flange width. Still a third set of rolls, with their axes vertical, works on the two outer flange surfaces. These control the depth of the shape and form the flange metal to widths much greater than is possible with grooved horizontal rolls. The finishing pass gives the shape its final "truing-up," with both horizontal web and vertical flange rolls usually in the same stand.

Plates are produced from slabs on a wide variety of mills, all of which follow the basic principle of squeezing the hot metal between smooth cylindrical rolls adjusted to form the required width, thickness and length. Depending on the width of plate required, or its end use, the slab may be rolled longitudinally, or in a direction transverse to its length, or in alternating directions, until the required dimensions have been reached. *Sheared plates* are generally the wider plates which are cut to required width and length by shearing or flame cutting after rolling. *Universal mill plates* are produced on a mill which, in addition to horizontal rolls, has a set of vertical rolls to control the width of plate and provide a relatively smooth, straight edge; such plates are cut only to length.

Raised-pattern *floor plates*, for walkway and roadway applications, are rolled in the same manner as other plates, except that one of the finishing rolls contains indentations which produce a continuing pattern of raised lugs on one surface of the plate. Although each producing mill has its own pattern, all such plates have about the same anti-skid properties.

Square and round bars are formed on a mill with rolls grooved to suit the size of the shape involved. The steel is passed back and forth through successively smaller grooves until the desired size is reached. More frequently, bars and bar shapes are made on a continuous mill with a series or train of rolls, lined up to permit continuous rolling of the material in one direction only. In this type of mill, each set of rolls must rotate more rapidly than the preceding set in order to keep pace with the elongating metal.

An increase in the weight per foot of beams, columns, channels, angles and plates of any particular nominal size is accomplished by adjusting the spread of the finishing rolls. The way this is done for the most commonly used shapes is explained and illustrated in the Manual, Part 1, in the section on "Standard Mill Practice." Note that as American Standard beam and channel weights are increased by thickening the webs, their flange widths are increased, but their depths remain unchanged. Similarly, as leg thicknesses of angles are increased, the leg widths may become slightly oversize. In the case of wide-flange shapes, as weights are increased their dimensions increase in two directions. The spreading of both the horizontal and vertical rolls produces greater depth, web thickness and flange width, so that one nominal size includes a number of different actual depths. Plates, of course, present no problem; any required thickness can be produced merely by spreading or closing the rolls, and any width can be obtained by shearing or edge rolling, as previously discussed.

In spite of this seeming flexibility, a purchaser cannot specify just any weight or size he wishes. If producers were to accept such orders, not only would the finishing rolls have to be reset for each order, but also it would be necessary to identify each shape at the finishing pass and keep it separated from all other orders until it had been finally shipped. Any surplus left over from rolling a billet would have to be scrapped, or stored on the possibility that some other purchaser might happen to want the same size. Such operations would slow down production seriously and greatly increase the cost to all purchasers.

Commercial practice, coordinated by AISI*, has established a series of fixed-size shapes with a sufficient range of dimensions and intermediate weights per foot to satisfy all reasonable requirements. The extent of standardization achieved is evident from a study of the listings under "Dimensions" or "Properties" in the Manual, Part 1. Note the relatively small gradations in dimension of the successive shapes included under any one nominal size. For further information, read the discussion on "Structural Steels," at the beginning of the Manual Part 1.

It should be noted, this standardized series of structural shapes is far from static. From time to time, improvements in production technology and changes in construction trends result in introduction of new shapes and elimination of less efficient shapes, as well as extension of established popular series of shapes by inclusion of new lighter or heavier sections.

MILL TOLERANCES

Mill tolerance is a term used to describe permissible deviations from the published dimensions of cross-sectional profiles, as listed in mill catalogs and in the Manual, and from the thickness or lengths specified by the purchaser. The variations are negligible in small shapes, but increase and must be taken into consideration in detailing and fabricating connections for members made up from larger shapes. Other mill tolerances permit some variation in area and weight, ends out-of-square, and camber and sweep.

Factors which contribute to the necessity for mill tolerances are:

(1) The high speed of the rolling operation, required to prevent the metal from cooling before the rolling process has been completed
(2) The varying skill of the operators in adjusting the rolls for each pass, particularly the final pass
(3) The deflection (springing) of the rolls during the rolling operation
(4) The gradual wearing of the rolls, which can result in some weight increase, particularly in the case of shapes
(5) The warping of the steel in the process of cooling
(6) The subsequent shrinkage in length of a shape which has been cut while still hot

Rolling, cutting and other tolerances attributable to mill production of structural shapes and plates are

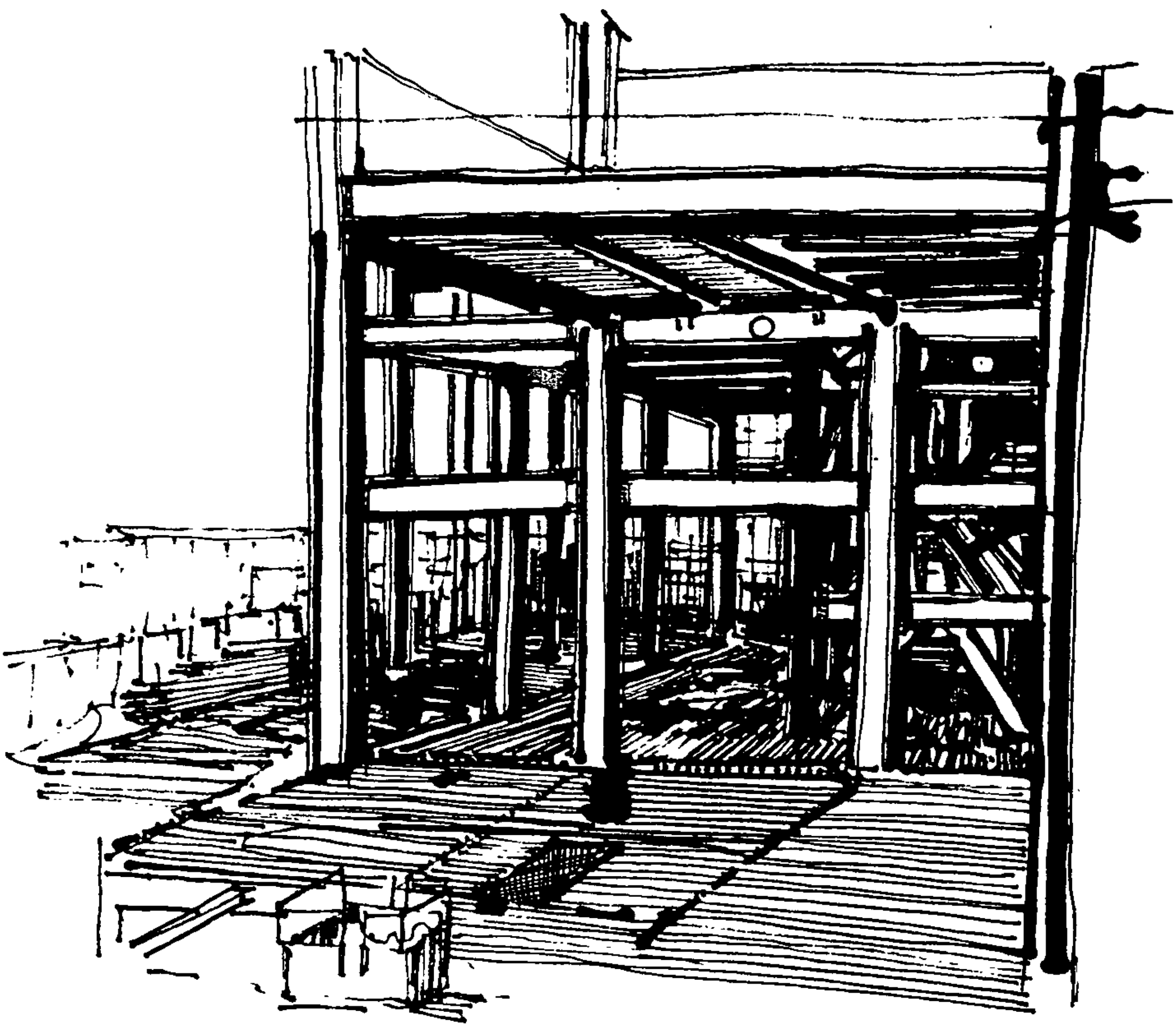
*American Iron and Steel Institute

shown in the Manual, Part 1, under "Standard Mill Practice." A more exhaustive presentation is found in the ASTM A6 *Standard Specification for General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use.*

It is most important that the effect of mill tolerances be clearly understood by the draftsman. He must know when to take them into account, particularly in ordering mill material and in detailing connections involving heavy rolled shapes.

With the coming of computerized design in the field of structural steel, engineers are able to design structures with a smaller margin between actual stress and allowable stress. This requires an exactness in chemicals and physicals in steel making which must be maintained by rigid quality control. Heat numbers and test reports for identification of every individual piece are necessary in some structures.

A means of identification is tied into the detail drawings for accurate traceability.



CHAPTER 7

Columns

At the start of a new job, shop drawings of the columns are usually made first. This is necessary because columns are generally more complex than other members of a structure, and require more lead time for detailing and fabrication to insure shipment and erection in proper sequence. This practice makes it easier to schedule other detailing work for the entire squad.

Before starting shop drawings of columns and beams, the drafting department and shop management must determine the fabrication method that will be used for both shop and field connections. Methods change with new developments, but at the present time the following systems are common:

(1) Shop welded and field bolted
(2) Shop and field bolted
(3) Shop and field welded

The shop preference for a particular system varies with the available equipment and shop experience. Fabricating plants which have equipment and shop layout adapted to punched or drilled work, perhaps with some machines operated by numerical (tape) control, may lean toward the use of bolts; other shops may be better suited for welding and prefer that all shop connections be welded. Many can handle either type of fabrication, but to balance workloads between shop areas prefer to select the connection system on a job-to-job basis.

Generally the connection system is selected by the designer and indicated in the design drawings and the job specifications. However, the designer may be receptive to a change. A change in connection systems should be approached with caution. For example, a member sized for gross area in a welded design may not have adequate net area in a bolted design. The effect on erection procedures and field connections should also be considered. The final decision is reached by agreement and approval by the fabricator and the purchaser.

A parallel but equally important decision is to determine on which members the detail material will be assembled, i.e., the column shafts or the beams. For instance, columns may have considerable detail fittings, splice plates, etc., attached to them. If all this detail material can be assembled and attached to the columns in the shop, the plain beams (no fittings) can bypass the shop assembly area and go directly to the inspection, painting and shipping areas—an efficient procedure. Special connections and conditions may require some compromise. Any deviations are resolved by management as the drafting proceeds.

Since the greater part of the connection material may appear on the columns, preliminary planning is helpful even before detailing is started. In the case of large tier buildings this includes an advance preparation of details covering job standards for wind bracing connections, column splices and other features which repeat throughout the structure. Column and beam gages are determined, and layouts of bracing connections and standards for framed and seated connections are made. Job standards involving connection material are customarily placed on separate sheets for reference in the drawing room and for use in the template shop.

The shop details required for even a relatively small project are seldom produced by one detailer working alone. The tight time schedules of most contracts may require from two to ten detailers working on a single floor level of framing. Early development of complete column details speeds the work and minimizes debates which sometimes arise when several detailers attempt to work up connections to the same column. A well developed set of job standards goes even farther in this direction by providing common standards for both column and beam detailers.

ENGINEERING DESIGN DATA

The information needed for detailing columns, as well as other structural members, is found primarily on the structural design drawings. These drawings show, by plans, elevations, sectional views, enlarged details, tabulations and notes, the size and location of all parts of the structural frame. They should include all information necessary for complete detailing.

Plan views show the location of column centers and indicate the orientation of column faces. Beam and girder framing which appears on column centers is assumed to connect at the center of the column web or flange. Since the structural plan is generally a small scale line diagram, enlarged sections are sometimes employed to locate off-center beams and to clarify special framing conditions. This is particularly true for perimeter (spandrel) framing, beams around stair wells and ramps, and members at elevator openings. Enlarged parts of the plan, such as those adjacent to corner columns, may be used to indicate the designer's solution or to alert the detailer to complex situations.

Beam connections to columns may be designed to resist wind or earthquake forces in addition to vertical floor loads. Such special connections are sometimes sketched and tabulated on the design drawings and keyed to the beams by numbers or symbols. Ordinary framed or seated connections are usually designated by note or specification reference, as are the fasteners or welds to be used. When vertical bracing, trusses or built-up girders are required, the necessary views are shown in vertical sections or exterior elevations.

The previous discussions on the design of connections have, on the whole, not considered either the type and/or size of column to which they were connected—nor did they need to have such consideration. In the investigation of the column design and detail, it is most important that careful consideration of the connection design be included. A rigid (Type 1) or semi-rigid (Type 3) connection depends on the inherent stiffness of the column and the beam, but mostly on the method of attachment, i.e., the connection detail, stiffeners, gages, etc., in order to develop the calculated shears and moments. A simple (Type 2) connection is assumed to transfer shear forces only.

This chapter will discuss the various types of connections primarily as they apply to the W-shaped columns. The same principles apply to other designs which might be fabricated to special shapes. The connection methods in current practice are essentially welded or bolted, although riveted construction is frequently encountered in older structures which may be undergoing renovation, modification or extension.

TYPES OF COLUMNS

The most frequently used columns consist of 8, 10, 12 and 14-in. W shapes. Even though design conditions sometimes require sections built up of several components, designers utilize W shapes, as rolled, whenever practical. In Fig. 7-1, W columns, coverplated W columns, and several types of built-up columns are shown. Special H or I shaped columns and box sections, sometimes with interior webs, are made by welding together web and flange plates. Double or triple shaft columns, laced, battened or connected with diaphragms, are used in mill buildings where crane runway and roof supports are combined in one member. Tubular columns of round, square or rectangular shape are used in light structures and, for architectural reasons, often supplant W sections in schools and small commercial buildings.

COLUMN SCHEDULES

To furnish the fabricator information on the size and length of columns required in a tier building, the designer prepares a *column schedule*, similar to the one shown in Fig. 7-8. Columns are identified and oriented on the design plans by an appropriate symbol, usually

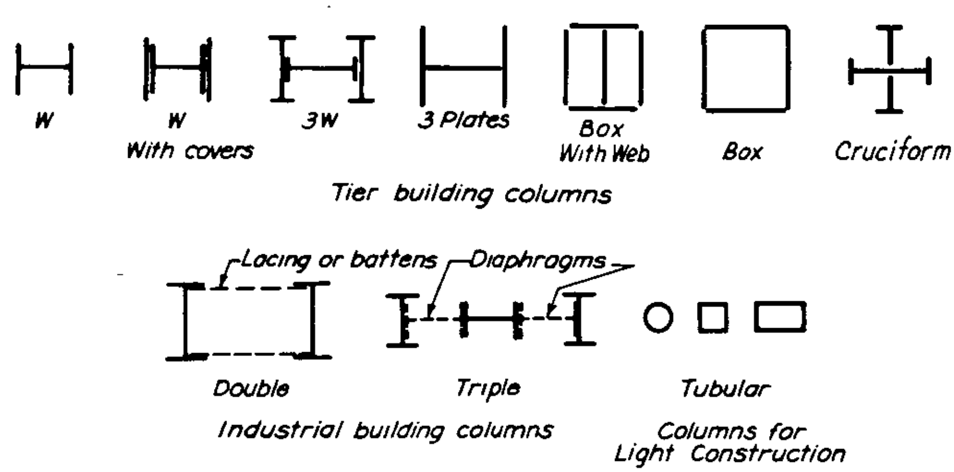


Figure 7-1

the column shape in cross section, and are located by a system of numbering. Their location may be established by using either a simple numerical sequence, as **1**, **2**, **3**, etc., or a two-way grid system, with column center lines assigned letters in one direction and numbers in the other direction. Thus, a column at the intersection of lines **D** and **4** would be **D4**.

The required size and makeup of a particular column, including loading, is given in the column schedule. As the total load supported by a column increases through an accumulation of loads from each level of framing, the size of the column usually increases. The schedule shows the column sizes and specifies the elevation at which the sizes must change. For reasons of economy in fabrication and handling, splices usually occur at every second or third level. Thus, each individual column length supports two or three floors, termed a *tier.* Horizontal reference lines in the column schedule represent finished floor lines or some other reference plane. Elevations of floor framing, as well as column splices, are referred to by note or dimension to these lines. Bottoms of columns (or tops of base plates), and the "cutoff points" at the column tops are similarly located.

The size and length of columns in low buildings of one or two stories, where the same section may be used from top to bottom, usually are shown on the plans and in elevations or typical sections.

The location of the column splice can affect the cost of a high-rise structure. The following situations are cited for consideration:

1. Since the lower column tier is normally heavier, it is appropriate to keep the splice level as low as possible in order to reduce weight of materials.

2. The elevation of the splice must provide sufficient space to allow for the splice plate and beam connection to be made without interfering with each other. If it is a braced structure, there should be sufficient space for the bracing connection. It is very undesirable for the column splice to share fasteners with or be dependent upon some other connection.

3. The splice elevation should accommodate the ironworker who will make the connection. It may appear desirable to splice a column at the mid-height or point of contraflexure, but since this is several feet above the steel framing it can require additional expense in the initial connecting of the next higher tier, in the installing and tightening of permanent bolts, or in the field welding of the splice, since scaffolding can be required for access. This is particularly troublesome during the erection of the next tier and is sometimes an unsafe procedure.

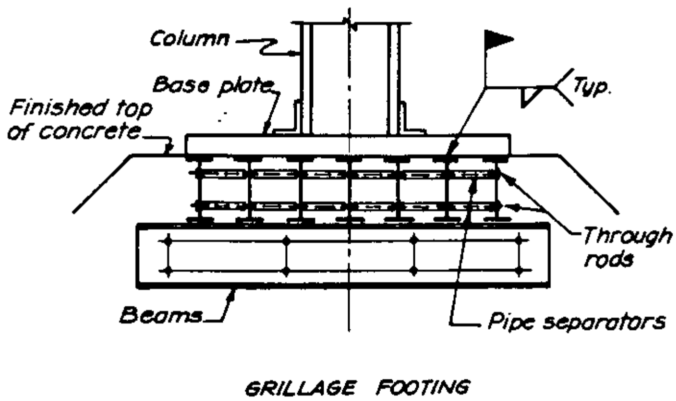


Figure 7-2

COLUMN BASES

Base plates distribute the column loads over an area of foundation large enough to prevent crushing the masonry. The size and thickness of base plates are usually listed at the bottom of the column schedule. For extremely heavy loads in major structures, or where subsoil conditions are poor, the designer may distribute the column loads by using a grillage (see Fig. 7-2). This consists of one or more layers of closely spaced beams (usually S shapes because of the thicker webs) encased in the concrete foundation.

Although the construction of foundations is not a part of the fabricator's work, masonry design plans may show certain items which the fabricator is required to furnish. These may include anchor bolts, leveling plates, grillages, machinery supports, curb angles and other embedments. These items are ordinarily shipped in advance and are placed by the masonry contractor prior to steel erection. The masonry plan usually shows typical column base details. An example of a masonry plan is shown in Fig. 7-8.

An *anchor bolt plan* is prepared by the fabricator concurrently with the details of advance material. This plan, which is similar in appearance to the masonry plan, gives complete information for field placement, including erection marks, elevations at the tops of base plates and leveling plates (elevations should not be given to the bottom of a base plate), grout thickness and the projection of anchor bolts above the top of concrete. Although much of this data may be taken from the masonry plan, it should be verified by any other means available. This is particularly true for elevations of the tops of footings, which must be compatible with base plate elevations. Also, the orientation and location of columns must agree throughout all tiers

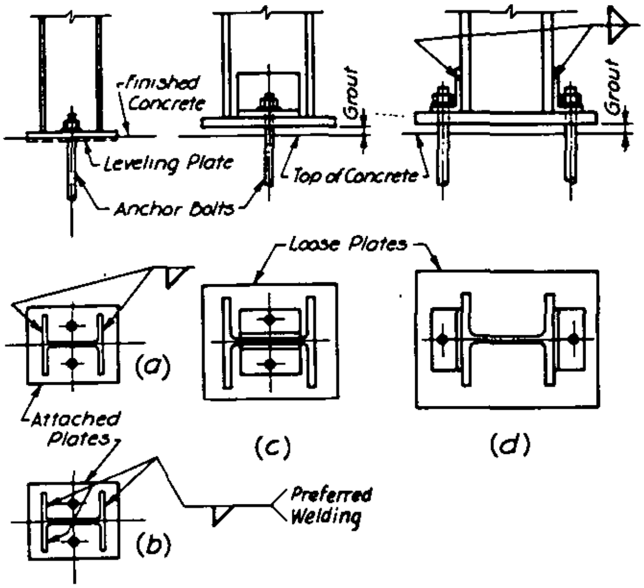


Figure 7-3

above the foundations. Attention paid to these details at the outset of a job will save much time and expense at a later date.

Small base plates, as shown in Figs. 7-3a and 7-3b, are often attached to the bottoms of columns in the shop. The difficulty of supporting such columns while leveling and grouting their bases makes it advisable to provide footings finished to the proper elevation. The required smooth bearing area is usually achieved by means of a steel leveling plate approximately 1/4-in. thick. This is easy to handle and set level to elevation prior to erection of the columns. Holes serve as a setting template for the anchor bolts. Very light columns may be set with wedges or shims in lieu of a leveling plate.

Leveling plates and loose base plates that are small enough to be set manually are placed by the masonry contractor (see Figs. 7-3c and 7-3d). Larger base plates that must be lifted by a derrick or crane are set to elevation and leveled by the steel erector. This is accomplished either by using shims of various thicknesses (see Fig. 7-4a), or by leveling screws with weldments to the edges of the base plate (see Figs. 7-4b and 7-4c). The top of the rough masonry footing is purposely set 1 in. or so below the bottom of the base plate to provide for adjustment and subsequent grouting. Cement grout is worked under the plate to insure full bearing under the entire plate area. For large base plates, the design should call for one or more large-size holes near the center of the plate through which grout is poured to obtain an even distribution. If the structural contract includes steel anchor bolt setting templates, it is customary to furnish light plates, similar in all respects to leveling plates except that the overall size need be only large enough to include the bolt pattern.

In lightly loaded structures, tall narrow frameworks, and mill buildings where crane loading is a factor, horizontal forces may tend to overturn columns, or cause an uplift from the base. To resist these forces, anchor bolts are used to tie the column to the foundation. Anchor bolts also serve to locate and to prevent displacement or overturning of columns due to accidental collisions during erection.

For ordinary size anchor bolts, 1 1/4-in. dia. and less, heavy clip angles bolted or welded to the columns, as shown in Figs. 7-3c and 7-3d, are generally adequate

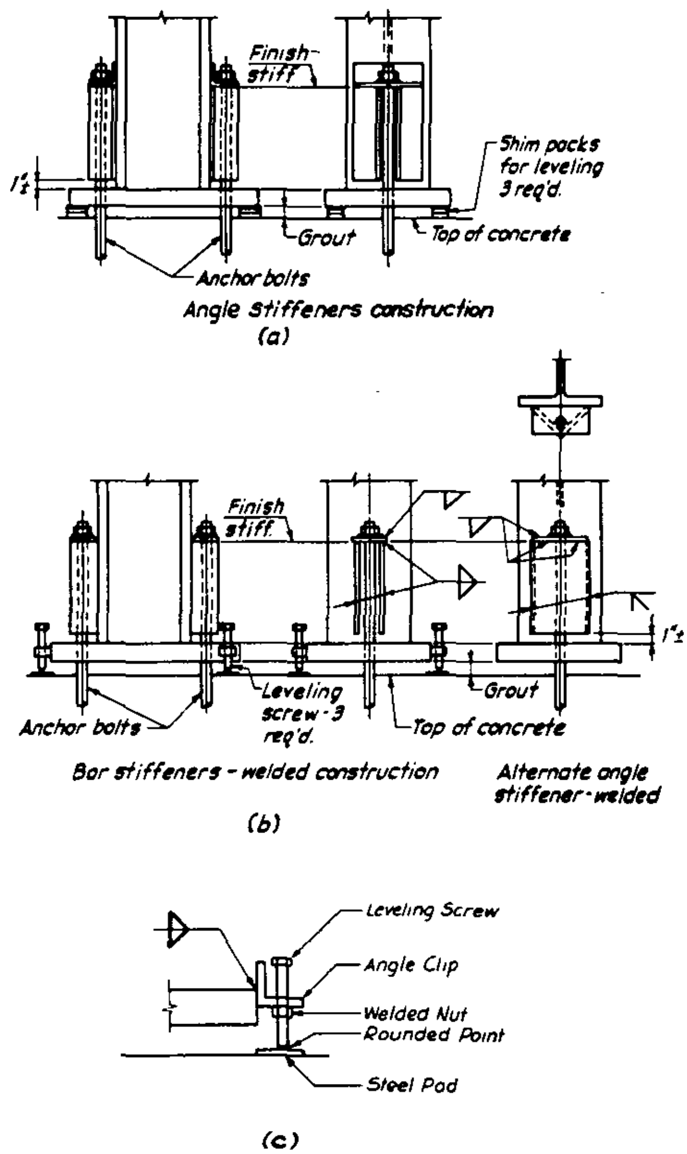


Figure 7-4

Table 7-1. Recomended Hole Sizes For Anchor Bolts

Bolt size	Hole size
3/4" to 1" incl.	Diameter + 5/16"
Over 1" to 2" incl.	Diameter + 1/2"
Over 2"	Diameter + 1"

to transfer overturning or uplift forces from the column shaft to the anchor bolts. When a more positive anchorage is needed to provide against uplift or to resist a calculated moment force, stiffeners are employed with horizontal fitting angles or bars. In such cases, the design plans should contain sketches and design of the required base details (see Fig. 7-4).

Table 7-1 gives recommended hole sizes in steel members to accommodate anchor bolts. The oversize permits a reasonable tolerance for misalignment in setting the bolts and permits more precision in the adjustment of the base plate and column to their correct center lines. The oversize hole should be covered with a flat washer.

Anchor bolts are sometimes located and drilled into the foundation after a piece has been installed in final position. The details should be arranged and dimensioned to permit access and clearance for the drilling. The bolts should be spaced to miss the reinforcing bars.

Insert holes are sometimes precast in the foundation. These holes are oversize and accommodate a "swedge" type bolt which is grouted in the hole, usually with the piece installed in final position. These holes should be sealed in locations subject to freezing, to avoid spalling of the foundation by the freeze-thaw cycle of water-filled holes.

Observe that the angle or bar stiffeners in the moment base of Fig. 7-4 are cut back about 1 in. from the base plate. This eliminates a pocket and permits drainage to protect the column base. These stiffeners are intended to resist uplift from an overturning moment and are not usually designed as part of the column area in bearing on the base plate. The clip angles shown in Figs. 7-3c and 7-3d preferably should be set back from the column end about 1/8-in. for the same reasons.

Base Plates

In the absence of specific job requirements, the surface preparation of rolled steel base plates is governed by AISC Specification Sect. 1.21.3. This section stipulates that if satisfactory contact in bearing is present in plates 2 in. or less in thickness, machining is not necessary. Plates over 2 in. through 4 in. in thickness may be either straightened to obtain this contact, or finished at the option of the fabricator. To insure satisfactory flatness, all unfinished base plates and leveling plates are noted "Straighten" on detail drawings. Plates over 4 in. thick must be finished. However, finishing is not required on the underside of base plates when grout is used to insure full contact on the foundations.

When finishing is required, as for **BP2** in Fig. 7-5, the plate must be ordered thicker than the specified finished dimension to allow for the cut. Finish allowances will vary, depending on the overall dimensions and thickness of the plates.

Table 7-2. Finish Allowances (Carbon Steel)

Size	Thickness, in.	Add to Fin. One Side, in	Add to Fin. Two Sides, in.
Maximum dimension 24" or less	1 1/4 or less	1/16	1/8
	Over 1 1/4 to 2 incl.	1/8	1/4
Maximum dimension over 24"	1 1/4 or less	1/8	1/4
	Over 1 1/4 to 2 incl.	3/16	3/8
56" wide, or less	Over 2 to 7 1/2 incl.	1/4	3/8
	Over 7 1/2 to 10 incl.	1/2	5/8
	Over 10 to 15 incl.	3/4	7/8
Over 56" wide to 72" wide	Over 2 to 6 incl.	1/4	3/8
	Over 6 to 10 incl.	1/2	5/8
	Over 10 to 15 incl.	3/4	7/8

Table 7-2 provides information on finish allowances for carbon steel for a variety of plate widths and thicknesses and for finishing one or both surfaces. These tabulated finish allowances are based on many years of experience and have been proven satisfactory for structural work. Manual Part 1 lists mill flatness tolerances for both carbon and alloy steels, and an adjustment should be made in applying Table 7-2 to alloy steel base plates in proportion to the differences with carbon steel. Base plate thickness should be specified in multiples of eighths of an inch.

Since no useful purpose is served by finishing more than the area in contact with the finished end of the column, the shop detail is dimensioned to show the area on which finishing (Fin.) is required (see Fig. 7-5). To reduce machine time, the cut should be made in the direction producing the least possible finished area. The finishing is usually carried across the full width of the plate to avoid interrupted machining operations, although it is not required from a design standpoint.

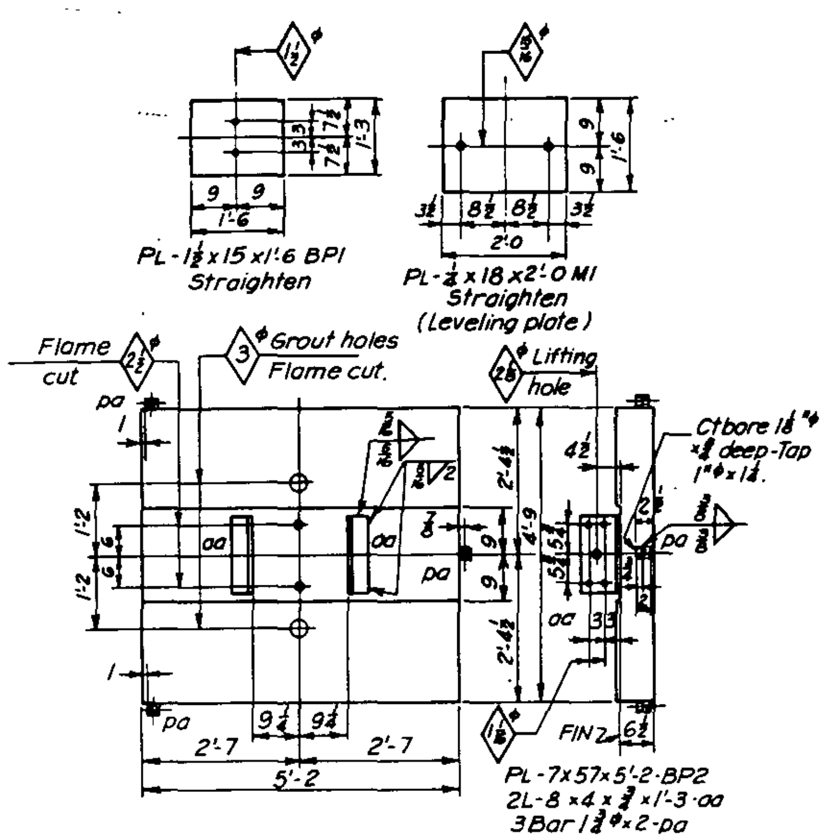


Figure 7-5

Holes are punched in those plates which are within the machine capacity of the fabricator. Thicker plates must be either drilled or flame cut to provide the holes. Most fabricators are limited to a maximum drilling capacity of 1½-in. diameter holes. From a practical and economic standpoint, the design should permit flame-cut holes when the holes are over about 1½-in. diameter in any thickness of base plate. The flame-cutting operation produces holes with slightly tapered walls and the holes should be inspected with a go-no-go gage to assure proper clearances of anchor bolts. Grout holes do not require the same accuracy of size and location and generally are flame cut. Heavy base plates should be provided with some means of handling at the erection site. On **BP2**, lifting holes are provided in the vertical legs of the connection angles (see Fig. 7-5).

Various means have been developed for use in leveling the base plates at erection. Three-point leveling is normally used, since it is faster and more sensitive to adjustment. Figure 7-4 indicates three leveling screws on the base plate. A threaded attachment is welded to the base plate and may consist of a welded nut, a threaded bar as in Fig. 7-5, or an angle and nut as shown in Fig. 7-4c, depending on the weight of the plate. The leveling screw must be long enough to compensate for the grout space and preferably should have the point slightly rounded to prevent it from "walking" as it is turned down. A small steel pad under the point reduces friction.

It is not intended that the leveling screws or the shims support the weight of the column. If the grouting is to be delayed until after steel erection, it is required that adequate shim packs be installed to properly distribute loads into the foundation without overstressing either the base plate or the foundation.

Anchor Bolts

Anchor bolts for structural work may take any of the forms shown in Fig. 7-6. Although not suitable for high strength steels, perhaps the most commonly used are hook bolts, which are illustrated in Fig. 7-6a. Their resistance to uplift is due largely to the bond formed between the shank and the poured concrete, although the hooks provide additional mechanical anchorage.

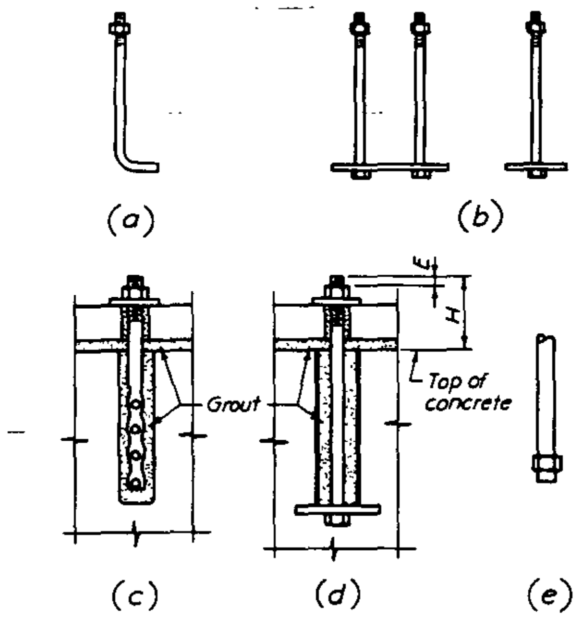


Figure 7-6

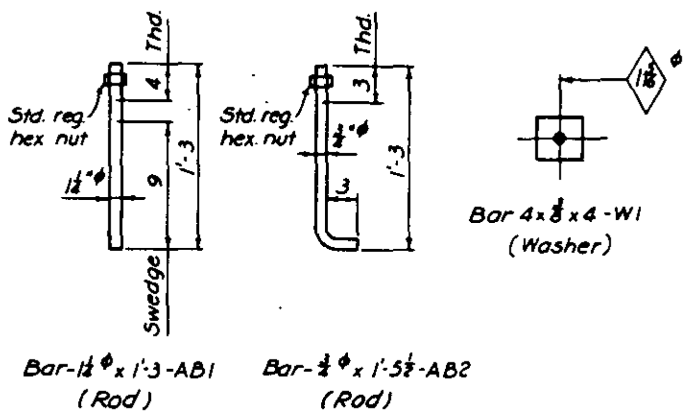


Figure 7-7

Figure 7-6b illustrates single and paired bolts provided with anchor plates to increase resistance through mechanical anchorage.

Figure 7-6c shows a swedged bolt which may be set either prior to pouring concrete, or, as shown, in a drilled hole for subsequent grouting. Uplift is resisted by bonding with the concrete and by the mechanical anchorage of the deformations.

Figure 7-6d shows an anchor bolt set in a metal sleeve. Its advantage lies in the opportunity for some horizontal adjustment at the time the base plate is set in place. The bolt is fixed in place by subsequent grouting.

The nut shown in Fig. 7-6e is generally acceptable in lieu of a bolt head. Since headed rods, in the lengths and diameters required for anchor bolts, generally are not stock items, this substitution relieves the shop of the costly tooling that would be required to form heads on odd lots of various rods.

Figure 7-7 shows typical shop details of anchor bolts. Note that no attempt is made to picture the swedging or to show conventional thread symbols, since the shop will understand what is required by reading the notes. Thread and nut sizes will be ANSI (American National Standards Institute) standard for the rod diameters used. Because of possible inaccuracies in the setting of anchor bolts, the distance H, shown in Fig. 7-6d, should be sufficient to permit the bolt to project a positive distance above the nut. Thread lengths will therefore be somewhat longer than the standard lengths furnished on regular bolts. Washers, which may be either round or square, will have holes which are 1/16-in. larger than the bolt diameter and will be furnished from ASTM A36 steel plates for most applications. Their use is required because of the large bolt holes provided in the base plate and column details.

Manual Part 4, Table I-C, lists a variety of ASTM specification-type material that is suitable for use as anchor bolts and tie rods. Distinction should be made between those items that are available as headed bolts and as rod stock. The headed bolts are generally stocked in lengths up to about 8 in., depending on material specification, and considerable delay and expense can be expected when non-standard sizes and lengths are specified. The designer should review the material availability when either high strength or large diameter are under consideration. Suitable nuts can be selected from ASTM Specification A563.

Table I-B, Manual Part 4, tabulates allowable tension values and Table I-D tabulates allowable shear values for the usually specified materials of ASTM A36, A572 Gr. 50, A588, and A449. It should be noted that these values are based on F_u, the specified minimum tensile strength. Also note that although A572 Gr. 50 and A588 steels both have yield strengths of 50 ksi, they have differing values of F_u:

A572 Gr. 50: $F_y = 50$ ksi, $F_u = 65$ ksi
A588: $F_y = 50$ ksi, $F_u = 70$ ksi

Occasionally it is required that rods or bars be welded to base plates to increase shear or pull-out resistance. The use of a weldable material such as A36 or A572 is recommended for this purpose. Regular deformed-type concrete reinforcing bars such as ASTM A615, A616 and A617 are not produced to a controlled chemistry and their weldability must be very carefully controlled.

COLUMN SPLICES

As the height of a building increases, so does the need for splicing the column sections because of available length or, more often, for economy, because of the change in loading at the different floor levels.

The structural design drawings should furnish sufficient information and detail to indicate the size, number, and arrangement of fasteners or welds and the size and type of splice material required by the loading or by the Specification.

The following Specification requirements are extracted in their entirety and will form the basis for discussion in this section:

1.15.8 Compression Members with Bearing Joints
Where columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient rivets, bolts or welds to hold all parts securely in place.

Where other compression members are finished to bear, the splice material and its riveting, bolting, or welding shall be arranged to hold all parts in line and shall be proportioned for 50% of the computed stress.

All of the foregoing joints shall be proportioned to resist any tension that would be developed by specified lateral forces acting in conjunction with 75% of the calculated dead load stress and no live load.

1.15.12 Field Connections
Rivets, high-strength bolts, or welds shall be used for the following connections:

Column splices in all tier structures 200 ft or more in height.

Column splices in tier structures 100 to 200 ft in height, if the least horizontal dimension is less than 40% of the height.

Column splices in tier structures less than 100 ft in height, if the least horizontal dimension is less than 25% of the height.

Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 ft in height.

In all structures carrying cranes of over 5-ton capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports.

Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress.

Any other connections stipulated on the design plans.

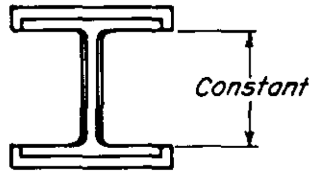
In all other cases, field connections may be made with A307 bolts.

For the purpose of this section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of gable in the case of roofs having a rise of more than 2⅔ in 12. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. Penthouses may be excluded in computing the height of structure.

1.23.7 Compression Joints
Compression joints which depend on contact bearing as part of the splice capacity shall have the bearing surfaces of individual fabricated pieces prepared to a common plane by milling, sawing, or other suitable means.

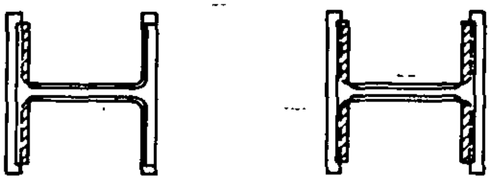
1.25.4 Fit of Column Compression Joints
Lack of contact bearing not exceeding a gap of 1/16-in., regardless of the type of splice used (riveted, bolted, partial-penetration welded), shall be acceptable. If the gap exceeds 1/16-in., but is less than 1/4-in., and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

The cross sections of the W shapes most frequently used as columns are such that, for any given nominal size, the distance between the inner faces of the flanges is constant. As the weight per foot increases in each of the nominal size groups, the column depths and web thicknesses increase. This results in full bearing of the lighter sections when they are centered over the heavier sections. The following illustration shows the relationship which prevails within the listed weight groups:



Column Size	Constant in
W8 × 24 to 67	7.13
W10 × 33 to 112	8.86
W12 × 40 to 336	10.91
W14 × 43 to 730	12.60

Where upper and lower column sections are not centered, or where different nominal sizes must bear on each other, some areas of the smaller section will not be in contact with the larger section. Shaded areas in the following sketches illustrate these conditions:



Stress transfer may be accomplished by providing fills under the flange plates, finished to bear on the larger section, or by interposing a butt plate in the joint on which the upper and lower sections of the column will bear.

The general requirements for the flange plate type of splice are (1) to provide sufficient area of fills in bearing to equal substantially that part of the lighter section which is not in contact, and (2) to furnish the fills with enough fasteners or welds to transmit the bearing stress into the column shaft. When design loads are not given, it will be satisfactory to assume column lengths equal to the story height and to use values from the column safe load tables in Manual Part 3. In either case, the calculated bearing stress f_p must not be greater than the allowable bearing stress F_p. The splices shown in Appendix C have proved satisfactory for column sections differing by up to 2 in. in nominal depth. Joints involving greater differences are special, and details should be shown on the design drawings.

Groove welded butt splices are frequently used in welded construction. Edge preparation is made in the shop, usually for partial penetration bevel or J welds (see Fig. 7-8). Appendix C further details this type of splice and shows the use of butt plates where the upper shaft dimensions result in less than 100% direct bearing. In the absence of flange plates, column shaft alignment and stability during erection is achieved by the addition of lugs for field bolting, as shown in Fig. 7-8 and further developed in Appendix C. These lugs are usually temporary, since architectural requirements often dictate their removal after welding.

Bearing on Finished Surfaces

Specification Sect. 1.15.8 recognizes the complete transfer of direct loads through bearing on finished surfaces. A physical splice connection would seldom be necessary except for safety and stability during erection, since the method of fastening is required merely to hold the parts securely in place. A splice would be necessary when the column can be subjected to considerable stress due to accidental or construction loading prior to placing of the floor system.

Specification Sect. 1.5.1.5.1 permits an allowable bearing pressure of $F_p = 0.90F_y$ ($F_p = 0.90 \times 36 = 32.4$ ksi and $F_p = 0.90 \times 50 = 45$ ksi) on the contact area of a "milled" surface. This value far exceeds the allowable values for axial stresses and will seldom prove critical in the member design. When the columns are of different nominal depths, filler plates and/or bearing plates are used for the load transfer.

Finished filler plates are proportioned to carry bearing loads at F_p allowables and the connections to the column shaft must in turn be designed to carry these calculated loads. The effect of eccentricity should be considered in developing the fastening bolts or welds. Unfinished filler plates should be used when the additional bearing area is not required. The unfinished filler is intended for "pack-out" of thickness and is set back ¼-in. or more from the finished column end. Since it does not transfer any load, it needs only a nominal attachment to the column shaft.

Bearing (butt) plates are used frequently on welded splices where the upper and lower shafts are of different nominal depths and splice plates generally are not used. The bearing plates generally will not be economical on a bolted splice, since the "pack-out" fillers cannot be eliminated and they can be designed to serve as load bearing fillers. Bearing plates usually are selected to be 1½ in. thick for a W8 over W10 splice, and 2 in. for W10 to W12 and W12 to W14. Bearing plates which are subjected to substantial bending

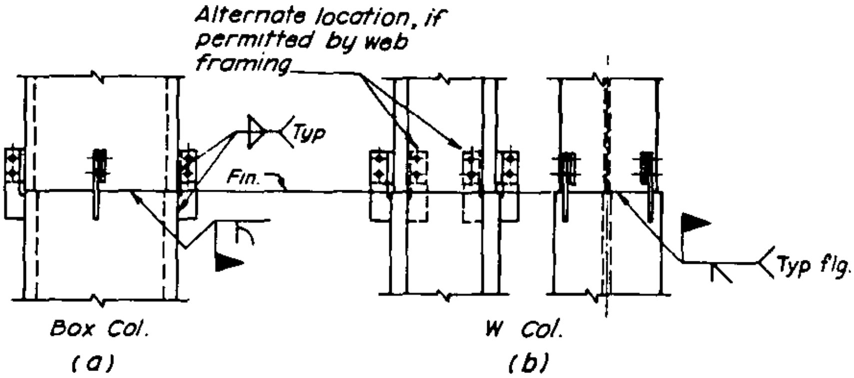


Figure 7-8

stresses, such as required on boxed columns, will require a more careful review and analysis. One method based on extensive experience is to assume a load transfer through the plate on a 45° bevel and then check the thickness obtained for shear and bearing stress.

Specification Sect. 1.21.3 sets forth the finish requirements of bearing plates. In the following, the term *milled* or *milling* is intended to define a surface that has been finished to a true plane by any suitable means:

(a) 2 in. thick and under may be rolled steel without milling, provided a satisfactory contact bearing is obtained.

(b) Over 2 in. to 4 in., inclusive, may be rolled steel that has been straightened by pressing or milling.

(c) Over 4 in. shall be milled for all steel-to-steel bearing surfaces.

(d) Bases of other than rolled steel, e.g., castings, shall be milled for all bearing surfaces.

Since a butt plate fits between the milled column ends, it is important that both surfaces be flat and true.

Lifting Hitches

Under certain conditions, a lifting hitch is used by the erector to facilitate handling columns at the site. Conventional splice plates on H-type columns can be easily modified by adding holes to receive the pin of a lifting hitch. H-type columns spliced by groove welding may be provided with a lifting hole in the web, if permissible, or with holes in auxiliary plates that are bolted or welded so that they do not interfere with field welding. The location of pin holes in splice plates and suggested treatment of H- and box-type columns which have no splice plates are shown in Appendix C.

SHEAR CONNECTIONS

The data given by the designer for shear connections in building work is usually brief, often being confined to the general notes. "Connections as shown in the AISC Manual" are frequently specified, the choice of types being optional with the fabricator. Although some designers may establish minimum connections, it is generally the fabricator's responsibility to provide adequate shear capacity subject to the approval of the customer's engineer. Detailed information, including load tables and examples of commonly used framed, seated, and end plate connections, is given in Manual Part 4. The design of these connections is covered in Chapters 5 and 6.

Some of the various types of shear connections used with columns are shown in Figs. 7-9, 7-10 and 7-11.

Framed and Seated Connections—Bolted

Figure 7-9 shows framed and seated connections for shop bolted-field bolted construction. The conventional two-angle framed connection with the beam web fasteners in double shear is shown in Fig. 7-9a. Although normally furnished as shown, with one angle loose, both angles may be shop bolted to the column flange. Since the latter detail is a "knifed" connection, some erection clearance must be provided between the angles, and the beam must be coped to allow its erection by "knifing" or lowering the beam into place from above. In either case, play in the open holes through the angles and beam furnishes adjustment to compensate for the mill variation usually present in columns. Although shop attachment of both angles to the beam web will permit framing the beam to either the column flange or the web, this detail is not generally recom-

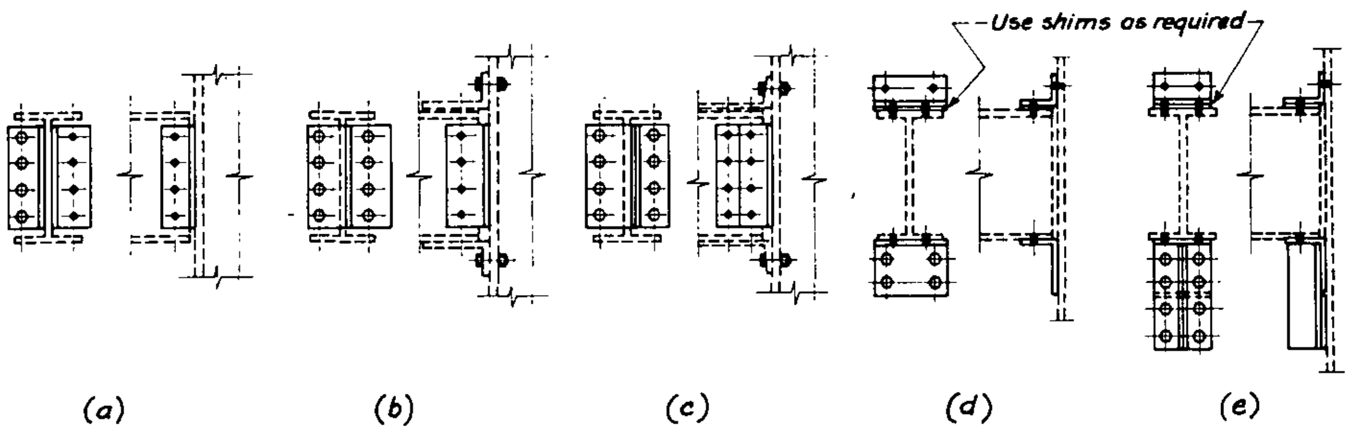


Figure 7-9

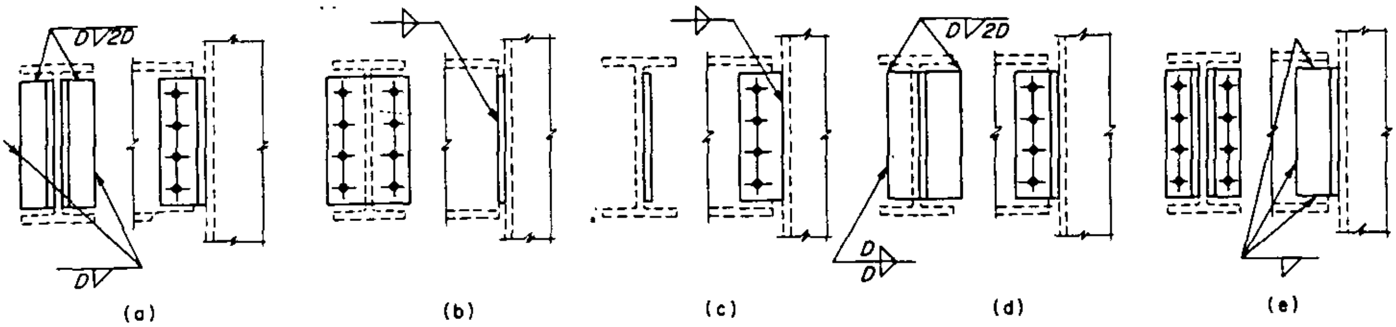


Figure 7-10

mended. The need to tilt beams from end to end in making column web connections requires shortened overall beam dimensions and use of fills at the column web to build out the difference. Since there are no open holes in the beam web to provide adjustment in length for possible mill variation in the columns, it may be necessary to shorten the beams and furnish shims for both column flange and web connections.

Figures 7-9b and 7-9c illustrate "wrapped" connections. They are framed connections utilizing tees or angles with the beam web fasteners in single shear. By either arrangement it is possible to use shop bolts for all fasteners through the column, and at the same time permit side erection of the beam. Since the web fasteners are in single shear, an additional row of web fasteners may be required. When shop attached top and bottom moment connections (shown dashed) are present, a one-sided connection (Figs. 7-9b and 7-9c) or a framed connection with one angle loose (Fig. 7-9a) is necessary.

The framing angles shown in Figs. 7-9a, 7-9b and 7-9c are not adapted to connecting beams to column webs, since an impact wrench cannot be used to tighten the bolts through the beam webs because of interference by the column flanges. It is therefore customary to employ the seated connections shown in Figs. 7-9d and 7-9e. The unstiffened seat in Fig. 7-9d is preferred if available capacities are great enough to support the beam reaction. The stiffened seat, shown in Fig. 7-9e, can be made as strong as necessary. Either of these connections can be used on column flanges, although the projection of stiffeners through architectural finish or column fireproofing may limit the choice to unstiffened seats.

Framed Connections—Shop Welded, Field Bolted

Beam connections suitable for shop welded-field bolted construction are shown in Fig. 7-10. Paired angles, shop welded to the column flange, are shown in Fig.

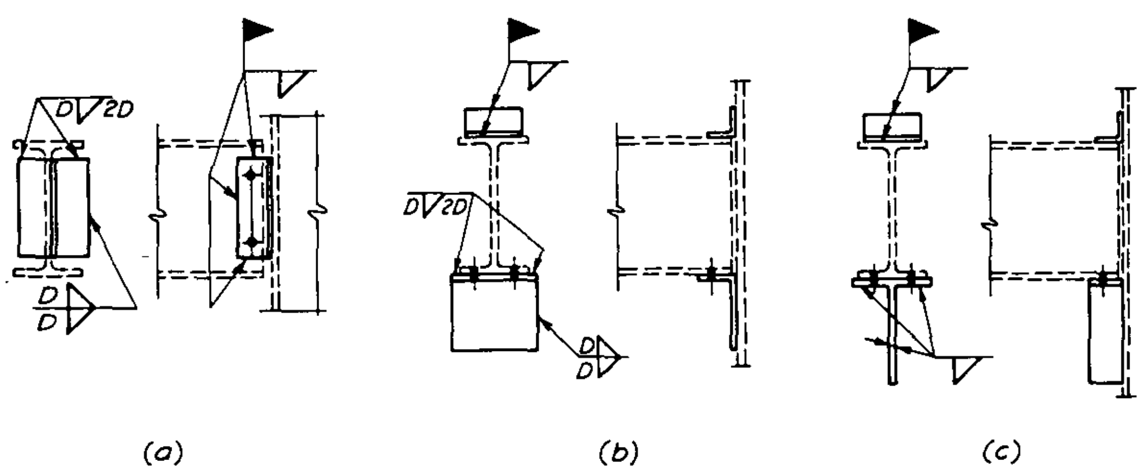


Figure 7-11

7-10a. Note that this is a "knifed" connection which requires a beam web erection clearance and a bottom flange cope. When this latter connection, or any other type of "knifed" connection, is used in conjunction with high-strength bolts, provision must be made to insure contact between faying surfaces. This is usually accomplished by furnishing $1/16$-in. thick shims for use by the erector if the measured clearance exceeds $1/8$-in. Fig. 7-10b shows an end plate welded to the beam. Its advantage of simplicity is somewhat offset by the close tolerance needed to achieve accurate beam length and square ends. The use of this connection for several beams in a continuous run may require a small reduction of beam length and provision of shims to compensate for overrun or underrun in column dimensions.

Figure 7-10c shows another very simple connection that utilizes a plate shop welded to the column flange. It is limited in shear capacity, but may be used when the connections are designed in accordance with the procedure outlined in Appendix B. Figure 7-10d illustrates the use of a tee, shop welded to the column, with holes for field bolts. The flange thickness of tees used for this purpose should be held to a minimum to permit the flexure necessary to the end rotation of the beam. Figure 7-10e shows the connection angles shop welded to the beam web and field bolted to the column flange. This type connection has the same disadvantage of required close tolerances as the end plate connection in Fig. 7-10b.

Seated Connections—Shop Welded and Field Bolted

Shop welded-field bolted seated connections are similar to the all-welded seated connections in Figs. 7-11b and 7-11c, except that the top angles are bolted as in Figs. 7-9d and 7-9e.

Framed and Seated Connections—Welded

All-welded framed connections are readily adapted from the four types shown in Fig. 7-10. Figure 7-11a shows a tee connection comparable to the one in Fig. 7-10d; the other connections of Fig. 7-10 can be converted in a similar manner. The seated connections in Figs. 7-11b and 7-11c can be used either as flange or web connections, their application being subject to the same restrictions noted for the bolted ones. The stiffened seat, shown as a structural tee, may be replaced by one built up of plates. The holes shown are for erection bolts. Erection bolts that are used to fasten beams to seats may remain. Sometimes they may also serve as permanent attachment, providing their use is permitted by AISC Specification Sect. 1.15.12.

Framed and Seated Connections—Field Clearances

Figures 7-9, 7-14 and 7-15 show clearances between the top connection material and the top of beam. This clearance is necessary to allow for possible mill variation from the published beam depths. Three typical situations and the clearances usually provided are shown in Fig. 7-12. The range of clearances given reflects the shop practice of some fabricators.

Figure 7-12a shows a seated connection where the top angle is field bolted and no beam web connection is present. The recommended $1/8$- to $1/4$-in. clearance applies to connections in Figs. 7-9d, 7-9e and 7-14b. This detail is generally used on column webs.

The $1/4$- to $3/8$-in. clearance shown in Fig. 7-12b is suggested when the top connection is either shop bolted or shop welded. This detail may be used on column flanges.

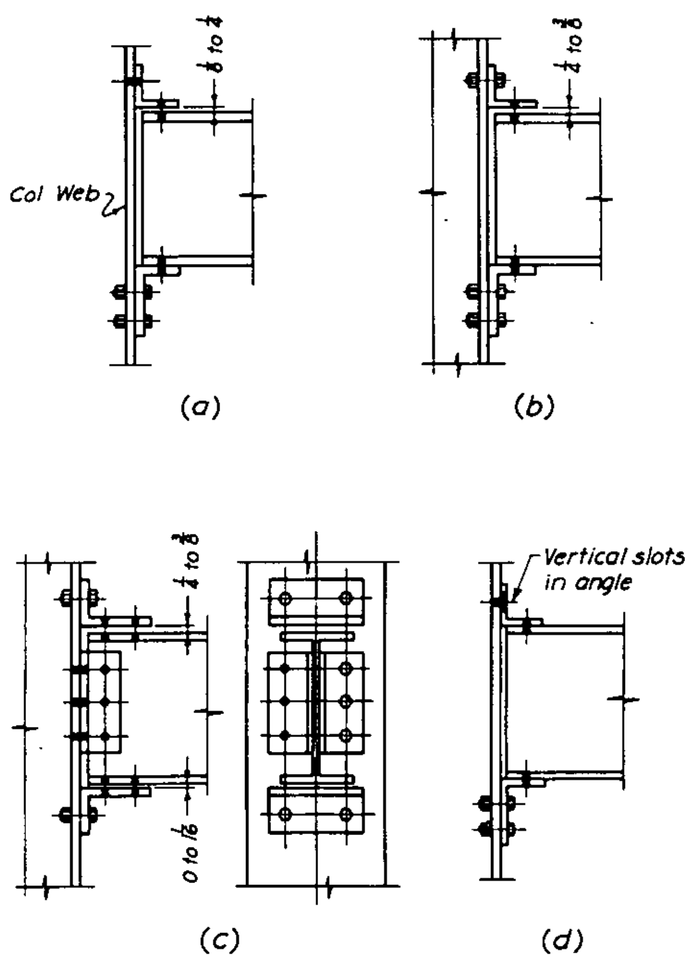


Figure 7-12

Providing clearance at the bottom as well as at the top, as shown in Fig. 7-12c, is optional when a beam web connection is included with shop attached top and seat material. This arrangement may apply to connections such as those shown in Figs. 7-9b, 7-9c, 7-14a, 7-15a and 7-15b.

In each of these cases, since beam depth may underrun as well as overrun, some fabricators supply shims for about twice the opening expected under the top angle. Others supply shims for openings as detailed and furnish additional shims only as required.

Where field bolts are permitted in fastening beams to seated connections, the need for clearances and shims can be eliminated by providing vertical slots in the top connection angle. Figure 7-12d shows such slots located with their centers matching the holes in the column. This will accommodate both overrun and underrun in the beam depth and is the preferred method of many fabricators and erectors.

When beam web connections are placed on column flanges with one angle bolted, it is recommended that the spread between outstanding legs equal the decimal beam web thickness plus a clearance which will produce an opening to the next higher ⅛-in. This rule is illustrated in Fig. 7-13a. Note that angle gages will occur in minimum increments of 1/16-in.

In the event both angles are shop attached to the column, forming a "knifed" connection, provision should be made for an erection clearance of about ⅛-in. The application of this rule is shown in Fig. 7-13b. When similar fittings for a "knifed" connection are welded to a column, and angle gages are not a factor, it is satisfactory to provide an opening equal to the fractional beam web thickness plus ⅛-in. When these connections are to be made with high strength bolts, shims must be furnished wherever measured clearances exceed ⅛-in.

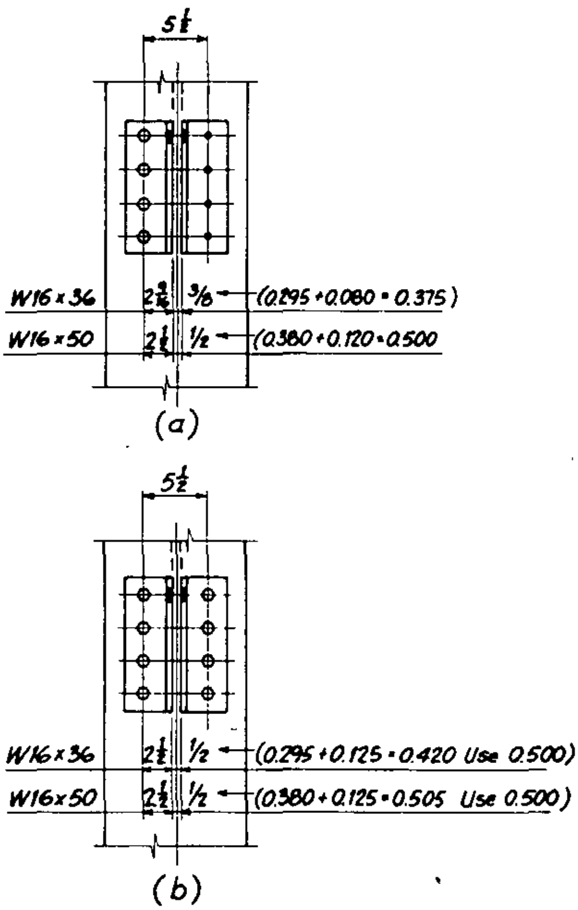


Figure 7-13

Erection Seats

When beams or girders frame opposite each other and take the same open holes in the web of a beam, girder or column, as in Fig. 5-17 or Fig. 5-19, consideration must be given to the problem of supporting one member while erecting the other to its final position. When filler beams frame opposite in the web of a header beam of about the same depth (up to about 4 in. difference), the erector may use blocking to support one beam off the bottom flange until drift pins and bolts can be entered through the common holes. An erection seat, usually an angle, is sometimes provided in deeper beam sections and in column webs. This erection seat is normally detailed to clear the bottom flange of the member it is to support by ⅛- to ¼-in. to accommodate beam and location tolerances. If the erection sequence is known, this erection seat is provided on the side needing the support. If the erection sequence is not known or is doubtful, the seat can be provided on both sides of the supporting web. The erection seat should be sized and attached with sufficient bolts or welds to support the dead weight of the member unless additional loading is indicated.

Erection seats frequently are shipped loose and in limited quantities. It is expected that the field erector will weld, clamp or bolt these seats as dictated by his erection scheme. It also is assumed that seats are available for reuse in more than one location. These seats may be punched for bolting or may be welded.

The erection seats generally are not required to be removed unless they create an interference or detract from the architectural appearance.

The makeshift practice of "hanging" the filler beam on an untightened bolt or a drift pin is dangerous; it violates safety codes and should not be permitted by erectors.

MOMENT CONNECTIONS

In modern tier building construction, most beams and girders framing to columns are designed to resist bending moments resulting from lateral forces due to wind or earthquake loadings. Since these loads are in addition to normal gravity floor loads, the beam connections are proportioned for moment as well as shear forces. These connections are usually indicated on the plans by a system of numbers or symbols, keyed to schedules and design sketches which give the designer's requirements. Bolted moment connections are made with angles or tees which transmit the moment from the beams to the columns. The vertical shear is transmitted by angle or tee connections on the beam web, or by a stiffened or unstiffened seat. Welded moment connections may employ plates to transmit both shear and moment or, where a rigid or continuous frame design is called for, the beams may be welded directly to the column.

The designer is expected to show complete data and detail sketches of desired connections where "rigid frame" or "semi-rigid framing" is to be used in the design. The design of moment connections is ordinarily not a part of the detailer's work. An example of the method of designing a Type 1* (rigid frame) connection is given under "Moment Connections—Shop Welded-Field Bolted" and "Moment Connections—End Plate," both in Manual Part 4. A Type 3* (semi-rigid framing) connection is analyzed under "Moment Connections—Welded," also in Manual Part 4. These and other types of moment connections are illustrated in Manual Part 4 under "Suggested Details" and in Figs. 7-14, 7-15 and 7-16 of this chapter.

See AISC Specification Sect. 1.2.

Shop bolted-field bolted moment connections are shown in Fig. 7-14. Depending on column material thickness, these connections may require stiffening diaphragms to back up the horizontal tees. Paired angles may be used, instead of the tees shown, for both framed and seated shear connections. When less moment resistance is required, single angles or parts of channels can be used instead of tees in connecting beam flanges to columns. It should be noted that prying action on fasteners, discussed in Manual Part 4 and in the AISC publication *Engineering for Steel Construction*, must be considered on the tension fasteners attaching the tees to the column.

Shop welded-field bolted moment connections are shown in Fig. 7-15a connecting to a column flange and in Fig. 7-15b connecting to a column web. Since the moment is resisted by the top and bottom plates, deflection of the beam at the connection is minimized, so that provision for end rotation of the beam is not necessary. The web connection in Fig. 7-15b is so arranged that all fasteners are accessible for impact wrench tightening. Figure 7-15c shows an end plate capable of transmitting both shear and moment. Note that prying action must be considered on fasteners in tension. Advantages and limitations of this connection are the same as cited for the end plate shear connection of Fig. 7-10b.

All-welded moment connections are generally less complex than their counterparts in bolted work, but fabrication may be more exacting. Figures 7-16a and 7-16b show two ways of making a column flange connection. The method in Fig. 7-16a requires a close joint between beam web and column flange, which may dictate finishing the beam length to close tolerances. In Fig. 7-16b this problem is eliminated, but at the cost of more welding and an additional plate for the shear connection. On lighter column sections, either design

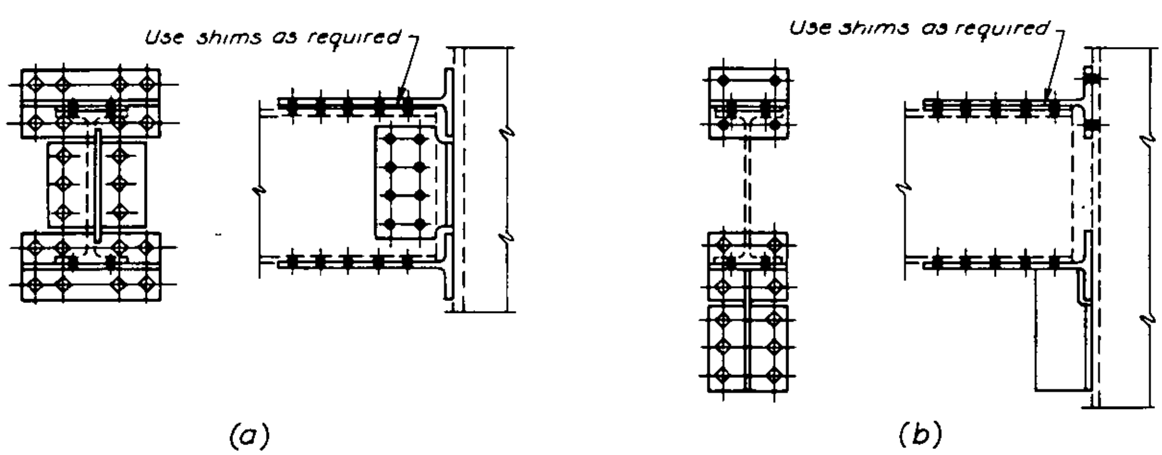


Figure 7-14

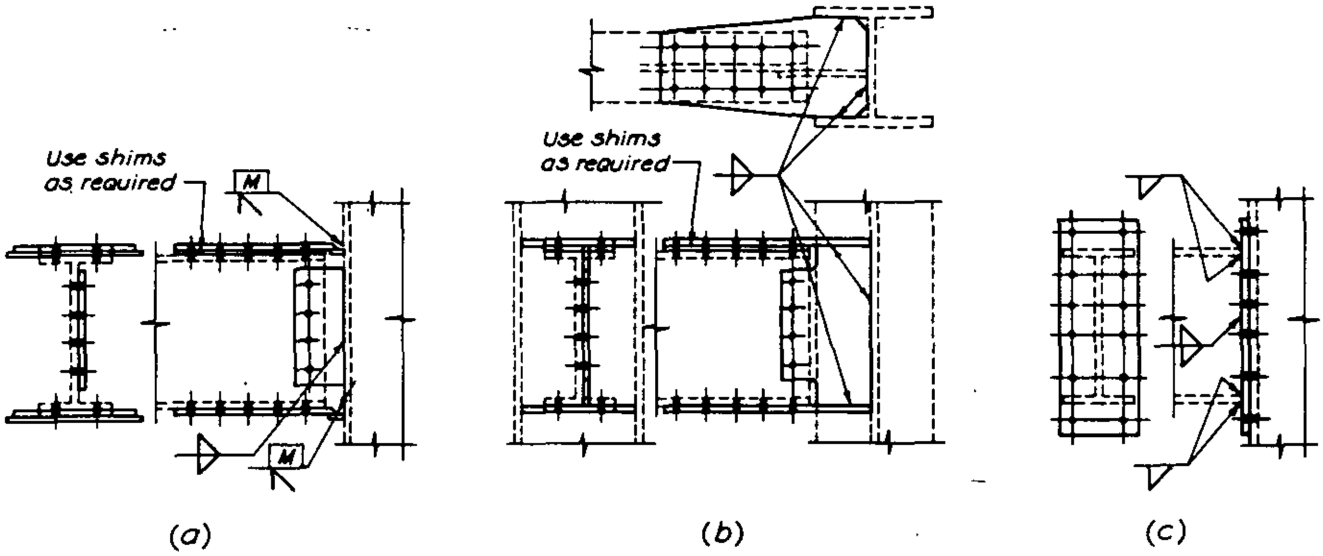


Figure 7-15

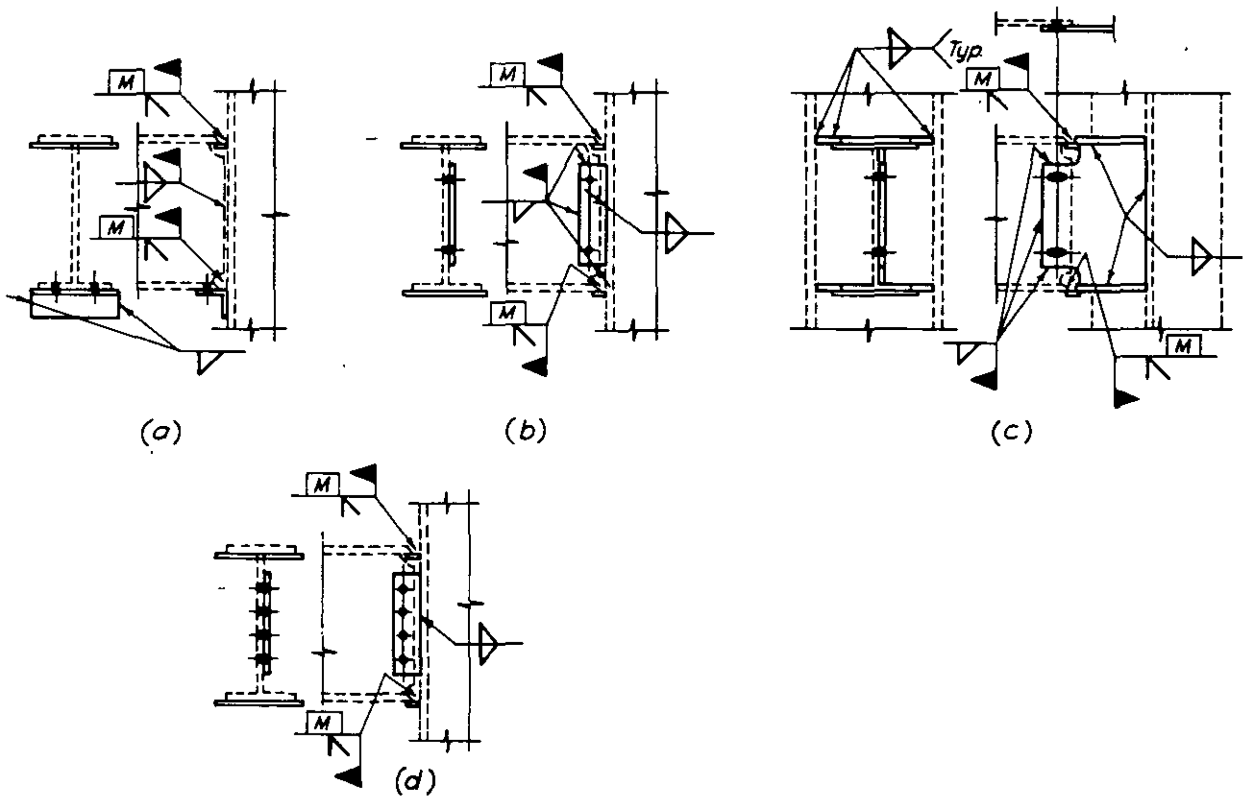


Figure 7-16

may require stiffener plates between inside faces of the column flanges. These plates distribute the moment stresses from the connections to the column web or to a similar connection on the opposite column face.

Although it might seem that the details in Figs. 7-16a and 7-16b could be used for a column web connection, the limited space between column flanges makes welding difficult. The connection pictured in Fig. 7-16c is frequently used for web connections. Note that this is in effect a beam stub, extending just beyond the column flange so that field welding can be performed easily. Groove welds and backing bars are used to obtain complete penetration welds in the beam flange joints. The shear joints are made with fillet welds.

The detailer is cautioned that connections to H-type column flanges, as shown in Figs. 7-15a, 7-16a and 7-16b, may not perform well on coverplated columns and, if so shown on a design, should be questioned. The force exerted by whichever of the moment plates is in tension will tend to separate the cover plate from the column flange.

In detailing beams connected as shown in Fig. 7-16, it may be necessary to compensate for field weld shrinkage. Experience has shown that this is about 1/16-in., perpendicular to the weld throat, for the typical groove weld. Shrinkage of this kind can cause problems in erection, particularly where several beams and columns occur in a continuous run. Corrections can be made by detailing beams with 1/8-in. added to the center-to-center dimension of end erection bolt holes, or by providing adjustment slots for erection bolts in the column fittings (see Fig. 7-16c).

These all-welded connections approach full rigidity and are generally limited to Type 1 "rigid frame" construction. Their use in structures designed as "conventional" (Type 2) or "semi-rigid" framing (Type 3) should not be undertaken without express approval from the designer.

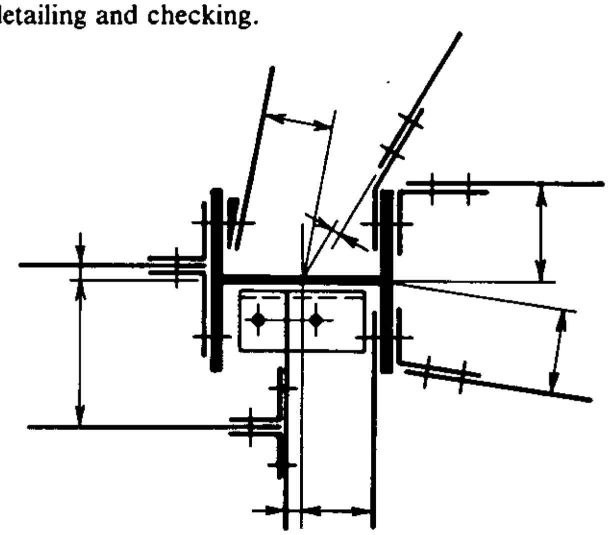
OFFSET AND SKEWED CONNECTIONS

Special care is required when developing connections for framing that is not centered on column web or flange faces, or is skewed with respect to the column. Design drawings show such offset or skewed beams by dimensions referred to column centers, but seldom show details of connecting such framing.

Small offsets in bolted construction can be handled readily by shifting holes in seat angles, by altering gages in framing angles, or by otherwise modifying regular connection material. Larger offsets require special treatment, using such details as one-sided connections, gusset plates or built-up brackets. Consideration should be given to the eccentric loading of fastener groups resulting from these offset connections. Small eccentricities in double angle or seated connections may be neglected. Where appreciable eccentricities occur, and for all one-sided framed connections, fastener capacities must be investigated. This can be done by using the coefficients given in "Eccentric Connections—One-Sided Connections" in Manual Part 4. Skewed, sloped and canted connections that present similar problems are discussed in the AISC publication *Engineering for Steel Construction*, which contains an extended discussion on eccentric framing conditions, including examples.

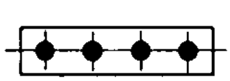
Sometimes beam webs can be connected directly to the inner or outer faces of column flanges, using intervening fills if required. Where a beam cannot be connected directly to the column, indirect support may be possible through connection to a second beam, which then transmits both reactions to the column.

Structural framing can connect to a column in a variety of ways. No general rule can be cited for all cases. The best connection is usually the one requiring the least number of fasteners or welds and the least amount of connection material. The illustration below diagrams a few of the many ways non-concentric and non-rectangular framing may be supported by columns. Such connections are generally worked out on preliminary drawings by an experienced detailer in advance of the detailing and checking.

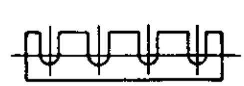


SHIMS

The shims furnished to the erector for use in filling out the spaces allowed for field clearance at column splices and shear or moment type beam connections may be either the conventional kind, with round punched holes, or the "finger" type with slots cut through to the edge.



Conventional



Finger

Whereas the conventional type shim is less expensive to fabricate, the finger type has the advantage of lateral insertion without the need to remove erection bolts or pins already in place. It should be noted that some fabricators prefer that shims 3/16-in. and under be listed on drawings and order bills as "strip" or "sheet," depending on the width of material,* thus:

240 Strip—4½ × 1/16 × 1'-6 SH1
120 Sheet—12½ × 1/8 × 2'-2 SH2

If finger shims are fully inserted against the bolt shank, they are acceptable for friction-type connections and are not to be considered as an internal ply. In such cases the allowable bolt load may be taken as though the shims were not present. The reason is that less than 25% of the contact area is lost—not enough to affect the performance of the joint.

SHOP DRAWINGS OF COLUMNS

Drawing Arrangement

Shop details of columns may be shown either in an upright position with the bottom of the shaft at the bottom of the sheet, as shown in Fig. 7-22, or in a horizontal position with the bottom of the shaft to the left, as shown in Figs. 7-20 and 7-21. The detailer is guided in this choice by the amount of space needed to produce a clear and uncrowded detail. More columns can be shown on a sheet when they are detailed upright, but where complicated fittings, bracing connections and sectional views are required, horizontal placement is advantageous. The space between views must be sufficient to contain all dimensioning as well as all details of gusset plates or brackets that may extend from the column shafts. The column details shown elsewhere in this chapter will give an idea of the location and spacing of views. Many fabricators maintain files of typical drawings which may be consulted for the approved treatment of specific problems.

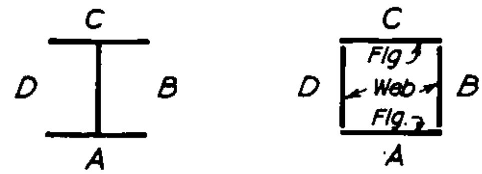
In addition to the title block, preprinted drawing forms frequently include shop bills for listing the material required (see Fig. 7-20). These usually appear either at the right side of the sheet above the title block, or across the bottom. Separate shop bill forms, preferred by some fabricators, allow use of the entire sheet area for detail sketches.

General notes, applying to the entire sheet, are placed in the lower right corner of the sheet, near the title block. Special notes may appear anywhere on the sheet, preferably near the detail concerned.

*For widths and thicknesses corresponding to strip and sheet designations, see AISC Manual Part 6 under "AISI Standard Nomenclature for Flat Rolled Carbon Steel."

Column Faces

The practice of assigning a letter to each of the four faces of a column is generally followed in tier building work (see Fig. 7-20). This identification by letter is particularly helpful to the shop in laying out the work, and reduces the probability of shop errors. Looking down on top of the column, the lettering progresses alphabetically in a counterclockwise direction around the shaft, thus:



In the case of H-type columns, faces **A** and **C** are always flange faces and faces **B** and **D** are always web faces. The same system is used on box columns, the flange-web relationship being as shown in the diagram above. The views of faces follow the same alphabetical sequence on the drawing, beginning with **A** and reading from left to right for columns shown upright, and from top to bottom for those shown horizontally. It is seldom necessary to show a separate view of face **D** for H-type columns, as any fittings on face **D** which differ from those on face **B** can be shown by dashed (invisible) lines. The punching or drilling, is, of course, common to both faces. The letter **D** on a combined **B** and **D** face view is shown dashed (invisible) to denote the far side of the web. For a box section, if a separate view of face **D** is required, it follows face **C** in sequence. Many fabricators do not show the letter **D** on the column detail except in the case of box columns, since the far side of the **B** face of an H-type column is obviously the **D** face. If this system of identification is used, material on the web face is noted "N.S." or "F.S.," indicating near side or far side, instead of noting it for face **B** or **D**.

The designation of face **A**, as it appears on the framing plan, is optional with the detailer. Since a view of face **A** is usually drawn to show splice details, it is best either to select the flange face which contains the most detail (fittings and fabrication), labeling it **A**, or to arrange the marking sequence to produce a face **B** web view that will contain the most detail fittings and will be shown as a near side view with full lines.

Faces which require no detail fittings or fabrication of any kind need not be shown. However, their presence should be indicated by a center line, in the proper relation to the other faces, labeled with a note such as "Face **C** Plain." In the event that all material and fabrication on face **C** is identical with **A**, it may be combined with face **A** by adding a note such as "Face **C** same as Face **A**" along the center line of face **C**.

Sections

Transverse sections taken through a column shaft are always projected from the web view and are shown looking toward the bottom of the column (see Figs. 7-18b and 7-18c). When such sections must be displaced from the **B** face center line, as shown in Fig. 7-18c, they should retain this same orientation. Unless a box or other section which requires dimensioning is involved, the main material outlines are seldom shown in transverse sectional views. When such sections are taken at floor levels, the cutting plane symbol is generally omitted and the section is labeled with the floor number. Sections showing isolated details, or the inner web of a box section, are provided with cutting plane symbols and section identification.

Combined Details

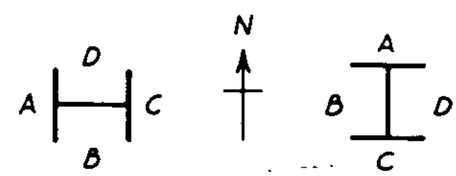
Columns in the same or different tiers, which are alike except for minor differences, are often shown on the same sketch. While this is a generally accepted practice, the detailer is cautioned against combining details for too many columns on one sketch, regardless of how minor the differences may seem to be. The large number of notes that may be necessary can render the drawing difficult to read, and cause shop errors. If exceptions cannot be shown without undue complications, it is better to detail the columns on different drawings. To reduce the time and effort such repetition requires, consideration should be given to making reproductions of the original sheets and altering the copies as necessary.

Column Marking

Columns, like other framing, must be given shipping marks. The mark appearing on the drawing is painted on the column shaft for identification in the shop and at the building site. As pointed out in Chapter 3, fabricators differ in the marking systems they favor. The detailer must use the standards adopted by his employer. However, a practice that is fairly universal for tier building work includes the use of the column location shown on the design plans as part of the shipping mark. Since the same mark is carried vertically through all succeeding sections of any one column, it is necessary to distinguish each section by a tier mark. This may be done either by adding, after the column number, the note **1st Tier**, **2nd Tier**, **3rd Tier**, etc., or by adding the floor numbers through which each column section passes. Thus, the marks of the several sections of column **D4**, shown in the column schedule in Fig. 7-19, may be shown either as **D4-1st Tier**, **D4-2nd Tier** and **D4-3rd Tier**, or as **D4(0-2)**, **D4(2-4)** and **D4(4-R)**.

Although tier building columns are assigned different marks, a number of columns, particularly those wi a given tier, may be identical. When this occurs, t column marks appearing on the sketch are bracketed and noted: "Alike" or "Alike except for marks." This relieves the shop of searching the drawing for differences which are not present.

Since the direction a column must face in a structure may not be readily apparent to the erector, particularly in tier building work, it is necessary to provide a compass or direction mark on one of the column faces. To accomplish this, a note such as: "Face **A** North" on the drawing following the column mark instructs the shop to paint "North" on the **A** face. When the axis of a structure varies so widely from a North-South line that there is some question as to the actual placement of a column, the erector will use the "called" North direction as shown by the compass arrow on the erection plan. The draftsman is cautioned to observe the orientation of each column when applying compass marks. As shown below, even identical columns will require different compass marks if their webs are not parallel.



Face A West *Face A North*

Right and Left Hand Details

The structural steel detailer will have frequent reference to the concept of detail and/or main shipping pieces that have been or could be drawn and billed as "right hand" or "left hand" (mirror image) pieces. This is particularly true of old existing drawings where the joining was made with rivets and the material preparation was almost always controlled by a template. Most fabricators are today restricting the use of this short-cut concept because, while the cost of the detail drawing preparation may be somewhat reduced, the opportunity for shop errors is considerably increased. The increased application of computer graphics is also a deterrent since most computer programs, while readily able to create a mirror image, are not able to handle the complexities of combination detailing.

The detailer must visualize the concept of right hand/left hand details in order to avoid inadvertent errors of similarity. The following brief comments will illustrate the general principles. The student who requires a more thorough coverage is referred to any standard textboc on technical drafting.

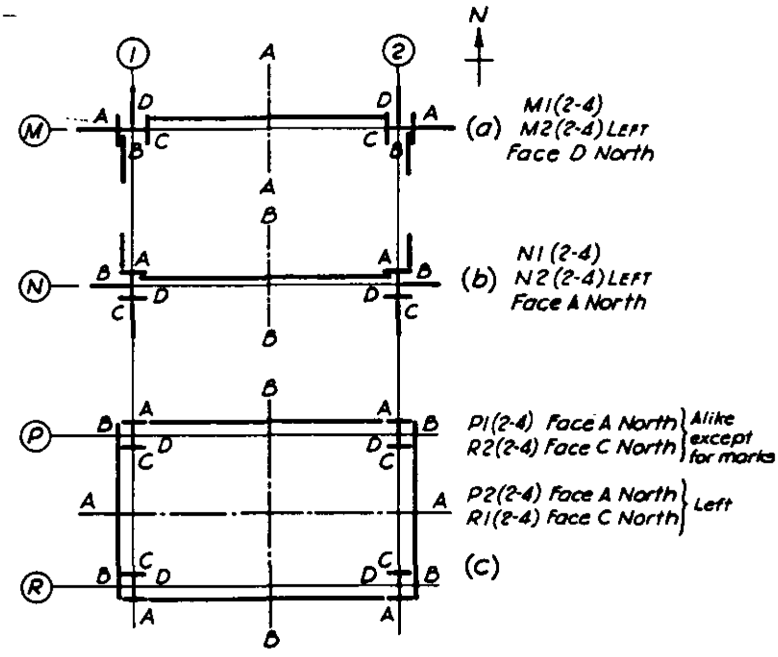


Figure 7-17

The conditions which give rise to right and left hand columns are due solely to the arrangement of framing in the plan view. The presence of a right and left situation depends on the symmetry of the column framing with respect to assumed vertical planes. In Fig. 7-17a, the framing to columns **M1** and **M2** is symmetrical about vertical plane A-A, which is also perpendicular to the column webs. If the corresponding framing at each column requires the same connections and has the same relation to floor levels, these two columns will be rights and lefts.

Likewise, the framing at columns **N1** and **N2** in Fig. 7-17b is symmetrical with respect to vertical plane B-B, which in this case is parallel to the column webs. Other things being equal, these columns will also be rights and lefts. Rights and lefts involving more than a pair of columns are possible, as shown in Fig. 7-17c. Here there are two axes of symmetry, shown by planes A-A and B-B. This relationship can occur between adjacent columns or at four widely separated corners.

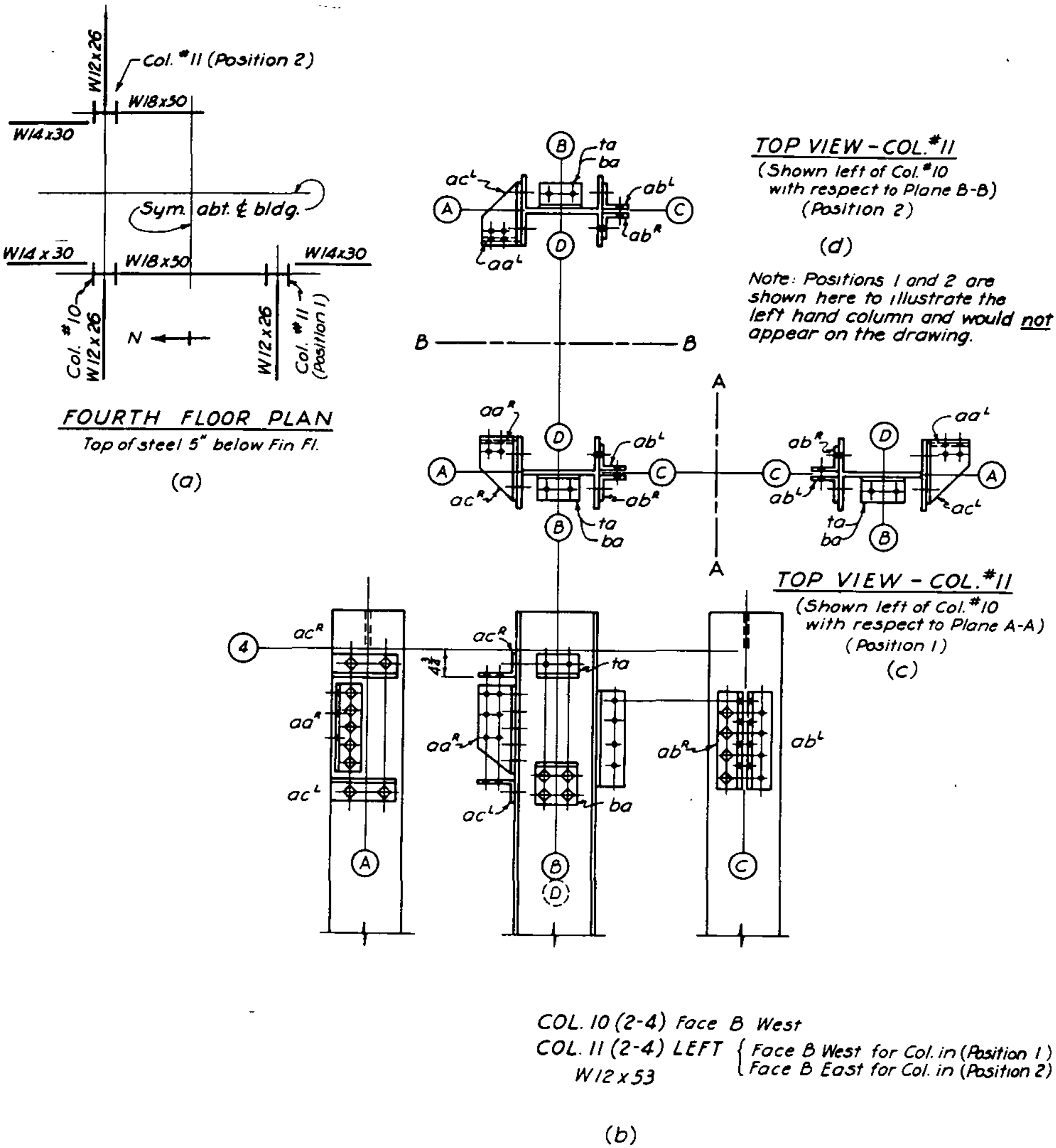
The reversal of the normal facing sequence, which must occur for a left or opposite hand column, is illustrated in Fig. 7-17. The right hand columns are lettered counterclockwise in the usual way. Face **A** on each left or opposite hand column is assigned to the flange which is the left of face **A** on the corresponding right hand column. The face lettering then proceeds in a clockwise direction.

Direction marks on left hand columns may be affected by the position they take with respect to the right hand columns. Figures 7-17a and 7-17b illustrate cases where either web or flange faces carry the same letter pointing North (or South). One note can do for each pair of columns. Figure 7-17c shows the notes that are required where pairs of identical rights and corresponding lefts, because of orientation, must have different direction marks.

As the detailer gets more experience in working with existing structures, he will be exposed to the following variations of nomenclature:

Right-hand	Left-hand
Right	Left
Thus	Reverse
As-shown	Opposite-hand

Details on Right and Left Columns—Unsymmetrical detail fittings, i.e., those for which a left hand sketch can be drawn, are identified on the right hand shipping piece by the superscript letter **R** appended to the identifying mark. In Fig. 7-18b, angle **aa**^{R} on the flange (**A** face) of column **10(2-4)** illustrates this condition. The views in Figs. 7-18c and 7-18d, representing sections of left hand column **11(2-4)**, in either of two possible positions (Position 1 or Position 2), show this same angle as a left, labeled **aa**^{L}.



W12x26
Col. #11 (Position 2)
W18x50
W14x30
Sym. abt. ℄ bldg.
W14 x 30
W18x50
W14x30
Col. #10
W12x26
N
W12x26
Col. #11 (Position 1)
FOURTH FLOOR PLAN
Top of steel 5" below Fin Fl.
(a)
TOP VIEW - COL. #11
(Shown left of Col. #10 with respect to Plane B-B)
(Position 2)
(d)
Note: Positions 1 and 2 are shown here to illustrate the left hand column and would not appear on the drawing.
TOP VIEW - COL. #11
(Shown left of Col. #10 with respect to Plane A-A)
(Position 1)
(c)
COL. 10 (2-4) Face B West
COL. 11 (2-4) LEFT
W12x53
Face B West for Col. in (Position 1)
Face B East for Col. in (Position 2)
(b)

Figure 7-18

When two fittings on the same shipping piece are right and left with respect to each other, they will appear on that piece labeled **R** and **L**. Which of the two is to be marked **R** and which **L** is of no consequence. The choice is optional with the detailer. Corresponding fittings on the left hand shipping piece are also identified as **R** and **L**, but their location will be such that lefts will oppose rights and rights will oppose lefts when compared with the original right hand shipping piece. This relationship is shown by angles $\mathbf{ab^R}$, $\mathbf{ab^L}$ and $\mathbf{ac^R}$, $\mathbf{ac^L}$ in Fig. 7-18. Note that the symmetry in plan between columns **10** and **11** exists with reference to vertical plane A-A for the Position 1 relationship, or plane B-B for the Position 2 relationship.

Angles **ta** and **ba**, being symmetrical pieces, simply appear in their left hand locations on the left hand piece and no **R** and **L** marking is required.

The right and left concept described for fittings extends to every aspect of column fabrication, including all fasteners, welds, copes, cuts and patterns of punched holes for which a left hand arrangement is possible.

A convention has developed around the "handing" of columns, as well as other fabricated pieces. When the shipping pieces are identical in all respects except for being of the other hand, they are referred to as "Rights" and "Lefts" or "Thus" and "Reverse." When there are differences of punching or in detail material, the "As-shown" and "Opposite-hand" nomenclature is frequently used; this method is described in Appendix D.

These various nomenclatures are not to be considered as so-called "industry standards", because the practice varies widely.

Drafting Economy

As mentioned earlier, considerable economy in drafting time can be achieved in detailing columns by combining sketches of identical or similar columns.

Shop and drawing room time can also be saved by the use of standardized connection material. Some fabricators have developed standard beam connections, based on the framed and seated connections shown in the AISC Manual, which can be used on columns with a minimum of dimensioning. Such a system may include coded marks which describe each connection completely. Standard templates could be stocked for immediate use.

A similar practice with even greater potential savings is the employment of job standards. These cover connection material of all descriptions, that repeats throughout a particular structure. Complete details and material billing, with assigned standard assembly marks, are developed and drawn on separate sheets. By this means typical fittings, from simple splice plates to complex moment connections, may be copied quickly onto shop drawings, using only those dimensions required to locate the connection and fabricate the main material. Job standard sheets are used in the template shop to produce the templates keyed to them by standard marks on the details.

Further economy in detailing can be achieved by referring the fabrication or fittings required at one location on a column to a previously completed view elsewhere on the same or a different column. Identical details may be referred from floor to floor by a note such as "5th Floor, same as 4th Floor" in which case the note will replace the omitted detail. Repeated details may also be referred from face to face on the same column, or from column to column on the same sheet. Covering notes replacing such omitted details may read: "Parts not shown same as face **A**, Col. **B4**" or "Remainder of face **C**, same as face **C**, Col. **B20**." In all such transfers, connections are understood to be located identically with respect to floor lines.

Although the practices described here are current in most drafting rooms, the detailer is cautioned not to use cross-noting to the point where the drawing becomes a puzzle to the shop. He should definitely guard against using notes which refer to other notes. This can cause confusion, particularly if right and left details are concerned. Even greater confusion may ensue if it becomes necessary to revise a sketch which is repeated elsewhere by note, when the revision applies only to the original sketch. It is recommended that the detailer limit the use of short cuts of this nature to those which are sanctioned by past practice. These can be found by reference to his employer's example and reference drawings.

Another possible time and work saver, which may benefit both the shop and the drafting room, is "subassembly detailing." This system presupposes a number of members (columns, girders, trusses, etc.) which have identical main material but which differ in some degree as to detail fittings or other minor fabrication. Subassembly details are prepared showing only the fabrication which repeats exactly on all members. This may include the assembly of trusses, the welding of box or girder sections, drilling or punching of all holes which repeat, the attachment of splices and other fittings, and any other work which is common to all members. Subsequent to this, final details are prepared, usually on reproductions of the original subassembly drawing, which complete all the work, including addition of fittings, etc., for each different piece.

The advantage of this procedure is that the shop can complete the bulk of the work on a run of identical subassemblies more efficiently than would be the case if each piece was worked individually, perhaps from separate drawings. Partially completed work may then be stockpiled until needed for final fabrication and shipment. Of course, this system should be used only after consultation with the shop and with the shop's full concurrence.

Column Details—Bolted Construction

Because much of the work required to fabricate the framing for a tier building is called for on the column details, careful planning is necessary to insure legibility of the completed drawing. An experienced detailer will clearly visualize the connections and then allow sufficient room for all necessary views and dimensions. Preplanning will result in a clear, uncrowded detail drawing. The beginner may find it desirable to make a few freehand sketches to consolidate all information and to visualize the relationships that exist between the framing on the several column faces at any level. With the problems thus pictured, space requirements can be foreseen, as well as possible points of interference that must be investigated as the detailing proceeds.

Examples of column shop details are given in Figs. 7-20, 7-21 and 7-22. These drawings show, respectively, the bottom, middle and top sections of column **D4** called for on the partial design drawings shown in Fig. 7-19. As stipulated in the general notes, the AISC Specification is used and, for the purpose of illustration, connections employing high-strength shop bolts are shown. All field connections are made with high-strength bolts. Note that the design also requires that all first and second floor beams must be provided with wind connections, consisting of heavy top and bottom flange angles, in addition to the vertical shear connections. These wind connections act in concert with all other connections, and with the structural frame, to resist bending due to horizontal wind forces. In this building, shear connections alone were considered sufficient to take wind loads in the upper tiers. As with vertical floor loads, the wind effect is cumulative and increases progressively downward. Hence, the designer has called for increased moment resistance in the lower tiers.

Column **D4(0-2)** in Fig. 7-20 contains these combined shear and moment connections. On the flange faces, where the beams can be swung in horizontally, both top and bottom angles are shop bolted to the column. Note that erection clearances have been provided at the top and bottom in accordance with Fig. 7-12c. In the connections to the column web, the top angles are shown bolted for shipment so they may be removed temporarily by the erector to permit lowering the beams into place. For these latter connections, erection clearances are provided only at the top of the beam, as shown in Fig. 7-12a. Vertical slots in the top angle, Fig. 7-12d, would eliminate the need for the clearance gap and the consequent shims. Beam web shear connections to the column flanges were selected from Manual Part 4 to support the loads as specified on the design drawings (not shown). Shear angles on face **A**, and at the second floor on face **C**, are of the conventional type with double shear in the beam web fasteners. One angle of each of these connections (shown with open holes) is bolted for shipment to allow removal prior to swinging the beams in horizontally.

The beam, 1¾ in. off center, framed to face **C** at the first floor, is supported by a connection consisting of a structural tee cut from a W10 × 49. Observe that conventional framing angles cannot be used here, because the 1¾-in. offset makes it impossible to use them with the established 5½-in. column gage. To allow clearance for shop tightening of the line of bolts through the tee flange closest to its stem, the tee was shifted ¼-in. toward the column center line and provided with a ¼-in. fill. This arrangement permits framing the W21 to the fill on the tee stem with field bolts in single shear. The resulting 1¾-in. eccentricity on the column flange bolt group was investigated and found to have negligible effect. However, eccentricity in the beam web fasteners, spaced out to permit erection of the beam and field fastener tightening, called for one additional row of bolts over normal requirements.

Shear connections in the column web are provided by stiffeners attached directly to the column web and coped to clear the vertical legs of the seat angles (see Sect. B-B). This method of coping eliminated the need for a filler plate as shown in Table VII, Manual Part 4. The fasteners through the seat angle do not develop the vertical load. Stiffener bearing area and bolts are proportioned to take the entire beam reaction. In this design, bolts through the wind bracing angles are considered to resist only the tensile force produced by the wind moments, and not combined shear and tension.

Base angles and splice plates are in accordance with design requirements. The 1⅛-in. holes in the splice plates are for the use of the erector in handling the column. Although it is doubtful if this column, weighing less than two tons, would require these holes, they are shown here to illustrate detailing practice. Heavy columns which must be plumb as they are swung into place are generally provided with some such means of attaching a lifting hitch (see Appendix C).

Columns **D4(2-4)** in Fig. 7-21 and **(4-R)** in Fig. 7-22 have shear connections which were selected from Manual Part 4. A special connection at the fourth floor, on face **A** of col. **D4(2-4)**, consists of two 8 × 4 × ½ angles designed to support the W14 × 30 centered on the flange, as well as the two 10-in. channels framed to either side (see Fourth Fl. & Low-Roof Plan, Fig. 7-19). The 4-in. outstanding legs furnish a double shear web connection for the beam and the 8-in. legs extend

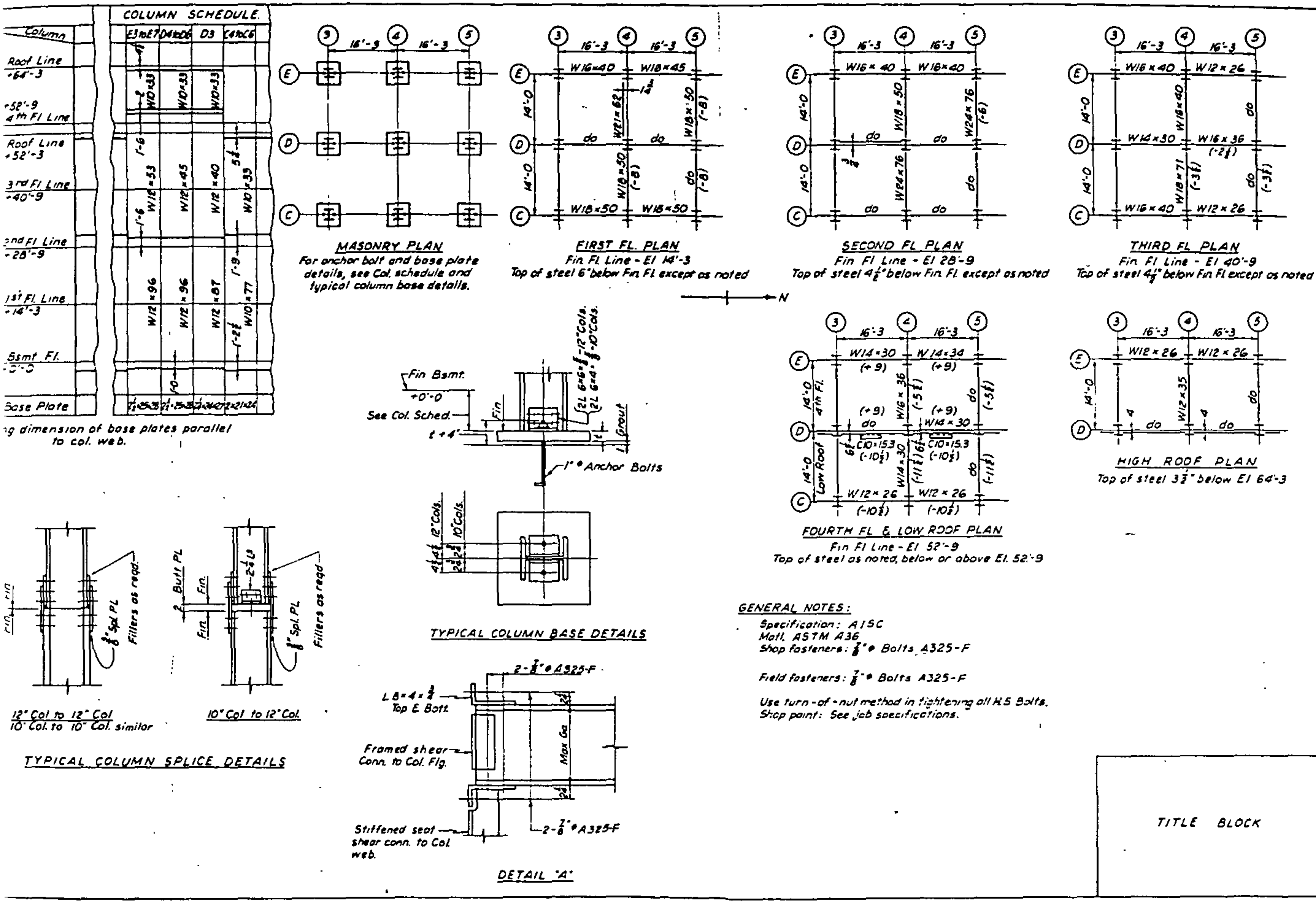
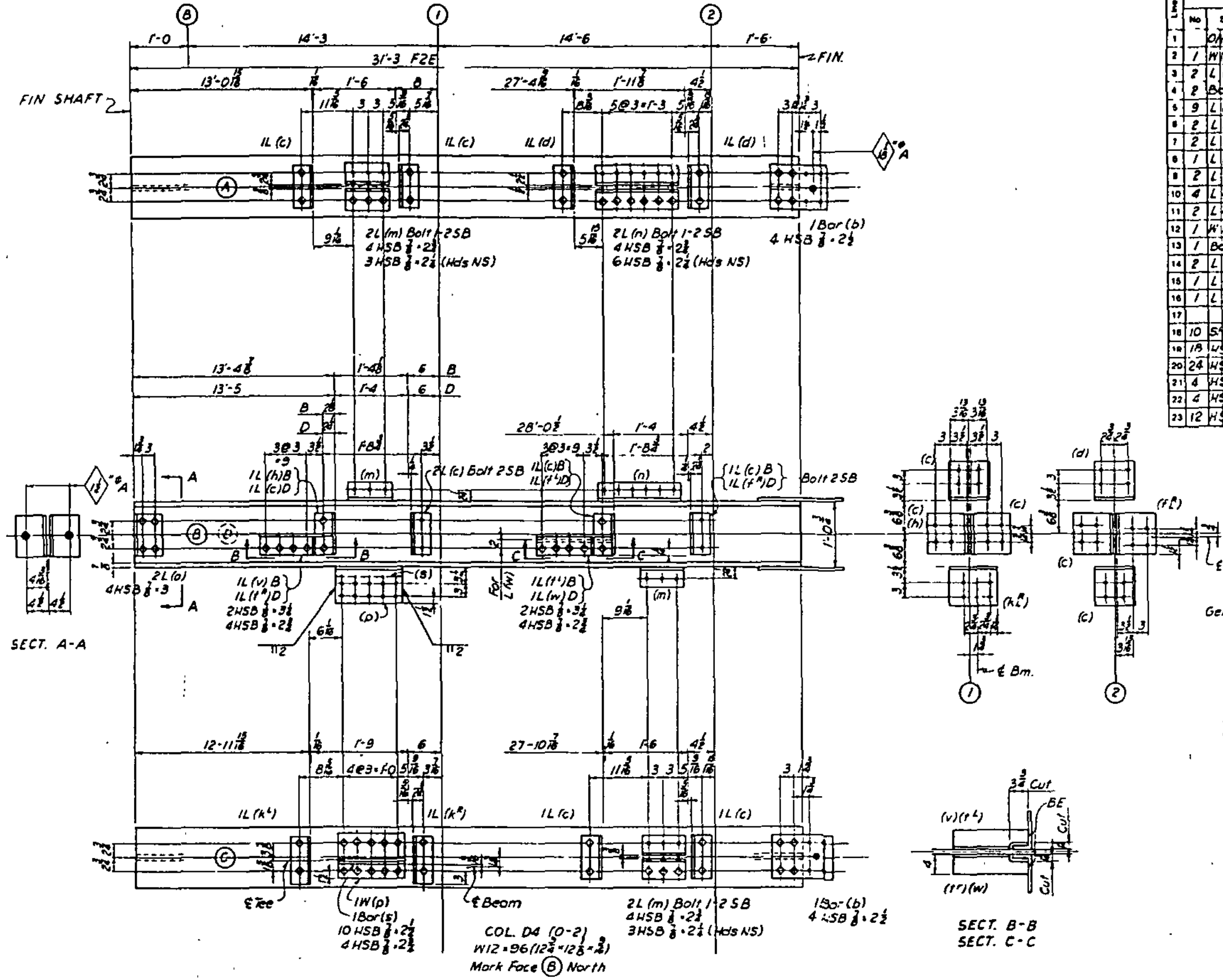

COLUMN SCHEDULE.
Column
E3 to E7 D4 to D6 D3 C4 to C6
Roof Line +64'-3
+52'-9 4th Fl Line
Roof Line +52'-3
3rd Fl Line +40'-9
2nd Fl Line +28'-9
1st Fl Line +14'-3
Bsmt Fl. +0'-0
Base Plate
W10×33 W10×33 W10×33
W12×53 W12×45 W12×40 W10×33
W12×96 W12×96 W12×87 W10×77
g dimension of base plates parallel to col. web.
MASONRY PLAN
For anchor bolt and base plate details, see Col. schedule and typical column base details.
FIRST FL. PLAN
Fin. Fl. Line - El 14'-3
Top of steel 6" below Fin. Fl. except as noted
W16×40 W18×45 W21×62 W18×50 W18×50 W18×50
SECOND FL PLAN
Fin Fl Line - El 28'-9
Top of steel 4½" below Fin. Fl. except as noted
W16×40 W18×40 W18×50 W24×76 W24×76
THIRD FL PLAN
Fin Fl Line - El 40'-9
Top of steel 4½" below Fin Fl except as noted
W16×40 W12×26 W16×40 W14×30 W16×36 W18×71 W16×40 W12×26
FOURTH FL & LOW ROOF PLAN
Fin Fl Line - El 52'-9
Top of steel as noted, below or above El. 52'-9
W14×30 W14×34 W16×36 W14×30 C10×15.3 C10×15.3 W14×30 W12×26 W12×26
HIGH ROOF PLAN
Top of steel 3½" below El 64'-3
W12×26 W12×26 W12×35
Fin Bsmt. +0'-0
See Col. Sched.
t+4"
Grout
2L 6×6×⅝-12" Cols.
2L 6×4×⅝-10" Cols.
1" φ Anchor Bolts
12" Cols. 10" Cols.
TYPICAL COLUMN BASE DETAILS
⅜" Spl PL
Fillers as reqd.
2 Butt PL
12" Col to 12" Col
10" Col. to 10" Col. similar
10" Col to 12" Col.
TYPICAL COLUMN SPLICE DETAILS
2-⅞"φ A325-F
L 8×4×½ Top & Bott.
Framed shear Conn. to Col. Flg.
Max Ga.
Stiffened seat shear conn. to Col. web.
DETAIL "A"
GENERAL NOTES:
Specification: AISC
Matl. ASTM A36
Shop fasteners: ⅞"φ Bolts A325-F
Field fasteners: ⅞"φ Bolts A325-F
Use turn-of-nut method in tightening all H.S. Bolts.
Shop paint: See job specifications.
TITLE BLOCK

Figure 7-19



BILL OF MATERIAL

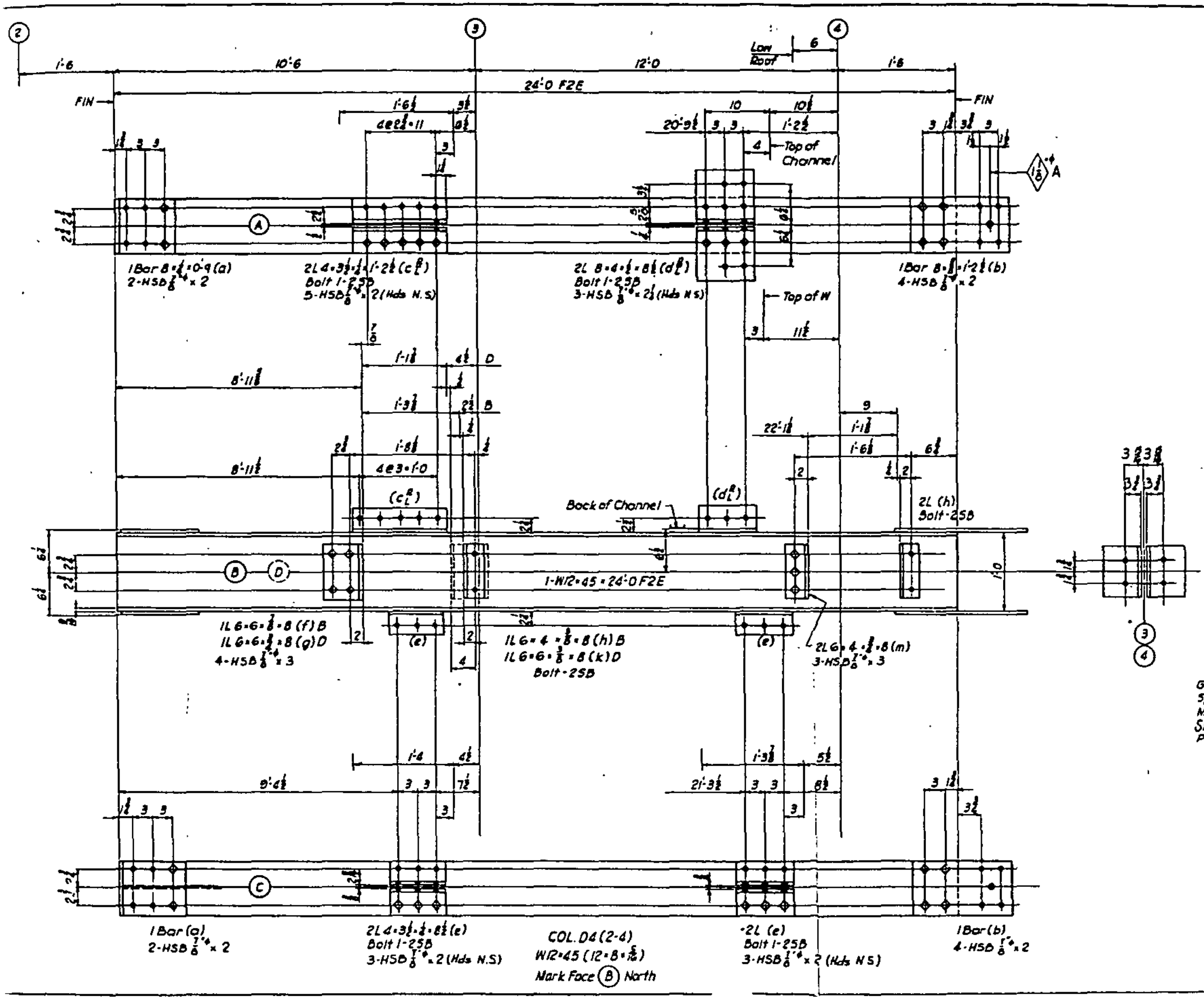
Line	SHOP BILL No	Shape	Length Ft	Length In	Assembly mark	Remarks	Weight	MILL ORDER No	Shape	Length
1		ONE COL.		D4	(0-2)					
2	1	W12 × 96	31	3		F2E				31'3
3	2	L 6 6 5/8		8	a	—				
4	2	Bar 8 5/8	1	0 1/2	b					
5	9	L 8 4 3/4		8	c					
6	2	L 8 4 3/4		8	d					
7	2	L 8 4 3/4		8	f_L^R					
8	1	L 8 4 3/4		8	h					
9	2	L 8 4 3/4		9 1/4	k_L^R					
10	4	L 4 3 1/2 1/2		8 1/2	m					
11	2	L 4 3 1/2 1/2	1	5 1/8	n					
12	1	W 10 × 49	1	2 1/2	p	Cut				
13	1	Bar 6 3/8	1	2 1/2	s	Fill				
14	2	L 4 4 1/2	1	3 1/4	t_L^R	Stiff-				
15	1	L 4 4 1/2	1	3 1/8	v	FIE				
16	1	L 3 1/2 3 1/2 1/2	1	3 1/4	w					
17										
18	10	Shop Bolts 7/8φ								
19	18	HSB 7/8φ · 2 1/4								
20	24	HSB 7/8φ · 2 1/2								
21	4	HSB 7/8φ · 3								
22	4	HSB 7/8φ · 3 1/2								
23	12	HSB 7/8φ · 2 1/4								

Gen Notes:

Spec: AISC latest edition
Matl. ASTM A36
Shop fasteners · 3/4"φ HSB · A325-F
Open holes 13/16" diam unless noted. Ope
holes are for high strength structural bolt
unless noted "A"
Paint · One coat SSPC 13
except shop contact surfaces, no paint c
NO PAINT within 5" of holes for high streng
structural bolts
BE denotes bearing end of stiffeners

TITLE BLOCK

REQUIRED		
No.	DESC.	MA
ONE	COLUMN	D4

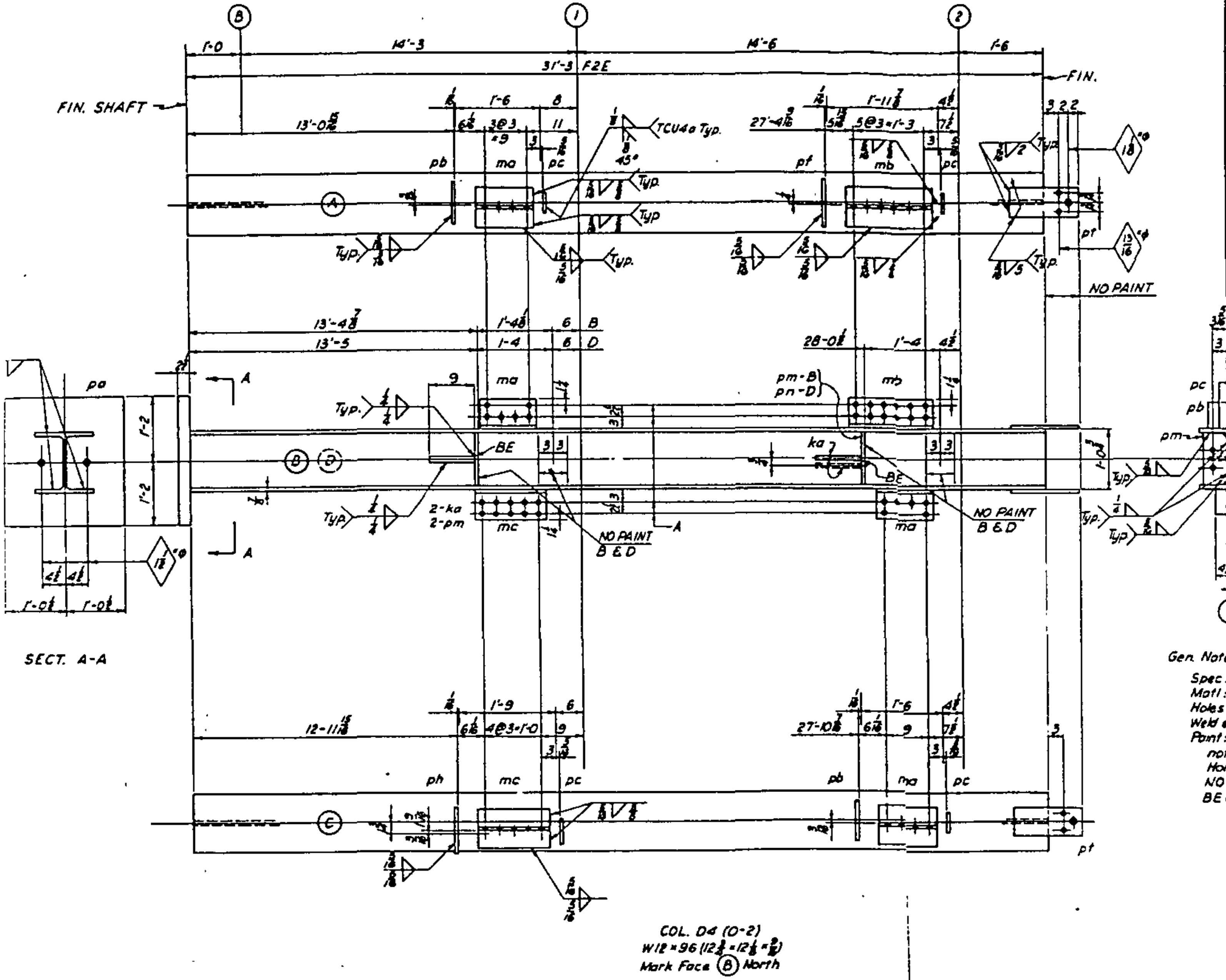


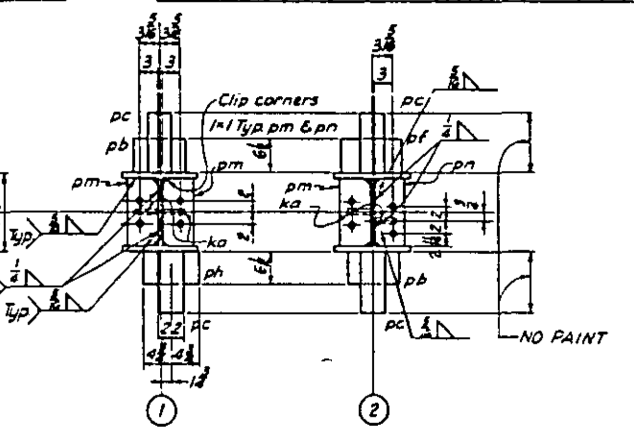
Gen. Notes:
Spec: AISC - latest edition
Mat'l: ASTM A36
Shop fasteners - 7/8"φ HSB-A325-F, Holes 15/16"φ
Paint: One coat SSPC 13
except shop contact surfaces, no p
Open holes are for high strength
structural bolts. No paint within 5
holes for high strength structur
bolts

TITLE BLOCK

BILL OF [illegible]ERIAL

Line	SHOP BILL No.	Shape	Length ft	Length in	Assembly mark	Remarks	Weight	MILL ORDER No.	Shape	Length ft	Length in
1		ONE COL	D4		(O-2)						
2	1	W12×96	31	3		F2E				31	3
3	1	PL 2×25	2	4	pa	Straighten					
4	2	Bar 6½×½		8½	pb						
5	4	Bar 4½×½		9½	pc						
6	2	HP 8×36	1	0	ma	Cut					
7	1	Bar 6½×¼		10½	pf						
8	1	HP 8×36	1	6	mb	Cut					
9	1	Bar 6½×½		9½	ph						
10	1	HP 8×36	1	3	mc	Cut					
11	3	Bar 5×½		10½	pm						
12	1	Bar 5×½		10½	pn						
13	4	Bar 4×⅜		9	ka	Stiff F/E					
14	2	Bar 6×⅜	1	2	pt						
15											





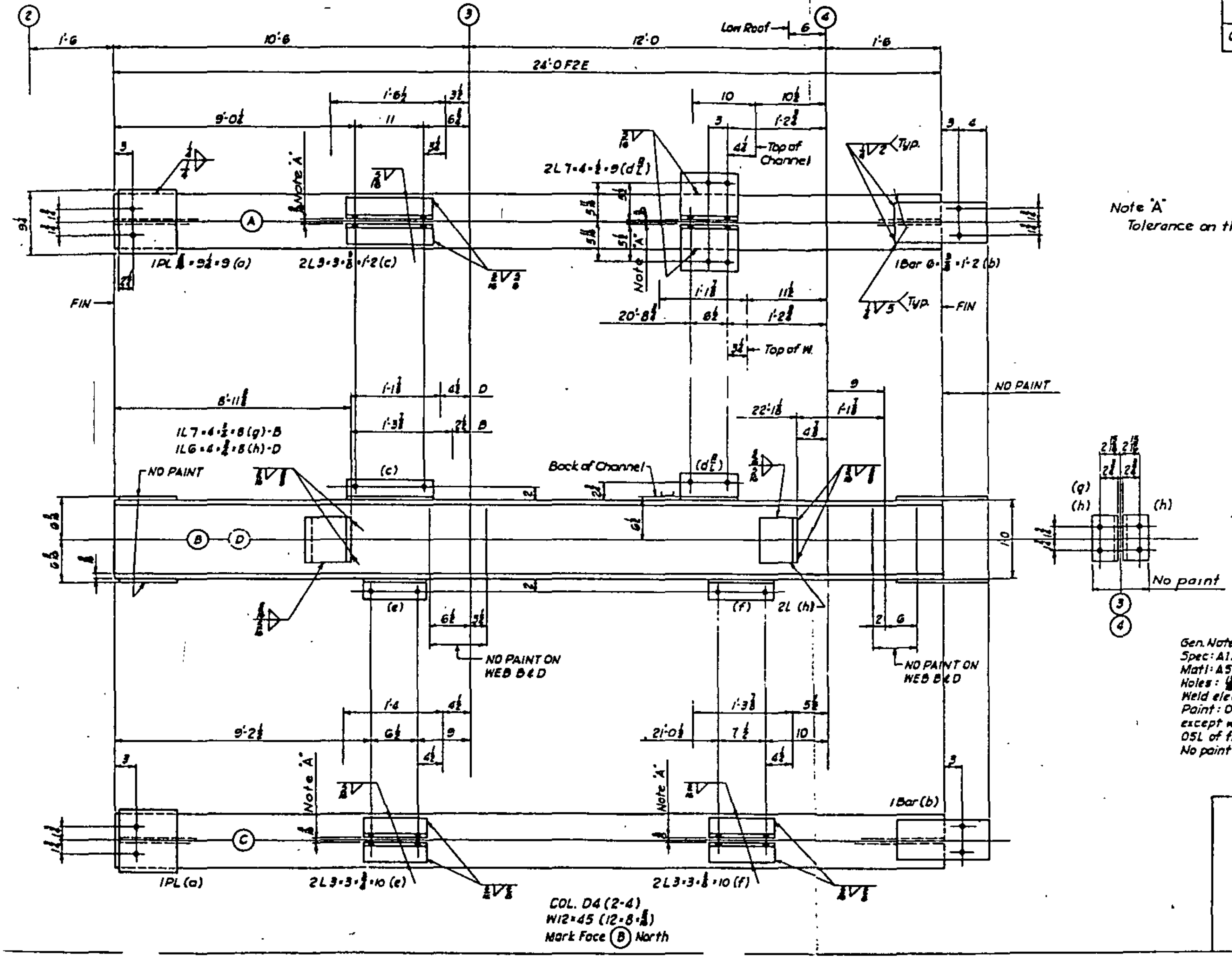
Gen. Notes:

Spec: AISC latest edition
Matl: ASTM A36
Holes: $\frac{13}{16}$"ϕ unless noted
Weld electrode: E70XX
Paint: One coat SSPC L3 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "A" are for high strength structural bolts. NO PAINT within 5 inches of these holes
BE denotes bearing end of stiffeners

TITLE BLOCK

Figure 7-23

REQUIRED		
NO.	DESC.	MA
ONE	COLUMN	D4 (



Note "A"
Tolerance on this gap = {+1/32, −0}

Gen. Notes:
Spec: AISC - latest edition
Matl: ASTM A36
Holes: 13/16" diam.
Weld electrode: E70XX
Paint: One coat SSPC 13
except where noted "NO PAINT" No paint on
OSL of framed or seated connection angles
No paint on shop contact surfaces.

TITLE BLOCK

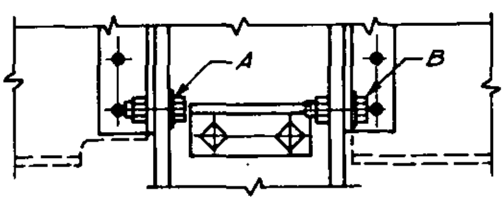
Figure 7-24

beyond the column flange to connect the channel webs. The single shear value of fasteners through the column flange is sufficient to support reactions from all three members.

The two W12 × 26 beams, shown 4 in. off center on the High-Roof Plan (Fig. 7-19), will frame most easily inside the column flange. Fills **fa**, shown attached by shipping welds at the **A** face of col. **D4(4-R)**, fill the space between the face of the beam web and the inner face of the column flange.

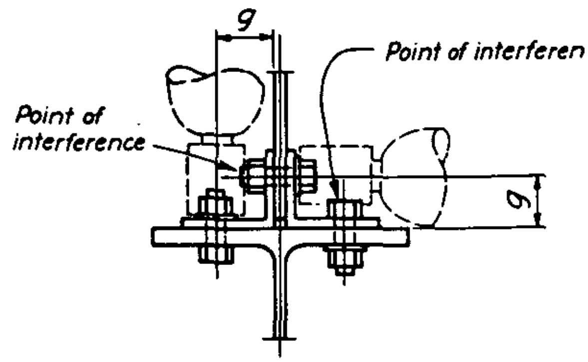
In locating details on column faces at a particular level, the detailer must be on guard constantly to prevent the introduction of difficult or impossible conditions of shop or field assembly. He must remember that the shop prefers to assemble all detail fittings to a column before bolt tightening is begun, and that the erector must place and temporarily fasten all framing in a tier before final field bolting can be started. Any arrangement should be avoided where one connection or framing member must be bolted completely before the assembly or erection of an adjacent piece. With the exception of seated beams, which can be shifted laterally, the detailer must assume that nothing in the structure can be moved once it is erected in place.

A common error in locating details occurs when fittings in the web of a column block the insertion or tightening of bolts in the holes in flange connections, as shown below:



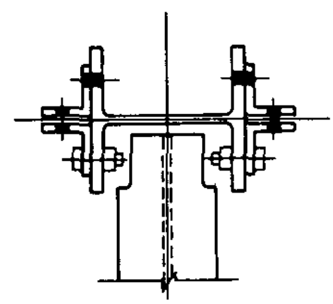
Such a situation is troublesome to the shop, because the web fitting cannot be assembled until after bolt **A** has been inserted. It may also create an impossible erection condition if the erector cannot enter or tighten field bolt **B**. Such interferences can be eliminated by changing the flange punching to avoid alignment with the web seat. This method was used at the third floor on the **A** face of col. **D4(2-4)** by spacing the flange bolts at 2¾ in. instead of the usual 3 in., to permit entry of shop bolts and field bolts above the web seats. Such interferences can be better recognized and corrected if the detailer follows the practice of detailing the relative positions and the hole locations of adjacent connections accurately to scale.

Another interference, particularly troublesome to the erector, arises when the detailer neglects to provide sufficient gage distance on the angles to permit fastener entry or clearance for impact wrench tightening.



In the illustration above, arrows show points of possible interference which can be avoided by increasing gages, *g*, by staggering fasteners along the gage lines, or by directing the shop to assemble bolts with the heads near side (or heads far side) to insure getting the necessary clearance. This problem was solved in Figs. 7-20, 7-21 and 7-22 by stipulating "(Heads NS)" for all shop bolts where the bolt "stick through" might have interfered with tightening the field bolts in the beam webs.

Another interference resulting from bolt shank "stick through" is the fouling of beam flanges by bolts through the column flanges when the erector attempts to lower a beam down the column web onto a seat. If necessary, cope the beam flanges to provide at least ½-in. clearance on each side between the protruding bolt and the edges of the beam flange, as shown in the sketch below:



Fastener clearances for bolted connections are found under "Threaded Fasteners—Assembling Clearances," in Manual Part 4. The detailer must satisfy himself on two points: (1) that shop installed fasteners will not interfere with the erection of framing and (2) that there is a possible sequence of entering and tightening by which a bolted joint can be made. If such a sequence is complicated or not obvious, the necessary steps must be outlined on the shop detail or on the erection plan, as applicable.

Note that no washers were used in the assembly of shop high-strength bolts in Figs. 7-20, 7-21 and 7 because the note on Fig. 7-19 stipulates "turn-of-tightening, for which washers are not required for the ven bolt and material specifications.

The dimensioning shown in Figs. 7-20 through 7-22 represents fairly universal practice in column work. Overall dimensions and dimensions locating floor levels are placed prominently, farthest away from the views. Detail dimensions, showing spacing of punching, gages, etc., are placed closest to the sketch. Checker's or shop inspector's figures, such as beam depths and extension dimensions, are placed in between, or wherever they will best serve their purpose.

Extension dimensions, used by the shop to establish and check the location of open holes or the tops of seat angles, are measured to the finished bottom of the column shaft. This is true even when such dimension lines are not shown full length. All dimensions relating to a connection are tied to the floor at which the connected beam appears on the plan. Note that dimensions which space drilling or punching in the column shaft are continuous within each group, and are not interrupted by the insertion of edge distances or gages.

Fittings which have been detailed once need not be dimensioned completely where they appear again on the same sheet. In Fig. 7-20, note that flange face gages on the beam connection angles appear only once for each different fitting, and that both the gages and hole spacing for repeating seat angles are omitted in the sectional views. However, the spacing of punching through the column shaft is always given, even though the fittings connected may be duplicates of one previously dimensioned.

Edge or end distances of fittings are not shown unless the punching is to be located unsymmetrically on the piece. Otherwise, equal ends or edges, resulting from the billed size of the fitting, will be provided. As a case in point, angles **(n)** on the **A** face in Fig. 7-20 will be punched with equal end distances of [(1 ft- 5½ in.) − (1 ft-3 in.)] ÷ 2 = 1¼ in. All other fittings on this sheet will be punched similarly, except angles **(k^R)**, **(k^L)** and **(f^R)**, **(f^L)**, which, being unsymmetrical, are given end distances.

Dimensions used to instruct the shop on required field clearances are shown between the outstanding legs of beam web connections, between the splice plates at the tops of columns, and out-to-out of fillers and butt plates where the bottoms of columns must enter between splice plates. The presence of these dimensions alerts the shop to clearance requirements that must be met. Critical or "tight" clearance dimensions are sometimes noted "Not more" or "Not less" where nominal allowances are permitted in one direction only. A more positive method is to note a tolerance dimension which stipulates upper and lower limits, such as "1'-6¼ ± 1/16". In this case any measured distance between 1'-6 3/16 and 1'-6 5/16 will be acceptable.

Figures 7-20 through 7-22 illustrate two methods of billing material. The first method, used in the column **D4(0-2)** detail (Fig. 7-20), utilizes a preprinted shop bill form as part of the drawing. Detail material is indicated on the drawing by assembly marks and is billed completely, in summary form, in the shop bill. The second method, illustrated in Figs. 7-21 and 7-22, uses a billing directly on the drawing. This latter method requires a separate shop bill, similar to the form printed on Fig. 7-20, on which the material is again billed in summary form.

The billing of shape descriptions follows the system recommended in Table 2-1 in Chapter 2. Notes for finish requirements follow the billing, either in the shop bill (see Fig. 7-20), or on the drawing (see Figs. 7-21 and 7-22). Allowances for finish on detail fittings are covered in the ordering procedure, and are seldom shown on either the drawing or bill. Finish allowances for column shafts, however, are usually given in the shop bill by an ordered length which includes the allowance. This is reflected in the shop bill on line 2 in Fig. 7-20 by the 31'-3¾ ordered dimension which provides for the 31'-3 finished dimension of column **D4(0-2)**. The separate shop bill (not shown) required for column **D4(4-R)** in Fig. 7-22 will contain an ordered length of 9'-6 for the 9'-5½ shaft length, finished one end.

These three column detail sheets also illustrate two systems of assembly marks. Figures 7-20 and 7-21 use an alphabetic system in which each letter identifies a different fitting. Identical fittings on the same sheet carry the same letter. In the two-letter system used in Fig. 7-22, the prefix letter describes a particular type of fitting. Thus, **a** = angle; **p** = plate, etc. The second letter denotes, in alphabetical sequence, the first, second, third and subsequent appearances of this type of fitting. In the two-letter system, different angles on the same sheet would be designated as **aa**, **ab** and **ac**, and different plates as **pa**, **pb** and **pc**. In either system, certain letters, such as **i**, **l**, **o** and others, are omitted as being too easily confused with numerals. There are other methods used including numeric and alpha-numeric. The full development of these methods will not be treated here, since the detailer will use the system adopted by his employer.

Although the general and special notes appearing on Figs. 7-20 through 7-22 are relatively simple, some explanation is in order. The note "FIN SHAFT" at the bottom of columns **D4(0-2)** and **D4(4-R)** directs the shop to finish the column prior to the assembly of detail material. The letters **B** and **D**, opposite certain dimensions and detail fittings, designate the web faces to which the dimensions refer or upon which the fittings assemble. The three numbers in parentheses that follow the column mark and size description are, respectively, the column depth, flange width and web thickness. This information, with the flange thickness shown on the web view, makes it unnecessary to refer to the dimension tables when any of these principal figures must be known during checking, computing fastener grips and providing clearances. The instructions in the general notes for painting were taken from the job specifications. The Steel Structures Painting Council (SSPC) Paint Specification No. 13 (Red or Brown One-Coat Shop Paint) will be applied in accordance with AISC Specification Sect. 1.24.* The notes require (1) complete fitting and shop bolting before any painting is done, in accordance with AISC Specification Sect. 1.24.3, and (2) omission of paint on faying surfaces of connections assembled by high-strength field bolts to provide the friction-type connections called for on the design drawing (Fig. 7-19).

Column Details—Welded Construction

Figures 7-23, 7-24 and 7-25 are details of column **D4** utilizing welded construction to support the framing shown on the plans in Fig. 7-19. It can be seen in the arrangement of views, methods of dimensioning, billing and notes that these details follow closely the bolted types shown in Figs. 7-20, 7-21 and 7-22. The principal difference lies in the presentation of connection details. Since the original design required wind-braced framing for the first two floors, the welded connections were designed accordingly.

Wind connections on the flanges of column **D4(0-2)** are made with plates shop welded to the column and field welded to the beams. Sufficient plate cross-sectional area and fillet welds are used to provide moment resistance at least equal to that of the ⅞-in. dia. A325 bolts. To achieve this "semi-rigid" effect, these connections are designed using the procedure outlined under "Moment Connections—Welded" in Manual Part 4. Although the flanges and webs of these beams could have been welded directly to the column, with less connection material, this would have resulted in a fixed end condition not desired in this structure.

To permit erection of the beams by swinging them in between the moment plates, vertical shear connections are made using tee sections cut from an HP8 × 36. This shape was selected because its 7/16-in. thick flange is about the maximum thickness usually considered for the flexible type framed shear connection specified on the design, Detail A, Fig. 7-19. An alternate solution would be a welded tee section made from two plates, preferably using a ⅜-in. thick plate against the column flange. Single plates welded edgewise to the column flange could also be considered, as was done in Manual Part 4, Moment Connections—Welded. Holes are provided for high-strength bolts to fasten the beam webs. Field welding might have been substituted for high-strength bolts.

Calculations for the bolted shear connections must consider the effect of eccentricity. Welds attaching the tees to the column plates are figured for vertical shear only. The stiffened seats in the column webs are designed in accordance with Table VIII, Manual Part 4.

First and second floor beams framing to the column web are also attached by moment plates, but the vertical loads are supported by stiffeners under the bottom plates. The top moment plates, not shown with the column, are detailed and shipped separately. This is required in order to permit erection of the beams by lowering them from above. Their appearance and the field welding required are shown in Fig. 7-26.

In the erection of welded construction, much depends on the adequacy of instructions for field assembly and welding. This usually takes the form of typical and special sketches with necessary notes, which are shown on the erection plan. Examples of such sketches are given in Figs. 7-26a and 7-26b. These show the field work necessary to make the flange and web connections on column **D4(0-2)**. Shims **SH1** and top plates **M21** and **M22**, shown detailed in Fig. 7-26c, are received at the site as loose pieces. Note that in field welding the top plates, some space along the edges remains unwelded. This is done purposely to permit the plates to stretch or compress slightly as the beam deflects under load or wind moment, thereby achieving the desired "semi-rigid" condition.

Prior to welding the column flange moment plates **pc** (Fig. 7-23) to the beams, the erector will clamp and draw each plate to the beam flange as closely as its stiffness will permit. Shims **SH1** will be placed in any opening remaining, and welds will be made in accordance with Note A (see Fig. 7-26a). Shims are not required for the web connection plates **M21** and **M22**, since they are field welded to both the column shaft and the beam.

Note that plates **M21** and **M22**, Fig. 7-26, are detailed with a width that is ⅛-in. less than the inside dimension of the column flanges for erection clearance. The curved transition is started about 1 in. from the flange toe to avoid a potential stress riser which frequently develops at an abrupt change in section such as this. Although it is preferable to place erection sketches and instructions on the erection plans to which they refer, lack of space may require their location on separate sheets. This is particularly true on large jobs, or on those having many special connections. The examples shown in Figs. 7-27 and 7-28 illustrate this practice.

**The detailer should note that AISC Specification Sect. 1.24.1 provides for omission of all shop paint on certain types of work. Although the columns detailed here might have been in this class of work, shop painting was specified to illustrate the noting incident to high-strength bolting and field welding.*

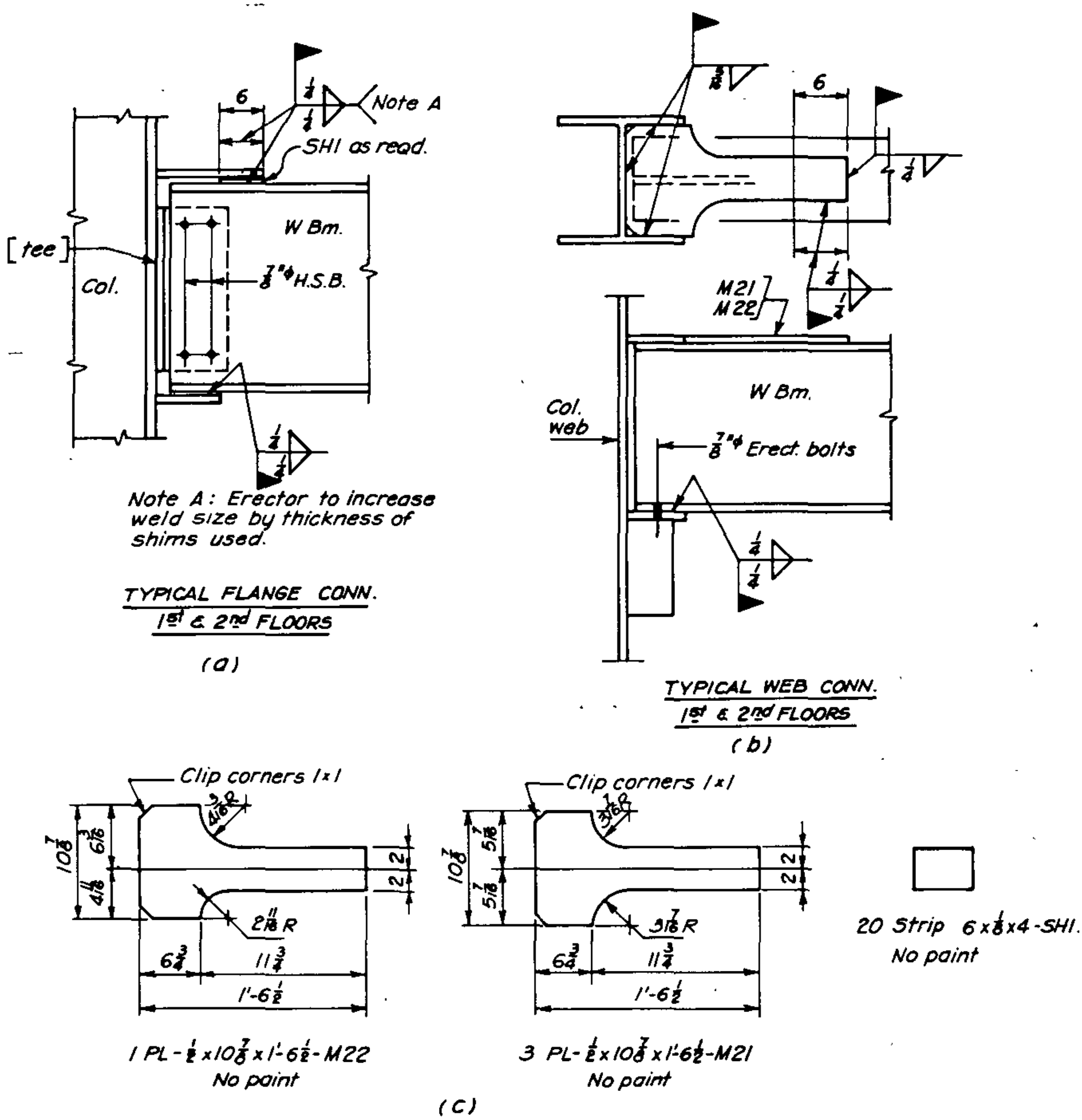


Figure 7-26

On all column details, areas noted "NO PAINT" instruct the shop to omit shop painting where field welding will occur, in accordance with Specification Sect. 1.24.5.* Similar instructions for omitting paint where high-strength bolts for friction-type connections occur are covered in the General Notes. Beams framing to these points, and their associated loose connection material, are similarly noted. After all field welding and bolting has been completed, a final paint touchup will complete the shop coat.

One feature of the beam connections to col. **D4(0-2)** requires special treatment on the erection plan. In order to insure placement of beams on the correct side of the bolted tee connections, the erection plan will show the column thus:

Otherwise, AISC Specification Sect. 1.25.5 requires that any shop paint adjacent to joints to be field welded shall be wire brushed to reduce the paint film to a minimum.

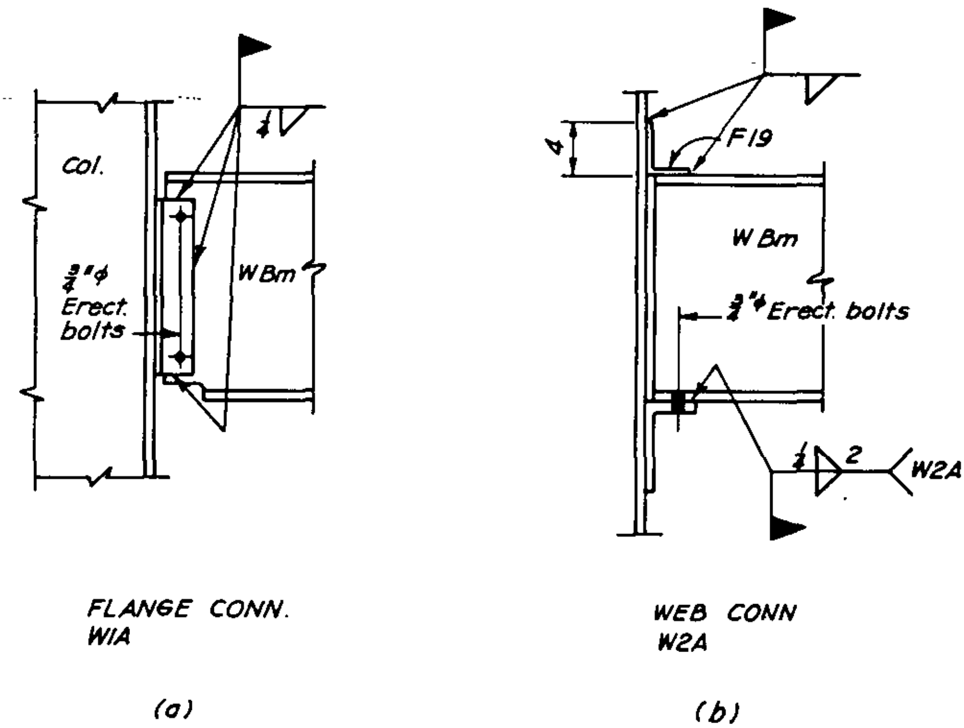
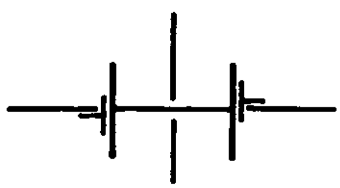


Figure 7-27



The base plate on Column **D4(0-2)**, Fig. 7-23, is welded in such a manner that all welds can be made in the horizontal position without turning the column. This detail requires the concrete footings to be finished to the elevation of the bottom of steel, or the use of leveling plates. Grouting will not be required.

Splice plates and splice welding on the columns in Figs. 7-23, 7-24 and 7-25 are based on the details recommended in Appendix C. Due to the relatively narrow flanges, it was necessary to use wide fills on the upper shafts, as shown in Cases IV-E and V-B. Figure 7-29b shows the erection drawing covering field welding of the column splices. Should any deviation from the published depths of columns, or a skewing of flanges, result in excessive clearances which cannot be closed by clamping, the erector will be required to use shims to fill these gaps. Shims for this purpose are generally furnished to the full size of that part of the splice plate overlapping the upper shaft, in ⅛-in. thicknesses for all of the splices (see Fig. 7-29a).

Column **D4(2-4)** shown in Fig. 7-24, illustrates connections that are designed for shear only. The 16- and 18-in. beams framing to column flanges are supported by all-welded framed connections selected from Table IV in Manual Part 4. The holes shown are for erection bolts needed during erection and plumbing of the structure, and for clamping the angles snugly against the beam webs prior to welding. This type of connection will require coping of the bottom flange of each beam to permit erection by lowering it from above. Field welding for these three beams happens to be the same, and is identified by the mark **W1A** on the partial plans of Fig. 7-28. It is detailed in the erection diagram sketch of Fig. 7-27a.

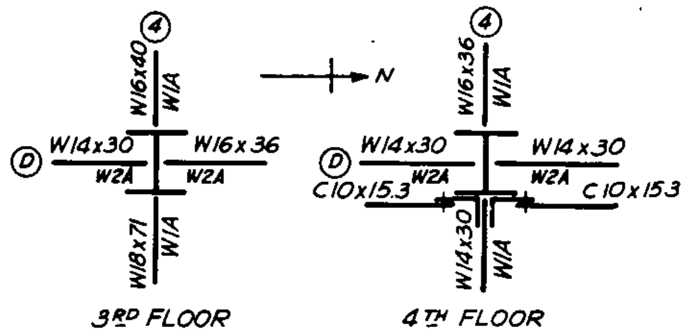
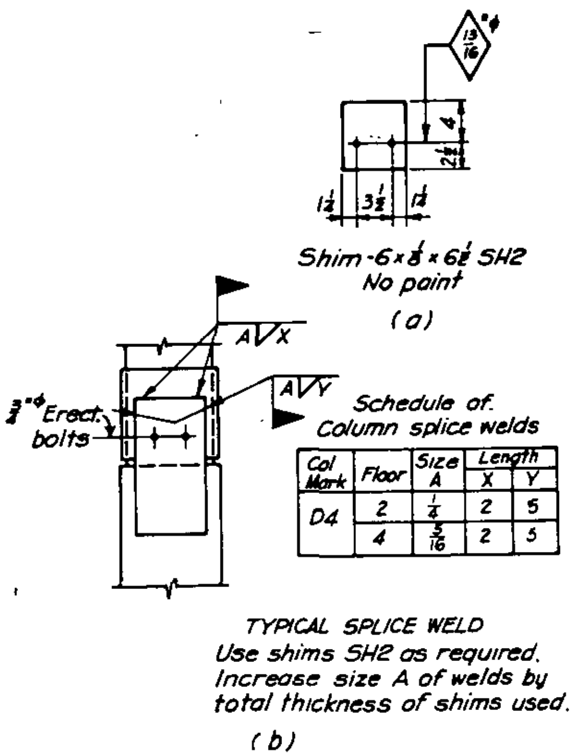


Figure 7-28

The connection on face **A** at the 4th floor is more complex than the others, as the two 7 × 4 angles are



Col Mark	Floor	Size A	Length X	Length Y
D4	2	¼	2	5
	4	5/16	2	5

Figure 7-29

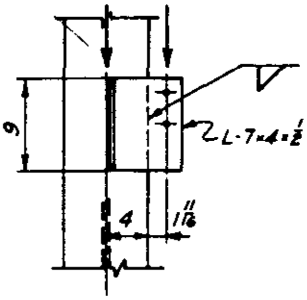


Figure 7-30

letailed to support all three members framing at this point. The W14 enters between the 4-in. outstanding legs; the two 10-in. channels are supported by the 7-in. legs (see Fig. 7-28). Since load capacities are not tabulated for this type of combined connection, it was necessary to work up a special design.

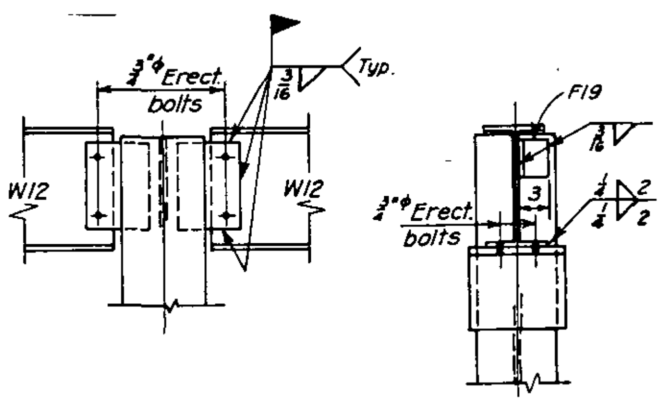
A ½-in. thickness for the angles is established by the channel location, 6½ in. from the column center. This requires ½-in. to fill out the space from the column face. The 7-in. legs, extending beyond the edge of the column flange, provide space for punched holes to connect the channel webs. Since two ¾-in. bolts are sufficient to support the channel reaction, it was decided to omit welding these members and to attach them permanently with A307 bolts. The field welds, **W1A**, applying to the W14 as shown in Figs. 7-28 and 7-27a, were designed by the use of Table XXIII in Manual Part 4, since the 4-in. OSL exceeds the leg size tabulated in Tables III or IV of Manual Part 4. Due to unequal loading, the welds attaching the 7 × 4 angles to the column are subject to twisting (moment) as well as vertical shear, as shown in Fig. 7-30.

Loads on the beams framing to the column web permit use of unstiffened seats, which were selected from Table VI in Manual Part 4. The top angles shown in Fig. 7-27b are the minimum, 4 × 3 × ¼ × 4 in. long with 3/16-in. fillet welds along the toes.

Figure 7-27, showing erection data for framing to column **D4(2-4)**, illustrates the use of reference marks. Figure 7-28 shows how the reference marks would appear on the erection plans. The use of A307 bolts with the channels at the 4th floor will be covered by the general notes on the erection plan.

Connections to column **D4(4-R)**, Fig. 7-25, include the use of an unstiffened seat on a flange face and a plate connection for off-center beams. Design of the seat is taken from Table VI in Manual Part 4. Although a 6-in. long angle could have been used, this would have required blocking the bottom flange of the W12×35 to permit horizontal welding to the seat. A 6 × 4 × ¾ seat angle with a tabular length of 8 in. was selected to eliminate this blocking. The seat is detailed with a 9½-in. length, as billed. This is necessary to permit placement of the vertical welds along the edges of the column flange.

The welds attaching the two W12×26 beams to the 5/16-in. plates are assumed to take vertical shear only. Welds attaching the plates to the column are proportioned for vertical shear and moment using Table XXIV, Manual Part 4. Erection data showing the field welding at the top of column **D4(4-R)** is shown in Fig. 7-31.



FLANGE CONNS.
TOP OF COL D4(4-R)

Figure 7-31

FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS ABIERTOS

DIPLOMADO GENERAL EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS

DIPLOMADO EN PROYECTO Y CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

MÓDULO IV

CONSTRUCCIÓN DE ESTRUCTURAS DE ACERO

TEMA

APAREJOS DE IZAJE DE ESTRUCTURAS METÁLICAS

ESPOSITOR: ING. VICTOR J. SÁEZ DE OCARIZ ALBISÚA

PALACIO DE MINERÍA
OCTUBRE DE 1998

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RIGGERS BIBLE

Hand Book of Heavy Rigging

INTRODUCTION

This book was written and compiled by a rigger who has been actively engaged in the construction of bridges. buildings and equipment installation of most every type and size since 1922. From craftsman to superintendent on jobs, from small to the largest in the world, and I might add, a person's most valuable experience is not always acquired on the largest jobs but more often on the smaller jobs where one has to resort to all of the ingenuity one possesses in order to successfully complete your job assignment with what equipment you have at hand with which to do the job.

One of my major objectives concerning this book is to assemble the largest amount of useful, practical and reliable information concerning heavy rigging and rigging accessories, and the safe application thereof, that has ever been assembled in any book regardless of size.

I have devoted neither time nor space to unnecessary, useless filler material to build up a large book, instead; I have combined the knowledge of my years of experience with the valuable information furnished me by the contributors previously mentioned in this book, into a pocket size hand book that can be conveniently carried at all times for instant reference, for all of your rigging problems.

My fascination for reeving up large capacity blocks started many years ago, at which time I made a practice of making a sketch of every different size and type of reeve up I came in contact with or came to the knowledge of in any way to be used for future reference.

In my climb up the ladder of supervision I gained first hand information on just how few men there were that possessed even a fair knowledge of reeving up blocks. Men that can reeve above six (6) parts are few and men that can reeve above twelve (12) parts are rare.

Heavy rigging is a highly specialized trade in which there are but few competent supervisors. Proof of this, is the many disastrous and costly rigging mishaps that have happened all down through the years through rigging failures. I would guess that 98% of all rigging failures are brought about through incompetence on the part of supervision by taking things for granted and relying on guess work. Rigging has always been a fertile field for guess work.

There is no reason, need or excuse to have a failure on rigging or rigging accessories. Wire rope can be visually inspected and hooks, shackles, rings, sockets and turnbuckles can be x-rayed at no great expense.

A competent rigger never guesses at any phase of his work. Before he starts a job, he works out a job plan, covering every phase and stage of his job, from beginning to completion. What he is going to use in regard to rigging and rigging accessories, the proper application thereof, and his jobs are always completed without rigging failures.

I have included reeving up diagrams in this book from six (6) to thirty-four (34) parts of line, and the various types of reeving such as, skip reeving tandem blocks, blocks with equalizer sheaves, straight blocks and blocks set at right angles to each other.

Also included are tables, formulas, rules and guides covering all rigging accessories and the proper application thereof, capable of handling loads up to and including two hundred and fifty (250) tons.

My advice to everyone that obtains one of these books is to read it carefully and thoroughly several times from cover to cover, especially the section on WIRE ROPE concerning selection, handling, installation, lubrication, operational care and inspection and remember all of the do's and don'ts.

Don't guess — Don't take chances - It Don't pay.

Don't guess — Don't take chances — It Don't pay.

RIGGERS BIBLE

Hand Book of Heavy Rigging

WIRE ROPE

Wire rope is one of construction industries most important, and indispensable pieces of precision equipment, and it is precision from the steel furnaces to the finished product. Some of the components are held to within a tolerance of plus or minus .0005" in diameter, as an example of the critical exactness to which it is manufactured.

Wire rope is subjected to the most merciless punishment of any construction equipment, even under the most ideal operating conditions and very little of it is ever used under ideal conditions, or at best under conditions but what could be improved on as to cleanliness and lubrication at least.

The wire rope users as a whole could annually, profit immensely if they would just follow the few precautionary measures and advice of the wire rope manufacturers, as to the selection, handling, installation, operational care and inspection of their wire rope, instead of relying on the advice of someone in their organization, whose ability and knowledge in this field is, in most cases practically nil.

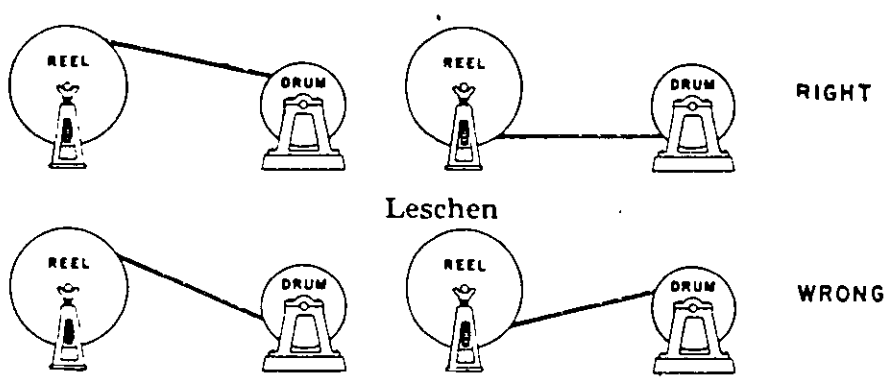
The wire rope manufacturers spend millions of dollars annually devising and experimenting with processes, means, and methods of making better rope and more efficient methods and ways concerning its operational use. Along with this they have specialists that are available just for the asking, to solve your wire rope problems for you. These specialists know and can show and tell you more about wire rope than anyone in the world. It behooves every wire rope user to take advantage of this vast field of knowledge as there are but very few people connected with the users of wire rope as a whole that possess the knowledge or know how, to select or choose the proper type rope for a particular job without first consulting a wire rope manufacturer.

Unloading and Storage

After the proper types of rope are decided on, storage is an important problem. To obtain maximum service from wire rope you have to put maximum effort into every precautionary measure from unloading at your job site, to operational care.

Regardless of whether you have ordered a half reel or a car load of wire rope, it will be packaged right, loaded right, and shipped to your job site in first class condition, and the operational results you get from it, will depend on your treatment of it from this point on. Never roll a coil or reel of wire rope out of a railroad car door or truck onto the ground or let it fall to a platform. Do not roll reels of wire rope over the ground where any hard objects can come in contact with the rope and probably damage the entire first layer beyond use. Never allow one reel to roll into another and strike the wire rope as it will flatten or distort it probably beyond use. Never pry or pinch a reel along on the Ground or platform with any kind of a bar or lever, wood, or metal unless used on one of the flanges otherwise you could damage the entire first layer of rope. Never pick a reel up with a choker around the wire rope, use a heavy bar through the center hole in the reel as this could very easily damage the entire first layer of rope. Never stack small reels on top of large reels in a rolling position in a warehouse or yard to conserve space, as the flanges of the small reels will distort or flatten the wire rope on the larger reels. If reels have to be stacked to conserve space, stack them on end on timbers to protect the bottom reels from water or dampness. Never use a fork lift to handle reels unless they are on end as the fork will flatten or distort the rope.

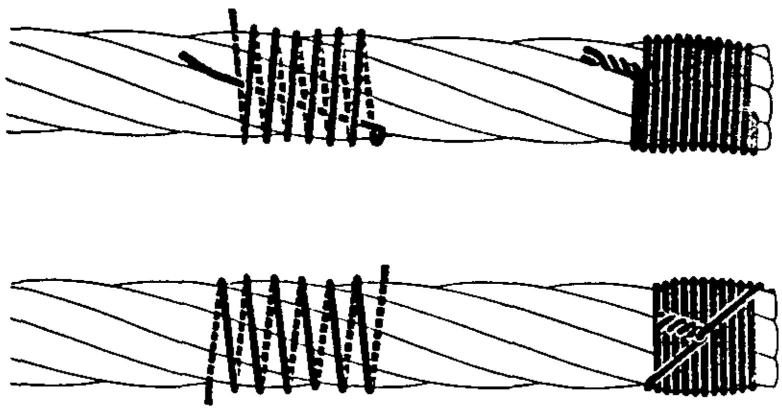
If you have an operation that will necessitate handling and cutting an excessive amount of wire rope it is advisable to have racks to accommodate all of the different sizes of rope, and a mechanical measuring device to run the rope through, then on to a reel to be delivered to the various pieces of equipment for installation. See the following detail for the correct method of spooling.



SEIZING

Do not neglect at this point the proper method of seizing of all wire rope **before** cutting it. This is of utmost importance and will contribute greatly toward successful wire rope operation. The following seizing details should be carried out before cutting any wire rope.

Two methods of seizing wire rope are shown
Bethlehem



Number of Seizings on Each Side of Cut

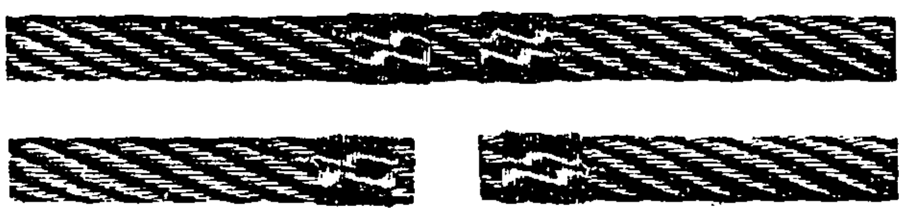
for Steel Wire Ropes, Form-Set (preformed)	1
for Steel Wire Ropes, Non-preformed:	
Ropes 7/8 in. in diam and smaller	2
Ropes 15/16 to 1 1/16 in. in diam	3
Ropes 1 1/8 in. in diam and larger	4
Lang lay ropes 1 1/2 in. in diam and larger	4
for Iron Wire Ropes, Non-preformed	2

Diameter of Seizing Wire to be Used

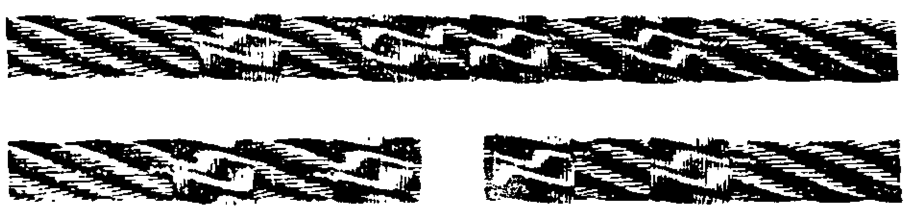
diam of Rope	*Approximate diam Seizing Wire*
7/16 in. and smaller	.026 in. or No. 23 Gage
1/2 in. and 9/16 in.	.032 in. or No. 21 "
5/8 in. to 7/8 in.	.041 in. or No. 19 "
15/16 in. to 1 1/16 in.	.054 in. or No. 17 "
1 1/8 in. to 1 1/2 in.	.080 in. or No. 14 "
1 5/8 in. to 2 in.	.121 in. or No. 11 "
2 1/8 in. and larger	.135 in. or No. 10 "

Length of Each Seizing

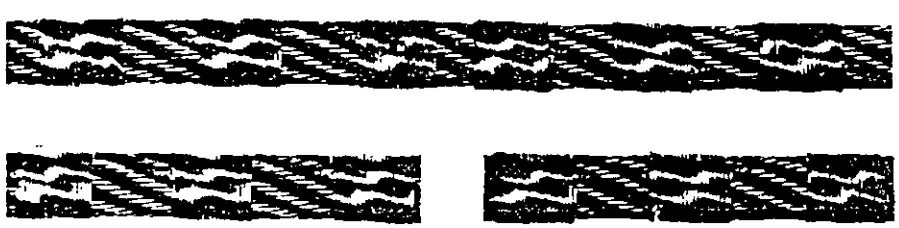
Wire ropes 7/8 in. in diam and smaller . . . equal to rope diam
Wire ropes 15/16 in. in diam and larger . . . equal to 1 1/2 times diam



Seizings for Form-Set Steel Wire Ropes before and after cutting.

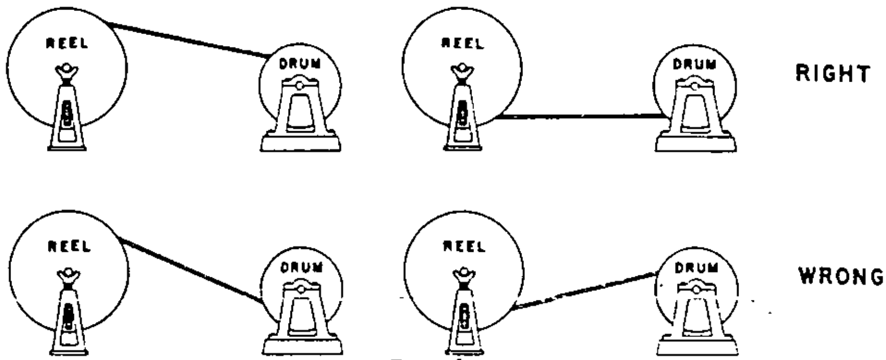


Seizings for non-preformed Steel Wire Ropes 7/8 in. in diameter and smaller before and after cutting.



Seizings for non-preformed Steel Wire Ropes 15/16 to 1 1/16 in. in diameter before and after cutting.

It is highly advisable to deliver all wire rope to the equipment that it is to be used on, on reels if at all possible, then set the reels on a stand so the wire rope can be unwound on to the hoist drums in such a manner as to avoid reverse bending the rope. (See diagram below).



Leschen

Never allow wire rope to touch the ground and become covered with grit and sand, if the operation is not large enough to warrant spare reels, hang your coil of rope on a timber, or pipe so it can be unrolled, or you can build a very inexpensive and serviceable turntable out of some short lengths of planks and unwind it in the same manner laying down flat, either method is worth many times more than what it costs just for one line alone. Do not underestimate what cleanliness alone will mean in added rope life. If possible, always use a spare reel, it pays.

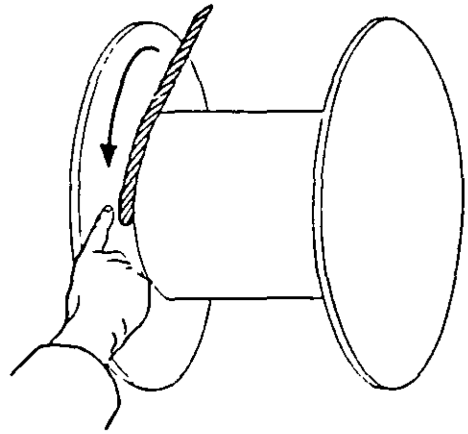
STARTING ROPE ON DRUMS

It is important that wire rope be attached at the correct location on flat or smooth face drums in order to have it spool evenly with the turns lying snugly against each other and with even layers. If the rope is started incorrectly the turns in the first layer may tend to spread apart on the drums with resulting flattening or crushing of the rope when succeeding layers are spooled.

A simple method of deciding how a rope should be started on a drum is by use of either the right or left hand. In either case, the observer stands behind the drum, with the rope coming towards him. The fist denotes the drum, the index finger the on coming rope.

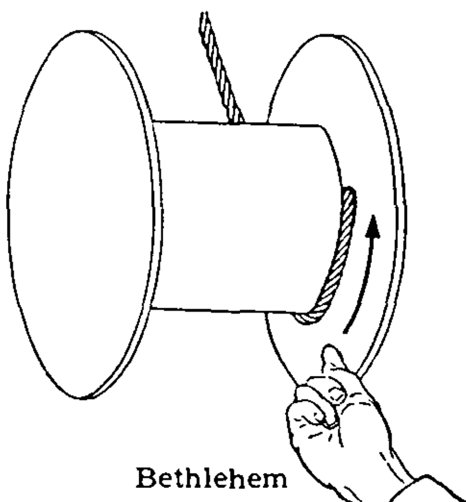
FOR RIGHT LAY ROPE

Use Right Hand



For Overwind on Drum:

palm is down, facing drum
index finger points at on-winding rope
index finger must be closest to a flange, which is the left side flange.
wind must be from left to right along the drum.

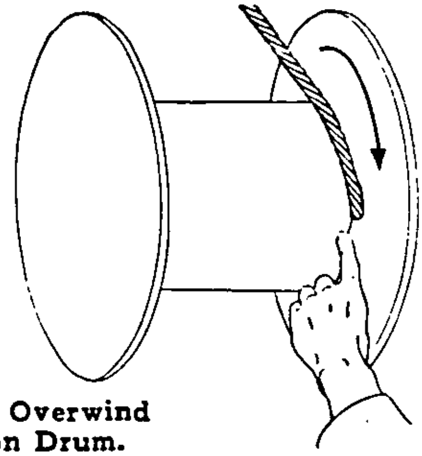


Bethlehem

For Underwind on Drum:

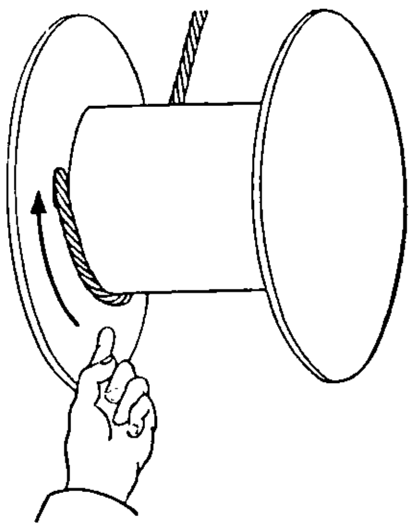
palm is up, facing drum
index finger points at on-winding rope
index finger must be closest to a flange, which is the right-side flange.
wind must be from right to left along the drum.

FOR LEFT LAY ROPE
Use Left Hand



For Overwind on Drum.

palm is down, facing drum
index finger points to on-winding rope
index finger must be closest to a flange, which is the right-side flange.
wind must be from right to left along the drum.



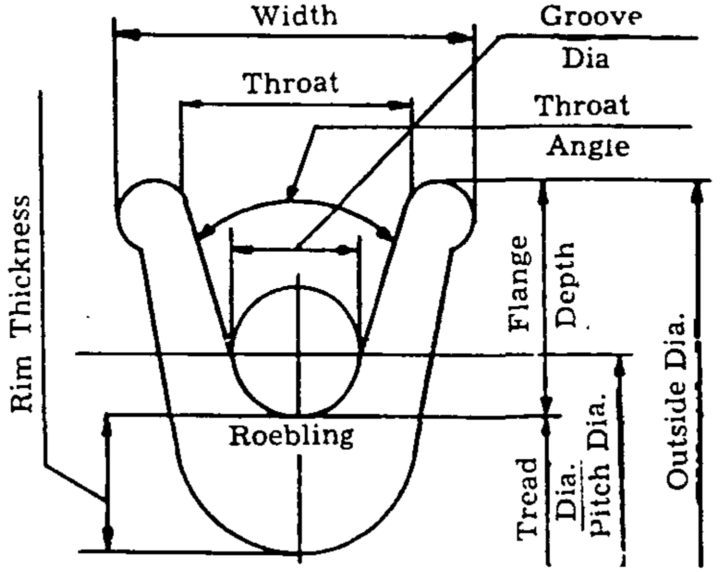
For Underwind on Drum.

palm is up, facing drum
index finger points to on-winding rope
index finger must be closest to a flange, which is the left-side flange.
wind must be from left to right along the drum.

Check All Sheaves

Average Diameter Recommended

For	6 x 7	Rope,	72	times	rope	diameter
“	6 x 8	Type D,	72	“	“	“
“	6 x19	Rope,	45	“	“	“
“	6 x25	Type B,	45	“	“	“
“	6 x30	Type G,	45	“	“	“
“	6 x37	Rope,	27	“	“	“
“	8 x19	Rope,	31	“	“	“
“	18 x 7	Rope,	51	“	“	“



Before installing any new wire rope on your equipment be sure to thoroughly check all sheaves, fair leads and load block sheaves.

The first thing to ascertain is that **all** sheaves have a minimum tread diameter as specified by the wire rope manufacturers. This should be given serious consideration as it will govern to a great degree whether your wire rope will have a long or short span of usefulness before having to be replaced.

Check the groove diameter of all sheaves with a **groove gauge only;** you cannot tell anything about the condition of a sheave with a rule. All wire rope companies make groove gauges and a good rigger is never without one.

One of the common abuses that wire rope is subjected to is to work it over sheaves that are grooved for a larger diameter rope, in this case the rope has no groove support on the sheave and is more or less operating on a point bearing which allows the rope to become flattened or distorted, which naturally brings a quick end to its period of safe operation.

A sheave with the proper groove size as recommended by the wire rope manufacturer will support the rope through an arc of about 150°, as shown on next page, Figure #1.

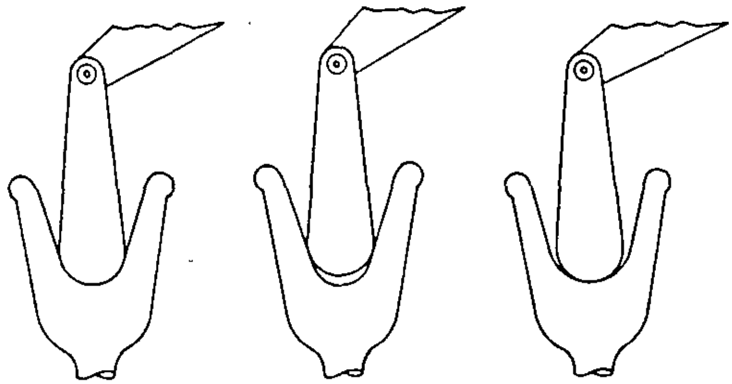


Fig. 1 **Fig. 2** Leschen **Fig. 3**

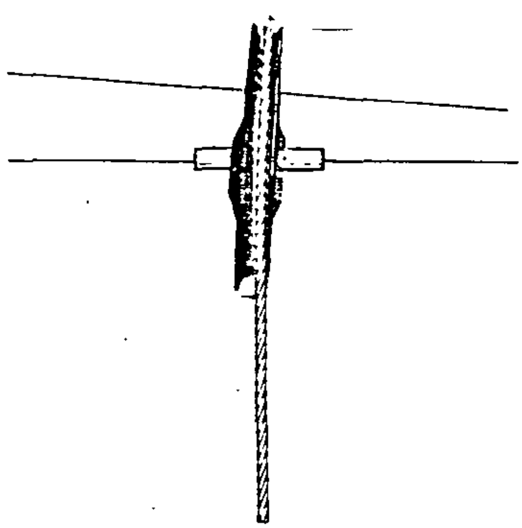
A rope operating over sheaves with undersize grooves, as shown in Fig. #2 this page, brings about a pinching, wedging condition that literally eats up both the rope and the sheaves.

A rope operating over sheaves with oversize grooves, as shown in Fig. #3 this page, has only a point bearing in the sheave, and rapidly becomes flattened and distorted and has a very short span of service.

WORN BEARINGS

Worn bushings or bearings, as shown at bottom this page, will allow the sheaves to wobble on the pins, which causes a scrubbing action of the rope on the throat of the sheaves, which will definitely shorten the life of the rope very much. .

Worn Bearings



This worn and corrugated sheave groove is a good example of a very bad condition. A new rope put on a sheave like this would be seriously and dangerously handicapped.

Corrugated treads is a condition that should be corrected without fail at any time it is found to exist on your equipment.

This is an indication of one of two things, either the sheaves are excessively soft and should be replaced with sheaves of harder material, or the sheaves are too small to support the working load imposed on them. If the latter is the case, the only solution is to replace the sheaves with ones of a larger diameter. In practically all cases, you will find this condition is caused by soft sheaves.

Laced Blocks

Another abuse that is widely tolerated on small rigs is lacing up the blocks instead of reeving them. Four (4) sheave blocks should never be laced. I have seen five (5) sheave blocks laced that would tilt the traveling block when empty nearly 20°. Blocks of that size are heavy and do a lot of damage to the load line when traveling tilted. Avoid this by all means.

If smooth drums or lagging is being used they must be free of impressions, corrugations, or of grooves worn in them.

If grooved drums or lagging is being used the grooves must not be worn excessively and the new wire rope must correspond in size exactly to the grooves of the drums or lagging as specified by the wire rope manufacturers.

No worse condition could be imposed on wire rope than to work either oversized or undersized rope on grooved drums. Either one would at regular intervals cross over the sharp ridges between the grooves. The undersize rope would not have any groove support and would flatten out. Over sized rope would be suspended or saddled in an occasional groove and riding the top of the ridges in others.

If you find any descrepancies in your sheaves or hoist drums, correct them without fail. It is like putting money in the bank. It is not uncommon for one main load to cost several times more than all of the sheaves on a rig and on a big rig the line costs many times more.

Check the hoist drum wedges for exactness in size and do not work a rig with less than three turns of wire rope on the drums. After installing a new load line, raise and lower the traveling block several times empty or with a very light load to allow the line to normalize itself as to shape and set in the position that it will work in. This procedure is worth many times the cost of the time it takes to do it.

Lubrication

The lubrication of wire rope in service is one phase of the maintenance procedure that never should be neglected. Good lubrication protects the rope against corrosion, helps to keep the wire coated, preserves the core so that wear and friction within the rope are minimized, and reduces wear to the sheave and drum equipment over which the rope operates.

Corrosion must be avoided if the strength and safety of a wire rope is to be maintained. A corroded rope is reduced in strength since some of its metallic area has been lost, but, unlike the reduction that occurs as a result of normal wear and broken wires, its effect is impossible to estimate. Therefore, if it is necessary for the remaining area and strength of a wire rope to be known, within a fair degree of accuracy, at all times during its life—and this is extremely important on installations where a rope failure would result in loss of life or costly damage—good lubrication and the absence of corrosion are of utmost importance.

In addition to this primary function, good lubrication contributes to operating economy. When it is considered that to conform to the curvature when bending over a sheave, the many elements of which a wire rope is composed must move with facility, and also that the rope is exerting considerable force against the sheave, it can be seen that many wearing surfaces are involved, and the beneficial effect of a properly coated oil film is evident at once. Most persons would not think of operating their automobiles without oil in the crankcase, nor would they operate any other machine without lubrication. However, many do not realize that wire rope should be subject to the same consideration, and fail to appreciate the extra service it can render if an adequate lubricating practice is established.

Service records from the field and tests have proven that proper lubrication is an important aid to increased rope life. It might be of interest to cite one example where a 9/16" diameter 6 x 19 wire rope was submitted to a series of fatigue tests. Part of this rope was laid-up with and part without lubricant. Samples of each section were operated over sheaves of two different diameters until the same number of broken wires in a given length had developed. The following comparison shows the results, based on the number of bends necessary to produce these broken wires.

	10" Tread Diam. Sheave	24" Tread Diam. Sheave	
Dry Rope.........	16,000 bends	74,000 bends	= 4.6 times
Lubricated Rope..	38,700 "	386,000 "	= 10.0 "
Roebling	= 2.4 times	= 5.2 times	

In a wire rope with a fiber core the lubricant not only helps to keep moisture out, but it aids in preserving the core so that it will withstand the pressures encountered in operation and support the strands of rope in their proper positions. A dry fiber core deteriorates rapidly, and failure of this vital member makes it impossible for the rope to render satisfactory service.

At the time of manufacture, a wire rope is thoroughly impregnated and coated with a type of lubricant that will best withstand the conditions under which it must operate in the field. During operation, however, the tension in the rope and the pressures encountered in its operation over the drum and sheaves tend to force the lubricant to the surface where whipping action and the effect of the elements constantly reduce it in quantity. The effectiveness of lubrication is reduced further as the rope picks up dirt, thus causing the lubricant to cake and flake off during operation.

It is evident, therefore, that to maintain the original condition of lubrication as applied to the rope when manufactured, it is necessary to replenish it during service. The question then arises as to what type of lubricant should be used, how it should be applied, and at what intervals application should be made.

Types of Lubricants

Lubricants that are suitable for wire rope application can be obtained in practically any consistency, ranging from fairly thin oils to heavy, almost tar-like substances. The material that is used, however, must be free from any element which might attack the constituent parts of the rope, and it must be (commercially) chemically neutral.

As a rule the thinner types of lubricant have the best lubricating properties, but they do not afford protection against corrosion for long periods if the rope is subject to the washing action of water or is constantly exposed to the elements. The heavy, tenacious types of lubricant are best for these conditions but they have less lubricating value, and to obtain proper pentration they must be applied in the heated state. Therefore, the choice of lubricant must depend on the method of aplication and the specific factors which must be confronted.

Where corrosion is apt to be a very serious factor, such as ropes used for dredging or on a particularly wet mine shaft installation, a heavy lubricant will give the best protection. Medium lubricants are used on fairly long service ropes that operate out-of-doors and have only the elements with which to contend. Ropes used on cranes, derricks, conveyors, etc., would fall within this range. In case of relatively short life ropes such as those used on excavating equipment, where corrosion is not of serious consequence, a thinner lubricant is most effective. Generally, the thinner lubricants are applied to ropes that operate indoors, such as on shop cranes and elevators, because here corrosion is not apt to be involved, and a material that has good lubricating qualities and can penetrate the rope even when applied cold—gives the best results.

In many cases where the protection afforded by a medium or heavy-bodied material is needed, difficulty is encountered in applying either type in the heated state which makes their use impossible. Here a thinner lubricant can be used but to insure adequate protection it must be applied at more frequent intervals.

Wire rope should be clean and dry before lubricant is applied. In some cases, where the rope picks up dirt and grit and loses its effectiveness raidly, it is sometimes necessary to clean it prior to each application. Cleaning can be accomplished by jets of compressed air or superheated steam, or it can be done with stationary or power-driven brushes. If brushes are used, it may be necessary first to loosen the dirt and the old lubricant with penetrating oil or a good grade of kerosene.

Methods of Application

There are many ways a lubricant can be applied satisfactorily, and, of course, a method that is easy and economical in one case may not represent the most efficient method in another. Where relatively short lengths of rope must be treated it is easiest to do the job by hand, but with long lengths this method is time-consuming and tedious. Therefore, longer ropes, such as shaft hoist or incline ropes, or elevator ropes in the larger buildings, can be treated best by some type of applicator. Where the medium and heavy types of lubricant are used, they must be applied in the heated state so that proper penetration and adherence to the rope will be effected, and naturally it is easier to handle such materials in some sort of applicator.

In applying a lubricant by hand, it can be swabbed on or poured on the rope. These methods are satisfactory if a wiper is arranged to remove the excess from the surface of the rope and to prevent waste.

Applicators of various types have been designed, some of which are removable after application and others which are adaptable to the continuous application of a small quantity of lubricant. The first type consists of a funnel which can be clamped around a rope operating vertically, or a trough or rectangular box to be used in conjunction with a rope operating horizontally. The easisest type to make in the latter category is a device that continuously dispenses the lubricant; other types have been made, however, so designed that the lubricant is applied to the rope only when it is in operation. Naturally with these devices the thinner types of lubricant must be used and, if employed on installations where protection against severe corrosion must be provided, they may not be adequate unless used to supplement heavier lubricants that are applied at set intervals.

For some types of installations, spray devices have given satisfactory service. A special lubricator for elevator ropes has been so designed that lubricant can be applied to a number of ropes at one time and details of this device will be furnished upon request.

Frequency of Application

It is impossible to make specific recommendations regarding the time interval that should elapse between lubrications, except to reiterate that the rope should be properly protected at all times. In some cases lubricant must be applied weekly, while in other instances once every six months or even once a year would be sufficient. The time element depends on how the rope is used, the loads it handles, its frequency of operation, its exposure to corrosive influences, and practically every condition in connection with its operation. Usually the frequency of application can be determined by observation, and the efficiency of a lubricating practice can be checked definitely by carefully inspecting sections of wire rope that has been removed from service and examining them internally for wear, corrosion, and the condition of the core.

Roebling wire rope engineers will be glad to make such examinations, and report on the condition of the rope and, at the same time, state whether or not the lubricating practice followed during its life has been sufficiently adequate to afford good protection.

Cutting and Reattaching at the Drum

On installations where the rope must wind on the drum in more than one layer, wear does not occur in a uniform manner, even when all possible precautions have been taken to make the winding condition as easy as possible. The rope usually is subject to the most wearing action at the change of layers and at the points of cross-over. If it is operated in the same position on the drum throughout its life, so that the same sections constantly are subject to this abusive action, rope life will be less than would be the case if this abuse were distributed more evenly.

This distribution of wear can be accomplished by periodically cutting a short length of rope from the drum end and reattaching it so that other sections fall at the change of layers and at the cross-over points. The usual procedure is to cut a length equal to 1¼ wraps around the drum so that the change of layers will be at least once removed from its former position, and so that the cross-overs will be shifted 90°. On very large diameter drums, such as on shaft hoists, a length equal to ¾ of a wrap may be sufficient.

This cut should be made before the sections in question show appreciably more deterioration than the remainder of the rope, and, in most cases, best results will be obtained if three or more such cuts are made at even intervals during the life of the rope.

How To Inspect Wire Rope

Methods of inspecting wire rope can be fairly well standardized, although inspection practices in factories must necessarily vary to allow the peculiarities of the individual installation. Evidences of deterioration in wire rope will, within fairly wide limits, be the same whether the rope is working on a mine shaft hoist or a small elecetric hoist. Frequency of inspection, however, can vary from daily to yearly, depending on the installation.

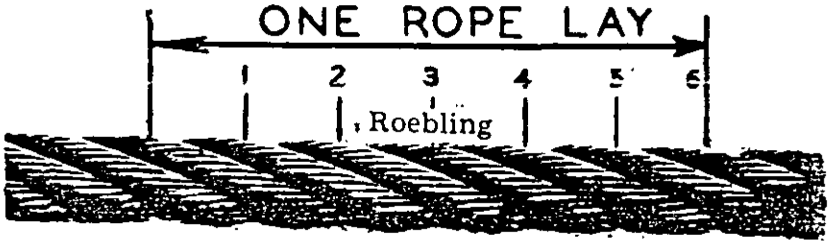
What Inspection Will Show

Deterioration of wire rope will be evidenced by: 1. broken wires; 2. worn wires; 3. pitted or corroded wires; 4. drastic reduction in rope diameter and excessive lengthening of lay; 5. marks of mechanical abuse, such as flattening and distortion.

To inspect wire rope properly is to discover its worst spot. Thus, an inspection must include the entire length of the rope. If the rope is covered throughout its length with a heavy lubricant on the outside that obscures a view of the broken wires and of the abrasion worn surfaces of the wires, provision must be made for the inspector to ascertain where the wire breaks are occurring, and the degree of the abrasion abuse the rope has suffered. In the majority of factories, however, it will be possible to ascertain the section of rope showing the most deterioration simply by looking at it.

Inspection Should Include

A quick check of the number of broken wires by rope lays, so that the worst lay can be selected. One rope lay,



Measuring One Rope Lay in a Six-Strand Rope.

Measuring One Rope Lay in a Six-Strand Rope

is the distance in which one strand makes one complete revolution around the rope.

The worst broken-up rope lay should be selected for accurate counting of broken wires, since lay constitutes the weakest point in the rope.

Note whether the majority of these wire breaks are occurring in one or two strands, rather than being fairly evenly divided among all strands. If they are concentrated in one or two strands, the rope will be considerably weaker than if they are evenly distributed.

It should also be noted where the majority of wires are breaking in relation to their position on the strand; that is, whether they are crown breaks occurring on the top side of the strand, or whether they occur in the valleys between adjacent strands. If they are crown breaks, they probably indicate normal deterioration; if valley breaks, there may be an abnormal condition.

A check of the diameter of the rope, throughout its length should disclose any drastic reduction from the catalog diameter at any point. Such reduction will indicate that

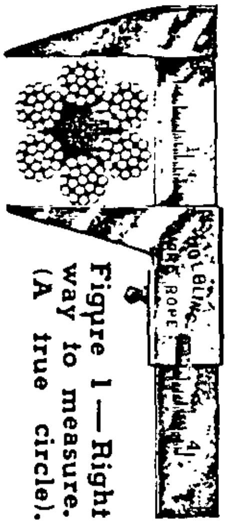


Figure 1 — Right way to measure. (A true circle).

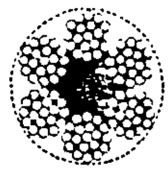


Figure 2—The diameter of wire rope is that of a true circle. Note where the dotted line touches the strands.

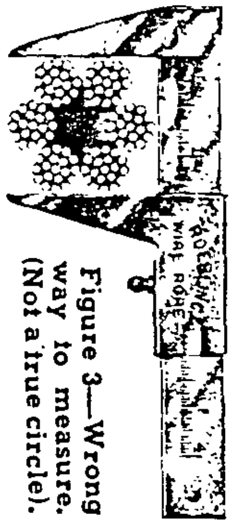


Figure 3—Wrong way to measure. (Not a true circle).

the hemp center has dried out and collapsed, the rope has been sretched unduly, or internal corrosion is present.

Inspect the degree of abrasion wear occurring on the wires. Either the length of wear should be discovered or an actual diameter check of the individual wires should be made. It is difficult to measure this wear accurately, especally in a rope built of small diameter wires; but some check must be made.

Note carefully, not only in the worst broken-up section of the rope but, throughout its entire length, whether any signs of rust are appearing. Internal corrosion in a rope may be evidenced by a showing of rust in the valleys between the strands. Pitting also will usually be discovered more readily in the valleys. It is possible to have internal corrosion, however, with no external signs.

Note whether wire breaks and abrasion occur in one short section, or whether they are distributed throughout the strength of the rope. Usually wire breaks will be concentrated in the portion of the rope which passes most frequently over the sheaves during normal operation; in sections of rope that wind onto drums most frequently; or in sections where wear is concentrated either by mechanical abuse or other factors.

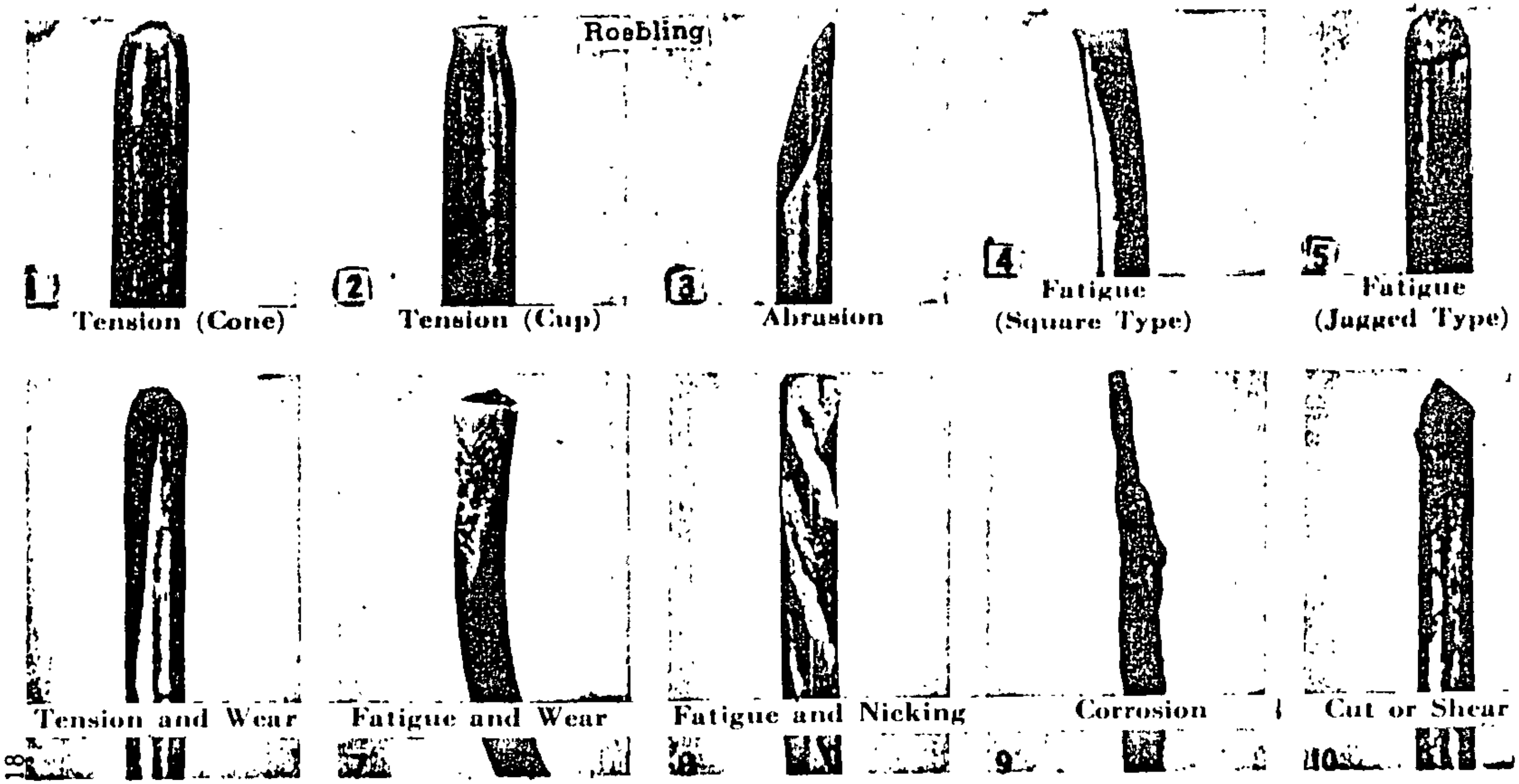
Note carefully the condition of the rope near attached fittings.

Diagnosis

Wire breaks in a rope occur from a number of different causes, of which the more common are shown in the chart on the next page.

If it is desired to make a detailed study of rope service to discover the maximum reasonable rope life for a given installation it is necessary to know the distribution and character of wire breaks, and other defects, their location on the equipment and the rate of deterioration. This data

Typical Characteristics and Causes of Broken Wires in Wire Rope



TENSION — Wire break shows one end of broken wire coned, the other cupped. Necking down of broken ends is typical of this type of break. (See Figures 1 and 2.)

Where tension breaks are found, rope has been subjected to too great a strain, either for its original strength or for the strength remaining in it after other factors of deterioration have weakened it. (See Figure 6 where wire is weakened by abrasion, but still shows characteristics of tension break.) Frequently tension breaks are caused by suddenly applying a load to a slack rope, thereby setting up incalculable impact stresses.

ABRASION — Wire break will show broken ends worn to a knife-edge thinness. (See Figure 3.)

Abrasive wear obviously will be concentrated at points at which the rope is rubbed most constantly. These points usually are the grooves of sheaves and drums and other objects with which the rope comes into contact. Unwarranted abrasive wear indicates improperly grooved sheaves and drums, or other localized abrasive condition.

FATIGUE — Wire breaks are usually transverse or square showing granular structure (see Figure 4). Often these breaks will develop a shattered fracture (see Figure 5). Both of the above characteristics depend upon conditions of operation.

Where fatigue breaks occur, rope has repeatedly been bent around too small a radius. Whipping, vibration, pounding, and torsional stresses will cause fatigue. This action is accelerated by abrasion (Figure 7) and nicking (Figure 8).

CORROSION — Can easily be noted by pitted surface of wire, with break usually showing evidence of one of the three foregoing factors. (See Figure 9.)

Indicates improper lubrication. Extent of damage by corrosion to interior of rope is extremely difficult to determine; consequently, corrosion is one of the most insidious and dangerous causes of rope deterioration. If fiber core of wire rope is not lubricated and is allowed to dry out, it will collapse and fail to afford proper support for strands, thereby causing marked reduction in rope diameter and extreme internal wear.

CUT or SHEAR — Wire will be pinched down and cut at broken ends, or will show evidence of a shear-like cut. (Figure 10.)

This condition is evidence of mechanical abuse caused by agents outside the installation, or by something abnormal or accidental on the installation itself.

in reasonable detail will indicate either whether a more suitable rope can be used, or whether improvements should be made on the installation.

Important though it is, the question of when wire rope should be replaced is very difficult to answer in general terms. There are no hard-and-fast rules that tell just when to remove rope from all installations. Safe, economical, and practical rope removal practice can be established only by close study of each installation.

Two considerations apply in every case:

1. Safety
2. Economy

Thus, the question to be answered really is, "How early should a rope be removed for safety's sake; and how late can it be removed for economy?

Margin of Safety

It is common practice to allow a factor of safety in wire rope installations. This factor of safety is the ratio between the maximum calculated load and the ultimate strength of the rope. These factors of safety vary considerably according to the installation. They are large where rope failure might cause loss of life or serious property damage.

Such safety factors, however, do not afford a really accurate measure of the safety of an installation. The true gauge of safety is not the original factor of safety when the rope was new, but rather the amount of usable life left when it is removed.

Economy

Economy demands that, insofar as possible, the maximum life of a wire rope be obtained. Good rope practice attempts to establish the greatest length of service commensurate with safety.

Performance Records

Considering each individual rope installation from the standpoint of these considerations and the information given in the preceding articles, obviously the safest and most economical practice in removing wire rope can be established only by observation of the effect on the rope of the individual characteristics of a given installation. Therefore careful records should be kept of the performance of the ropes, noting particularly the manner and rate of deterioration. As ropes are removed they should be submitted to the rope manufacturer for examination and determination of the remaining strength by an ultimate strength test. The recorded data on the manner and rapidity of deterioration, together with the actual test results, will allow subsequent ropes to be judged more accurately by surface inspection. After several such typical ropes are tested, sufficient data should be available to form a sound basis for judgment as to time of removal.

Estimating Remaining Strength

The procedure outlined above is the only practical way in which accurate and thoroughly reliable data can be collected on proper removal practice. Guides may be established, however, by which remaining strength can be approximated since this will be governed by the extent to which it has been affected by a combination of broken wires, fatigue, abrasion, corrosion, abuse and unbalance.

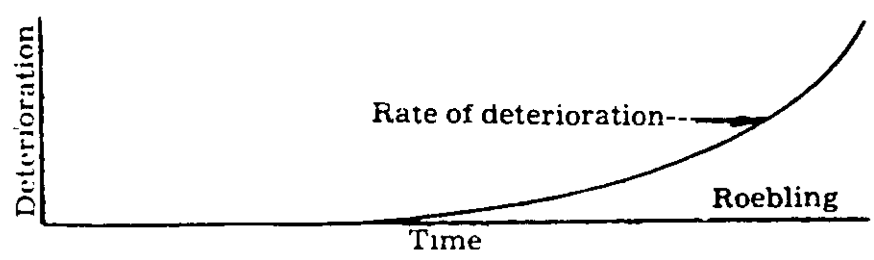
Assume that for a certain installation of 6 x 37 construction rope the safety and the operating departments have set a safety practice requiring that the rope be removed when 80 percent of the original strength remains. If broken wires were the only factor to be considered, 50 broken wires in one rope lay could be allowed. If, however, abrasion has worn away 25 percent of the cross-sectional area of the outside wires, only 42 broken wires per rope lay would reduce the strength to 80 percent. If abrasion has worn one-third the way through the outside wires, only 26 breaks per rope lay are permissible. Suppose abrasion is increased to 40 percent; then only 10 broken wires will reduce the strength of the rope to 80 percent.

It is readily seen that the effects of one deteriorating factorr is greatly augmented by those of another.

This hypothetical case assumes the use of a rope of 6 x 37 construction in which there are 18 outside wires per strand, and the figures given indicate the rope's condition only when there are no deteriorating factors other than broken wires and abrasion. Every installation therefore must be considered as an individual unit in judging the amount of deteroration safe to allow.

Rate of Deterioration

Even under ideal conditions of operation, the wire rope ultimately will reach the end of its service because abrasion and broken wires will eventually develop. After a certain length of service deterioration of a rope increases rapidly, as the illustration shows. Therefore it is not safe to assume



Deterioration of wire rope increases rapidly after a certain period that because 5 percent strength was lost each month during the first three months of service, the same rate will continue. The point at which the rate line swings acutely upwards may be reached any time during the operation of the rope.

Where corrosion is a factor there is no safe guide for judging the proper time of removal of the rope. Even ultimate strength tests on corroded ropes can be misleading because they cannot indicate the rapid rate of deterioration that might occur if the rope were continued in service. Corrosion in a wire rope can be controlled by proper and suitable lubrication. When corrosion has started in a rope it can be retarded by lubrication, but the lubricant cannot restore the rope to its original condition; therefore, this rope must be regarded as a corroded rope.

FLEET ANGLE

This is the angle between the center line of the lead sheaves and the rope. The lead sheave is the first sheave over which the rope passes as it winds off the drum.

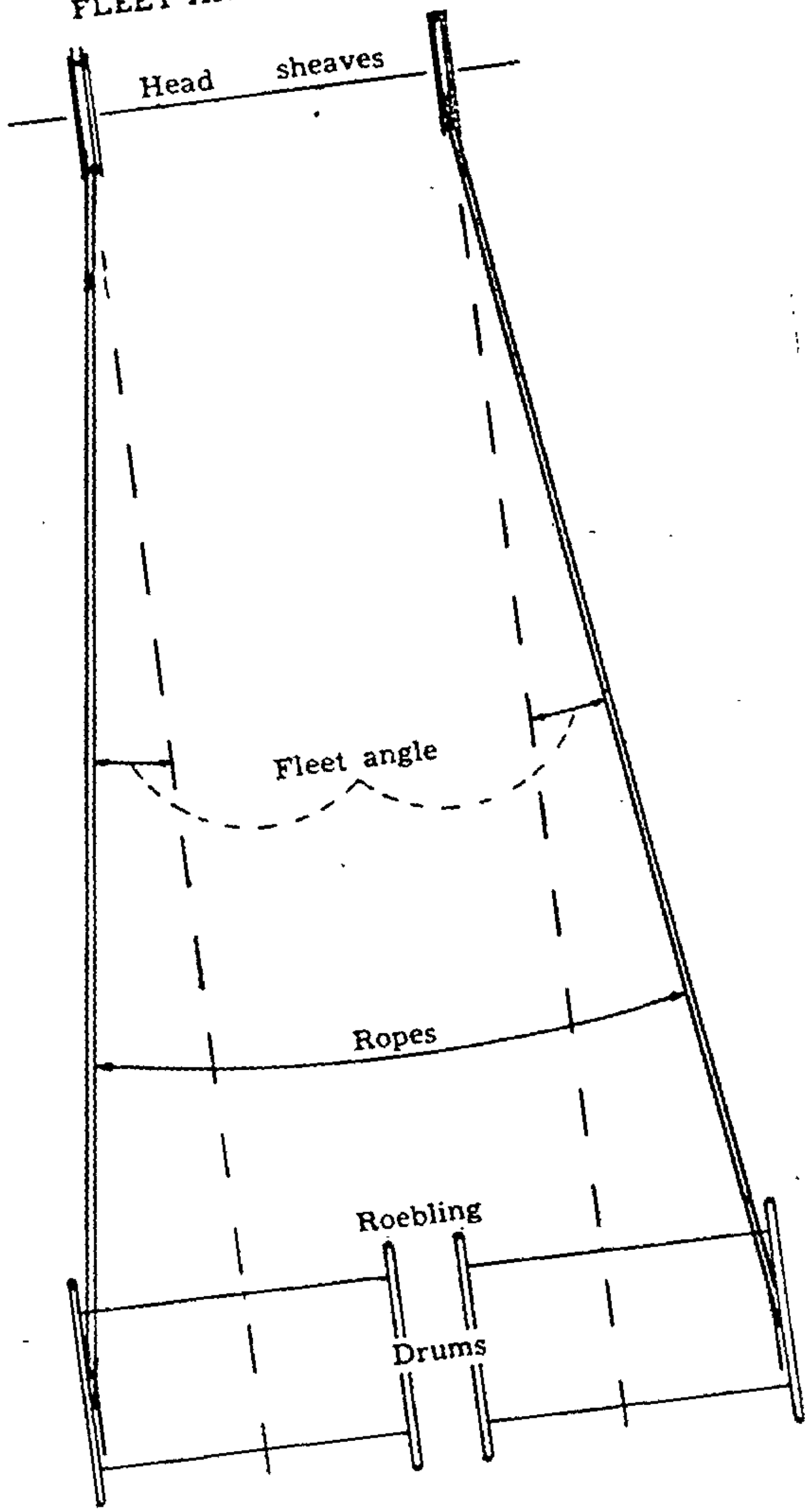
The fleet angle must be kept within certain limits. If it exceeds 1.5 degrees the rope may wind unevenly, creating scrubbing in spots, and winding loosely in others. Too large a fleet angle will cause the rope to scrape against the flanges of the lead sheave, adding another source of abrasive wear.

If the fleet angle is less than 0.5 degrees, the rope may tend to pile up at the flanges and to wind unevenly after more than one layer of rope is wound on the drum.

Rollers

In some installations rollers or other devices are used to guide the rope or to protect it from scraping against some part of the machinery with which it is not supposed to come into contact. Such rollers should never be less than eight times the diameter of the rope and must be kept in condition to rotate freely. Any rollers with corrugated or imperfect surfaces should be immediately replaced.

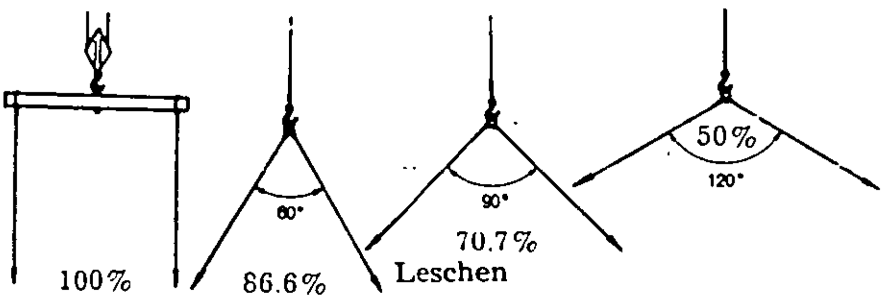
FLEET ANGLE DETAIL



WIRE ROPE SLINGS

It is very essential for the sake of safety that slings of sufficient strength be selected. In making the selection, where two or more legs are involved, consideration must be given to the important fact that the stress in a sling varies with the angle at which the legs are used.

In the diagram at the left below a bridle sling is shown with both legs vertical. These are usually attached to a spreader bar and used to lift locomotive or car bodies. The typical bridal sling is seldom used with both legs vertical. They are generally spread apart as shown.



If the legs are to be vertical, the full safe working load of the sum of the two ropes is available. When the legs are spread at an angle, the allowable safe working load will decrease as the angle increases.

How to Calculate Safe Loads on Slings

For a sling of given rope diameter the safe working load at 60 degrees, 90 degrees, and 120 degrees would be calculated by using the percentage, as shown on this page, or for any other angle, by reference to the charge on page #27 .

The degree of spread may be designated either by the included angle at the hook, or by the angles made by the legs with a horizontal plane.

In the case of a sling with equal length legs, spread at an angle of 60 degrees at the top, the legs would also make an angle of 60 degrees with the horizontal. If the angle at the top was 90 degrees, the legs would make an angle of 45 degrees with the horizontal. The relationship is based on the law that the sum of the interior angles of a triangle is 180 degrees.

How to Calculate Diameter of Rope Required

To determine the diameter of rope to be used for given conditions, the stress in each leg of a two-legged sling is taken as equal to half the load divided by the size of the angle at which the leg is inclined to the horizontal.

The following diagrams show how the tension in the rope increases as the angle with the horizontal decreases. Diagram F demonstrates what a very high tension is produced when a sling is used at a very flat angle.

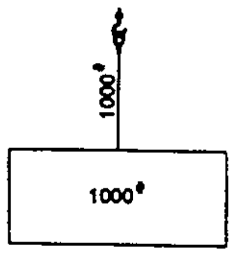


Diagram A

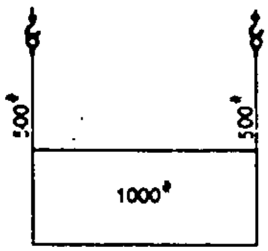


Diagram B

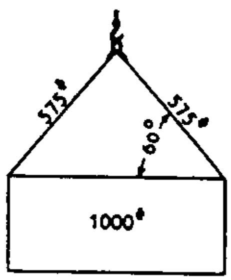


Diagram C

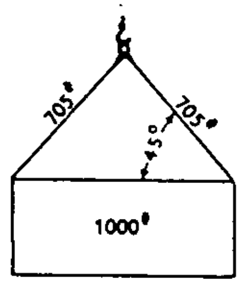


Diagram D

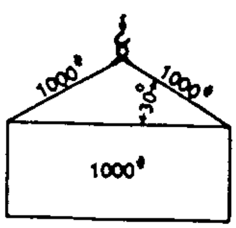


Diagram E

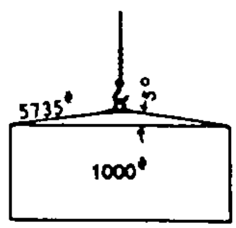


Diagram F

Leschen

The Chart on page 27 may be used in determining the safe load capacity of a given sling or in calculating the proper rope to use for a given load.

The sloping lines correspond to the legs of the sling taken at intervals of 5 degrees. They may be used as shown by the diagram in the upper left hand corner to determine the angles formed by the legs at the ring or link, and with a horizontal plane.

To illustrate this, a triangle is shown in heavy lines, one side of which is detailed as the leg of a sling. The legs form the 60 degree angle "A" at the top, and 2 -60 degree angles "B" at the bottom, between the legs and a horizontal plane.

The angle of spread (at ring or link) is shown in the vertical column A. The figures shown under B are the angles between the legs and a horizontal plane and are the ones used in the calculations. Under C are the factors to be used in determining the safe load capacities and under D the factors used when calculating the diameter of the rope required.

What is the safe load that may be handled with a 2-leg bridle sling made of ⅝" diameter wire rope, where the conditions require that the legs be spread at an angle of 70°?

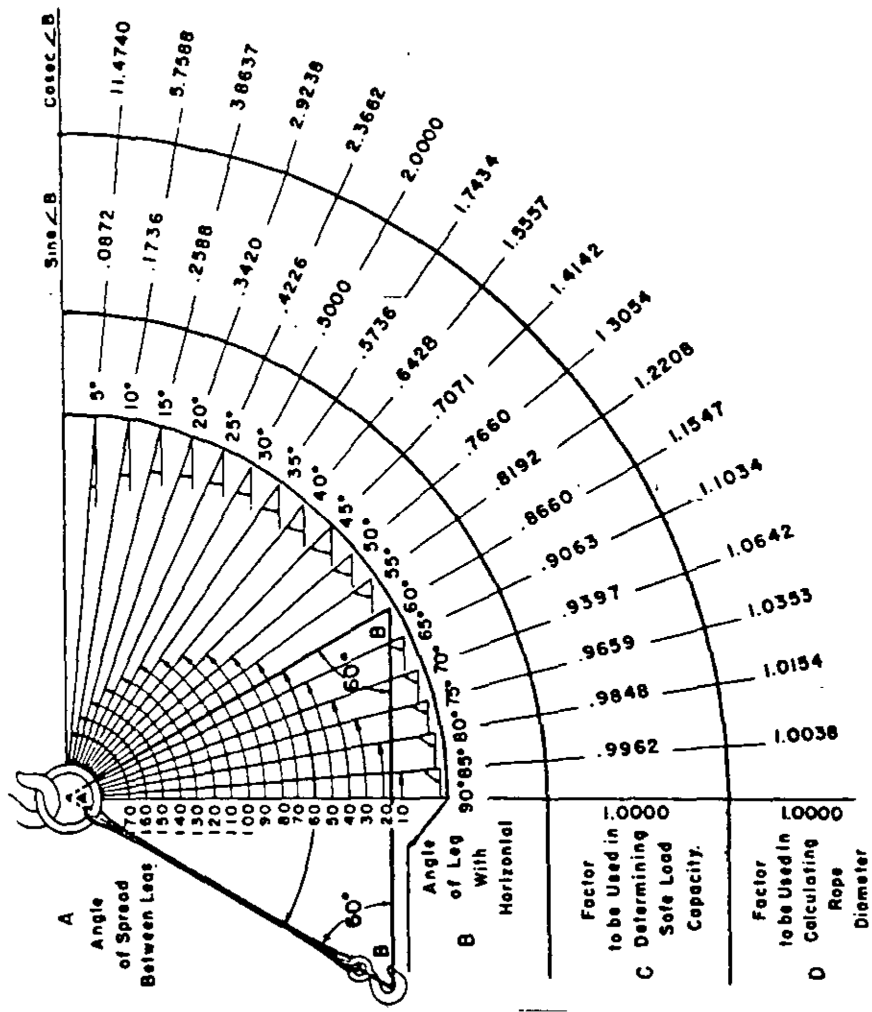
From the chart it will be seen that if the angle of spread between legs is 70°, the angle (B) that each leg makes with the horizontal (when in balance) is 55°. A stress of 2.3 tons could be applied to each leg, and 4.6 tons or 9200 pounds total the sling. To determine the safe load at the required angle the figure 9200 is multiplied by the factor .8192 shown in column C opposite the 55° horizontal angle.

Required safe load W, at given angle of spread =
safe load for two vertical ropes x factor in column
C = 9200 x .8182
7536 pounds

What diameter rope must be used in a two-leg bridle sling to handle 10,000 pounds when legs are spread at an angle of 80°?

As in the first example the chart will show that with the legs spread at an angle of 80° the horizontal angle would be 50° each. One half the load or 5,000 pounds must be carried by each leg.

To calculate the tension developed in each leg by this vertical load of 5,000 pounds multiply the load by the factor in column D opposite the angle of 50°.

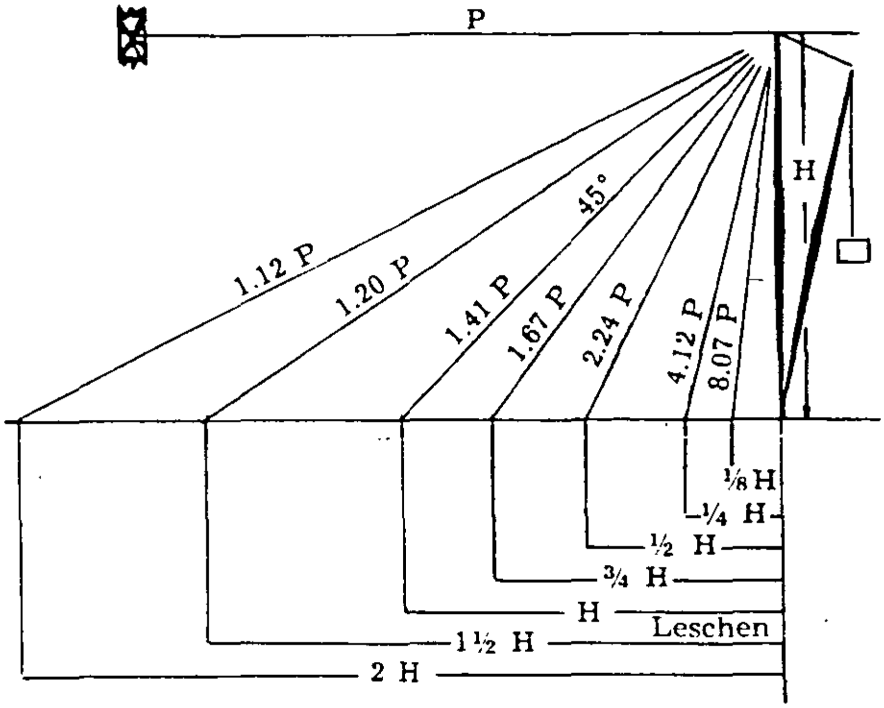


Tension T in one inclined leg = ½ total vertical load X factor in Column D.

= 5,000 pounds X 1.3054
or 6,527 pounds.

Applying a factor of 7, a rope with a breaking strength of 45,689 pounds or 22.8 tons is required. Leschen

GUY STRESSES



In guying a derrick the guys should be installed at as flat an angle with the horizontal as possible, since the stress increases rapidly as the inclination of the guy approaches the vertical. It is not advisable to have the inclination steeper than 45 degrees. Most derrick manufacturers recommend that on level ground the length of the guy rope should be about 3 times the height of the mast.

Referring to the diagram shown here, if for a certain load and position of boom, the stress in a horizontal guy located directly opposite the boom is represented by P, then at 45 degrees the stress is 1.41 P, etc.

In the diagram the value of the stress for each inclination of the guy rope is indicated. Note how rapidly the stress increases as the guy anchorage approaches the base of the mast. When the guy is anchored at a distance from the mast equal to twice the height of the mast, the stress is only 1.12 times the stress in a horizontal guy, but if the guy is anchored at a distance equal to one-half the height of the mast, the stress is equal to about 2¼ times the stress in a horizontal guy.

Table for Figuring Line Parts

Number of Parts of Line	Ratio for Bronze Bushed Sheaves	Ratio for Anti-Friction Bearing Sheaves
1	0.96	.98
2	1.87	1.94
3	2.75	2.88
4	3.59	3.81
5	4.39	4.71
6	5.16	5.60
7	5.90	6.47
8	6.60	7.32
9	7.27	8.16
10	7.91	8.98
11	8.52	9.79
12	9.11	10.6
13	9.68	11.4
14	10.2	12.1
15	10.7	12.9
16	11.2	13.6
17	11.7	14.3
18	12.2	15.0
19	12.6	15.7
20	13.0	16.4
21	13.4	17.0
22	13.8	17.7
23	14.2	18.3
24	14.5	18.9

$$\frac{\text{Total load to be lifted in pounds}}{\text{Single line pull in pounds}} = \text{RATIO}$$

Example one—To find number of parts of line needed where weight of load and single line pull are established.

$$\frac{\text{72,480 lbs. (Load to be lifted)}}{\text{8,000 lbs. (Single line pull)}} = 9.06 \text{ (RATIO)}$$

Refer to ratio 9.06 in Table—indicating 12 parts of line.

Example two—To find single line pull needed when weight of load and number of parts of line are established.

$$\frac{\text{68,000 lbs. (Load to be lifted)}}{\text{6.60 (Ratio of 8 part line)}} = \text{10,300 lbs. (Single line pull)}$$

REEVING BLOCKS AT RIGHTANGLES

When blocks are to be reeved at rightangles to each other, it is very important that the blocks be approximately square in shape as well as in relation to each other. The sheave diameter and the width of the blocks, or distance from center to center of the outside sheaves should correspond, in order to alleviate the scrubbing action of the wire rope on the throat of the sheaves as much as possible. Especially where the blocks have to work close together.

The lead line (#1) as shown on the following pages, concerning reeving blocks at rightangles is laying out at an excessive angle from the vertical center line of the block assemblies, to keep from interfering with the reeving diagrams that it would if it were shown in its near vertical position when working.

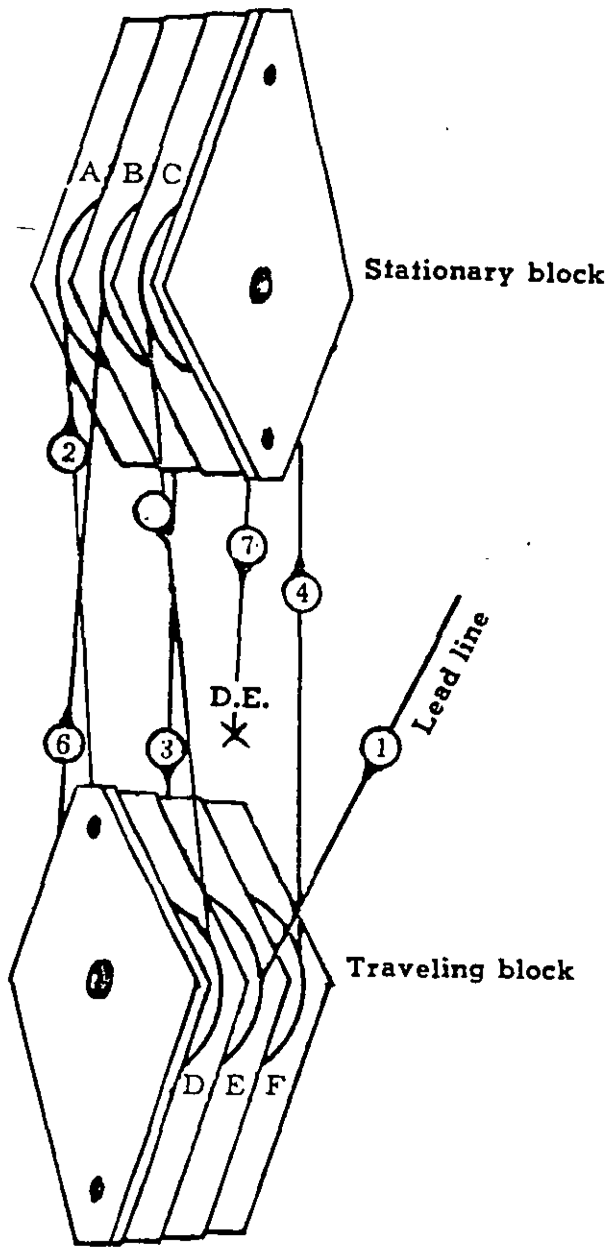
REEVING 6 AND 7 PARTS

This seven part reeve up with the blocks at rightangles using a pair of three sheave blocks is accomplished in the following way.

Enter the lead line from the front of the traveling block at sheave "E", then go up in front of stationary block and through at sheave "A" down behind traveling block and through at sheave "F", up behind stationary block and through at sheave "C", down in front of traveling block and through at sheave "D", up in front of stationary block and through at sheave "B" down to traveling block and becket off for a 7 part reeve up.

If a 6 part reeve up is desired with these same blocks, all that is necessary to accomplish it is, reverse the reeving diagram on the opposite page, which turns the block assembly upside down. You lost the one part that was formed by the lead line entering the traveling block.

In this case, where the lead line enters the stationary block, it does not add a part to the block assembly. The reeving remains exactly the same.


A
B
C
Stationary block
2
7
4
Lead line
D.E.
6
3
1
Traveling block
D
E
F

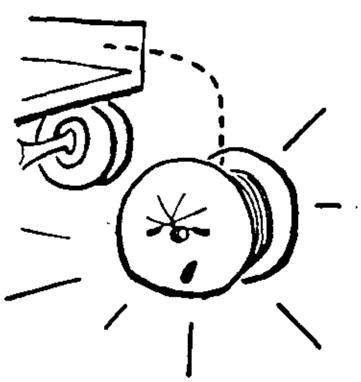
7 Parts

REEVING 7 AND 8 PARTS

This eight part reeve up with the blocks at right angles, using a three and four sheave block, with the four sheave block acting as the traveling block is accomplished as explained in the following reeving instructions.

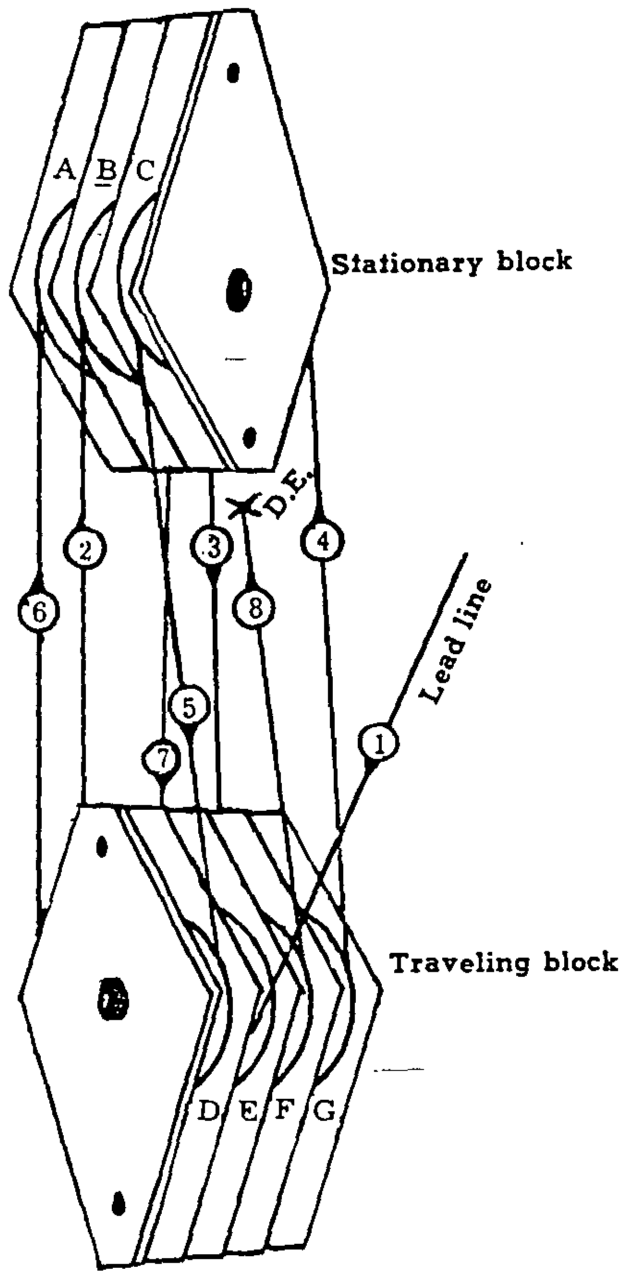
Enter the lead line (#1) from the front of the traveling block at sheave "E", go up in front of stationary block and through at sheave "B" down behind traveling block and through at sheave "G", up behind stationary block and through at sheave "C", down in front of traveling block and through at sheave "D", up in front of stationary block and through at sheave "A", down behind traveling block and through at sheave "F" then up to the stationary block and becket off for an 8 part reeve up.

For a 7 part reeve up, all that is necessary to accomplish it is, turn the diagram on the opposite page upside down, which reverses the blocks and loses the one part of line that was formed by the lead line, which now enters the stationary block. The reeving is exactly the same as for 8 parts.



Don't drop reels from car or truck

Wire rope arrives on the job either in a coil or wound on a reel. When unloading, don't drop the reel. Heavy rope may shift its position, or from its weight alone, cause the reel to collapse.


A
B
C
Stationary block
D.E.
2
3
4
6
8
5
7
1
Lead line
Traveling block
D
E
F
G

8 Parts

REEVING 8 AND 9 PARTS

This 9 part reeve up using a pair of four sheave blocks, as shown on opposite page, is accomplished as follows:

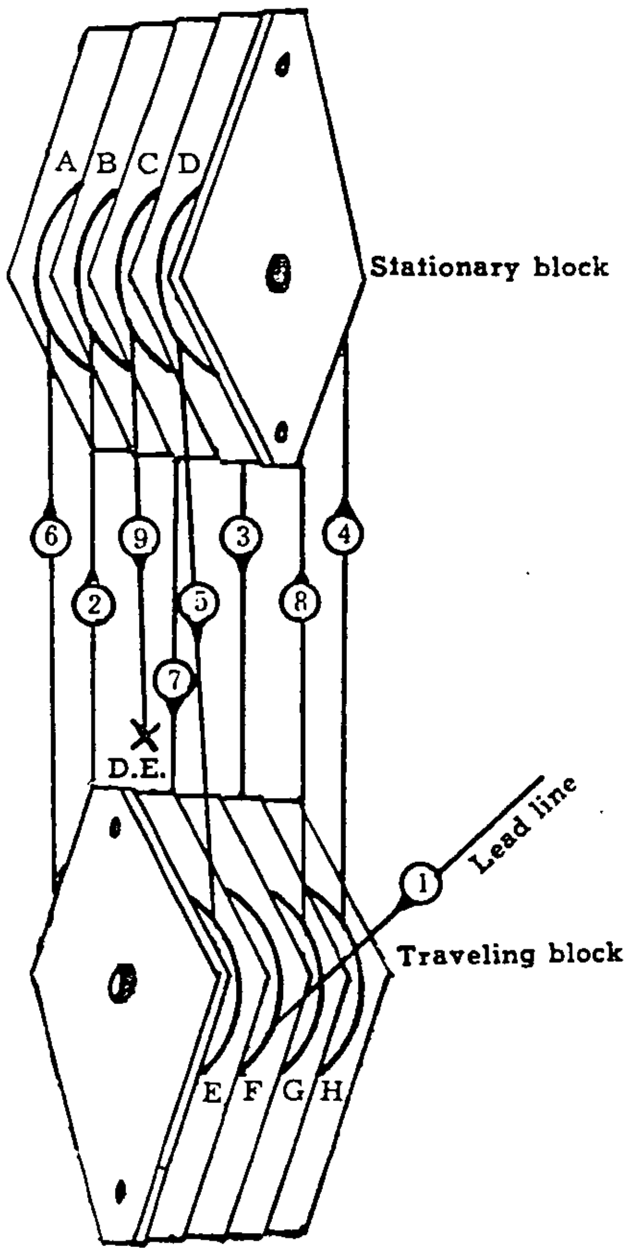
Enter the lead line (#1) from the front of the traveling block at sheave "F", go up in front of stationary block and through at sheave "B" down behind traveling block and through at sheave "H", up behind stationary block and through at sheave "D", down in front of traveling block and through at sheave "E", up in front of stationary block and through at sheave "A", down behind traveling block and through at sheave "G", up behind stationary block and through at sheave "C", then down to the traveling block and becket off for 9 parts.

If an 8 part reeve up is desired using the same blocks, it is accomplished by merely reversing the diagram on the opposite page, in which case the lead line enters the stationary block.

The reeving remains exactly the same, as for 9 parts.

Protect from the weather during storage

When not in use wire rope should be kept clean and dry and protected from excess dust and other elements. If an idle rope remains outdoors for some time, mount it on a reel stand or on heavy timbers and keep it covered. Also, coat the rope with protective Lepro coating that seals out air and moisture.



A
B
C
D
Stationary block
6
9
3
4
2
5
8
7
D.E.
Lead line
1
Traveling block
E
F
G
H

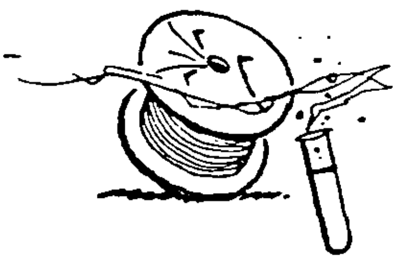
9 Parts

REEVING 10 AND 11 PARTS

An 11 part reeve up using a pair of five sheave blocks set at right angles as shown on the opposite page, is accomplished as shown in the following reeving instructions, in conjunction with the diagram on the next page.

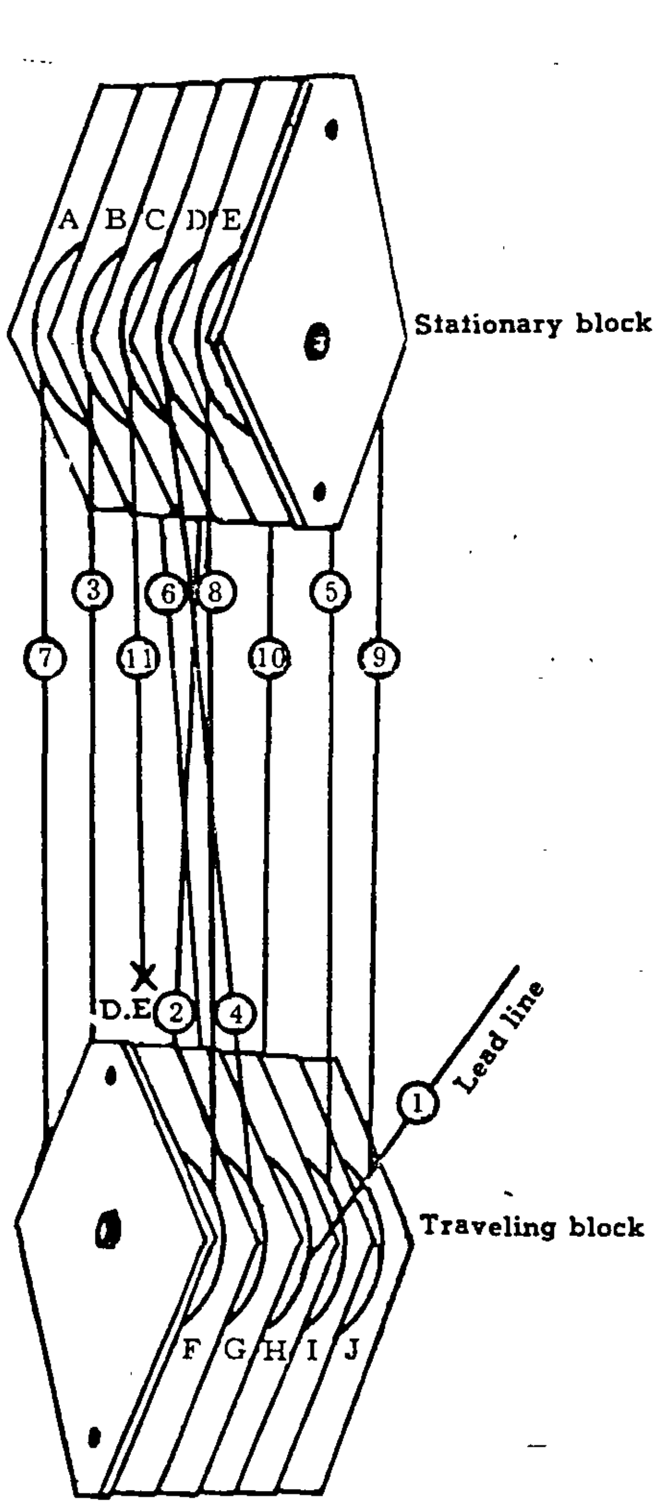
Enter the lead line (#1) at the front of the traveling block at sheave "H", then up behind the stationary block and through at sheave "B", down behind traveling block and through at sheave "G", up in front of stationary block and through at sheave "D", down in front of traveling block and through at sheave "I", up behind stationary block and through at sheave "A", down behind traveling block and through at sheave "F", up in front of stationary block and through at sheave "E", down in front of traveling block and through at sheave "J", up behind stationary block and through at sheave "C", then down to the traveling block and becket off for 11 parts.

For a ten part reeve up with the same blocks, reverse the block assembly so the lead line will enter the stationary block. The reeving remains exactly the same as for 11 parts.



Avoid acid fumes

Do not keep wire rope in a place exposed to acid fumes or other corrosive agents. Acid or acid fumes will tend to harden the wires, make them more brittle, and shorten wire rope life.


A
B
C
D
E
Stationary block
3
6
8
5
7
11
10
9
D.E
2
4
1
Lead line
Traveling block
F
G
H
I
J

11 Parts

REEVING 12 AND 13 PARTS

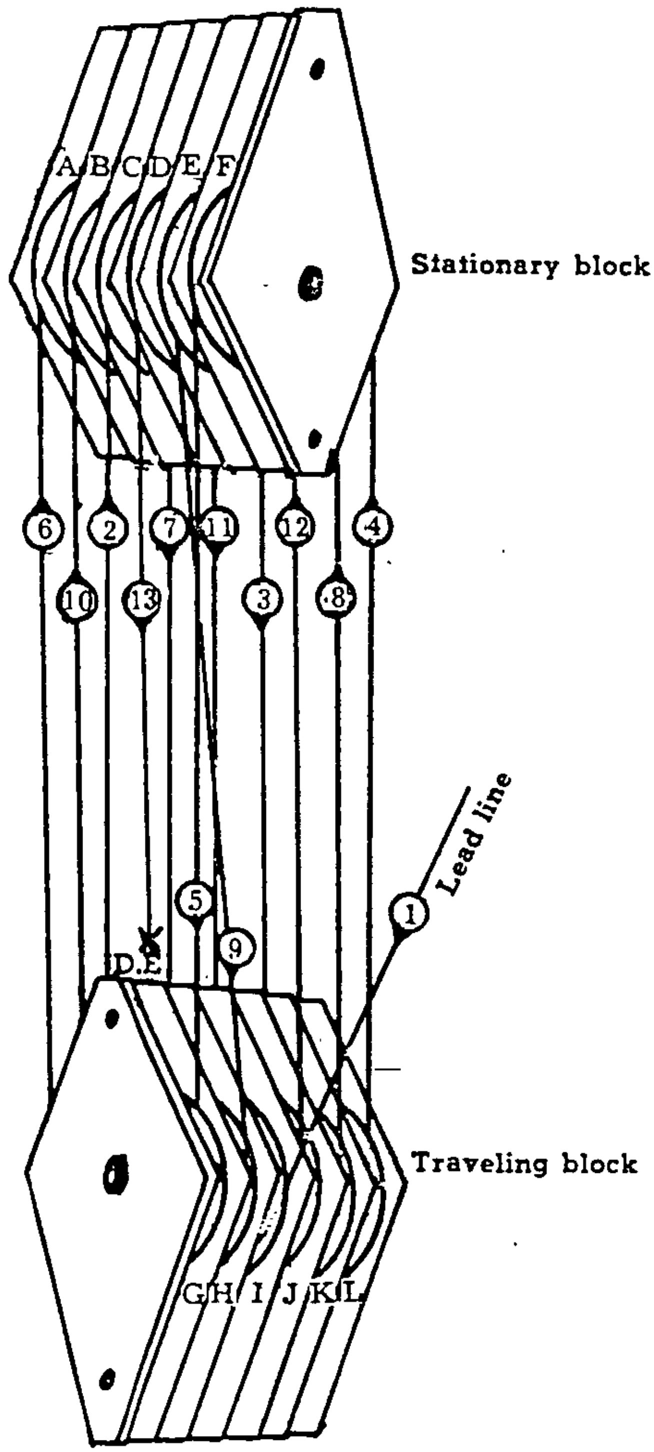
The following reeving instructions, and diagram on the following page, are for a 13 part reeve up using a pair of six sheave blocks at right angles.

Enter the lead line (#1) at the front of the traveling block at sheave "I", go up in front of the stationary block and through at sheave "C", down behind traveling block and through at sheave "L", up behind stationary block and through at sheave "F", down in front of traveling block and through at sheave "G", up in front of stationary block and through at sheave "A", down behind traveling block and through at sheave "K", up behind stationary block and through at sheave "E", down in front of traveling block and through at sheave "H", up in front of stationary block and through at sheave "B", down behind traveling block and through at sheave "J", up behind stationary block and through at sheave "D", then down to the traveling block and becket off for 13 parts.

For a 12 part reeve up with the same blocks, reverse the block assembly so the lead line will enter the stationary block. The reeving remains exactly the same as for the 13 parts.

Maintain proper lubrication

Whether idle or in use, wire rope should always be well lubricated. Lubrication helps protect the wire against the elements and corrosive agents. A wire rope in use is actually a machine, with many moving parts that are constantly causing friction. Lubrication not only reduces internal abrasion caused by this friction, but also promotes rope flexibility.


A
B
C
D
E
F
Stationary block
6
2
7
11
12
4
10
13
3
8
Lead line
1
5
9
D.E
Traveling block
G
H
I
J
K
L

13 Parts

MULTIPLE PART REEVING

The function of multiple part reeving is best exemplified in the enormous lifting power it gives to a hoist or rig, plus the ability to lift these heavy loads with comparatively small diameter wire rope. Increasing the number of parts of line increases the lifting capacity of a hoist or rig in proportion, and at the same time decreases the lead line efficiency. Which ever is the most desirable, or important, can only be had at the expense of the other, whether it is power, or speed and efficiency, they both cannot be had together.

It is obvious that it is much more economical and more efficient to handle extremely heavy loads with multiple part blocks with near the maximum number of parts of comparatively small diameter rope, than to attempt the same operation with very few parts of large diameter rope, in which case the hoist would have to be of such a size in order to have the necessary line pull, that it would be almost prohibitive.

Another thing to always bear in mind, is, a load is never equally distributed on all of the parts of line, from medium size blocks to the maximum sizes, whether you are picking, holding or slacking a load. When picking a load the greatest tension is always on the lead line. On large rigs that have above twenty (20) parts of line, the tension on the lead line can be from three (3) to five (5) times greater than on the becket line, depending entirely on the number of parts of line. When picking a load and stopping and holding it, the lead line tension will still be from 1½ to 2½ times greater than on the becket line. Now reverse this whole operation and slack the load. When slacking a load the greatest tension is always on the becket line in almost the same proportion as mentioned above. When slacking a load and stopping, and holding it, the load will come nearer to equalizing itself on all of the parts of line, but the tension will still be much greater on the becket line.

Always use the recommended number of clips according to line size to secure the becket, otherwise splice an eye to becket off.

The reeving diagrams that are shown on the following pages are, skip reeving straight blocks, tandem blocks and reeving blocks with one or more equalizer sheaves.

I have always been an advocate of reeving blocks with equalizer sheaves or skip reeving straight or tandem blocks in preference to reeving blocks at right angles, as an example, a pair of six (6) sheave blocks reeved at right angles to each other with the lead line entering the traveling block, makes thirteen (13) parts of line which has eleven (11) reverse bends of 90° each after passing the first sheave with each and every successive bend going off 90° in another direction. A pair of straight six (6) sheave blocks in line and skip reeved with the lead line entering the traveling block makes thirteen (13) parts of line which has two (2) reverse bends of 180° each.

It is strictly a matter of choice with the individuals concerned as to what type of reeving they prefer. It is my intention to show all types of blocks, and most every way they can be reeved.

There are scores of methods and procedures used by different people concerning preparations in getting ready to reeve up the many different kinds and sizes of rigs. This I am going to cover very lightly with a few exceptions, and devote the space to the actual reeving of the blocks according to the numerical sequence as shown in the reeving diagrams on the following pages.

On the smaller rigs using one (1) lead line, spool the rope on the drum, and be sure that the rope comes off of the reel in the same manner that it goes on the drum, if the drum overwinds the rope should come off of the top of the reel, if the drum underwinds the rope should come off of the bottom of the reel. This procedure will prevent reverse bending the entire length of rope.

Large blocks should be suspended close together if possible for reeving up purposes, with the exception of some topping lifts. I have had very good success by welding two (2) pad eyes to each block as near to the outside edges as possible, then use two (2) come alongs to suspend them.

If a line is to be replaced and the new line doesn't have an eye spliced in to becket off with, cut the old line leaving the blocks reeved up. Remove the old line from the drum, spool the new line and connect the becket end of the new line to the old line left reeved up in the blocks by any one of the several methods mentioned in this book under END PREPARATION without any enlargement whatever of the rope diameter at this point, then connect a bulldozer, truck or any available equipment to the becket end of the old line and slowly pull the new line through. It is often necessary to fairlead the old line during the pulling operation to keep from kinking the rope on one of the traverse bolts through the block, as well as to keep from pulling the block too far out of line.

If the new line has an eye spliced in to becket off with, and a splicer is available, cut the eye off and use the above procedure then splice another eye for your becket, it is quicker. Always use an eye to becket off with if a splicer is available, otherwise use the required number of clips as recommended for the various sizes of rope.

If a new line comes with an eye spliced in and you don't have a splicer available, and the eye is on the outside, take another reel and respool it, which will put the live end or drum end on the outside cut the becket off of the old line and connect the end of the new line to it as mentioned in the above procedure. Now pull all of the old line off of the drum and cut it long enough to fairlead off of the heel of the boom or hoist, connect it to a bull dozer or anything you have handy and slowly pull it through to where it can be wedged to the drum, then continue the spooling operation with the power of the hoist. Always be sure to wedge the rope on the proper end of the drum according to the lay of the rope, whether it is right or left lay, this is very important.

Be careful at all times when installing a new line, to never allow any portion of it to come in contact with the ground or allow any dirt, sand or abrasive matter to get on the line at any time. And be doubly careful to never allow a kink or dogleg to come in the line as it is impossible to remove it completely, and the wires of the strands will wear excessively at that point, before there is any noticable wear anywhere else.

— END PREPARATION —

Fused End with Link

Seized End with Loop

Leschen

Tapered and Speltered End with Loop

RIGS WITH DOUBLE LEAD LINES:

On the larger rigs that have two (2) lead lines your main fall blocks are merely reeved in the bight of the line. Your hoist drums in most cases will not spool all of the line necessary for the main fall.

Lower the boom and suspend your main fall blocks in line, plumb and square with each other, set the reel containing the lead line on stands, either in front or behind the traveling block. If the lead lines come down in front of the traveling block from the boom point sheaves, set the reel in front, if the lead lines come down behind the traveling block from the boom point sheaves set the reel behind the traveling block.

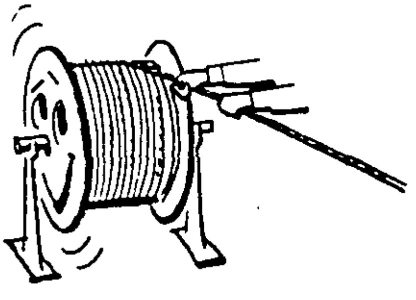
Take the end of the lead line from the reel, up to and over the boom point sheave #0, then back to the hoist drum and spool all of the line that the drum will hold. If the drum will not hold all of the line then plank the ground solid and unwind the remainder of the lead line off of the reel and coil in on the planks, turning every other bight of the line under so it will lay flat until you reach the end, then without untwisting the line at all, start reeving up your blocks according to the reeving diagram and watch your line closely as each bight comes off of the coil on the planks so there won't be either a kink or dog leg thrown in it. If this procedure is followed correctly, there won't be any distortion in the rope or tightening or loosening of the rope lays.

After both ends of the lead line are wedged to the drums, equalize the rope on the two drums and then raise and lower the empty traveling block several times, then use a light load several times for the same purpose. This will give the line a chance to set or become normalized in the position which it will work. This procedure is worth many times the amount that it costs. Use the correct spooling procedure always to avoid reverse bending of the rope.

8 PARTS

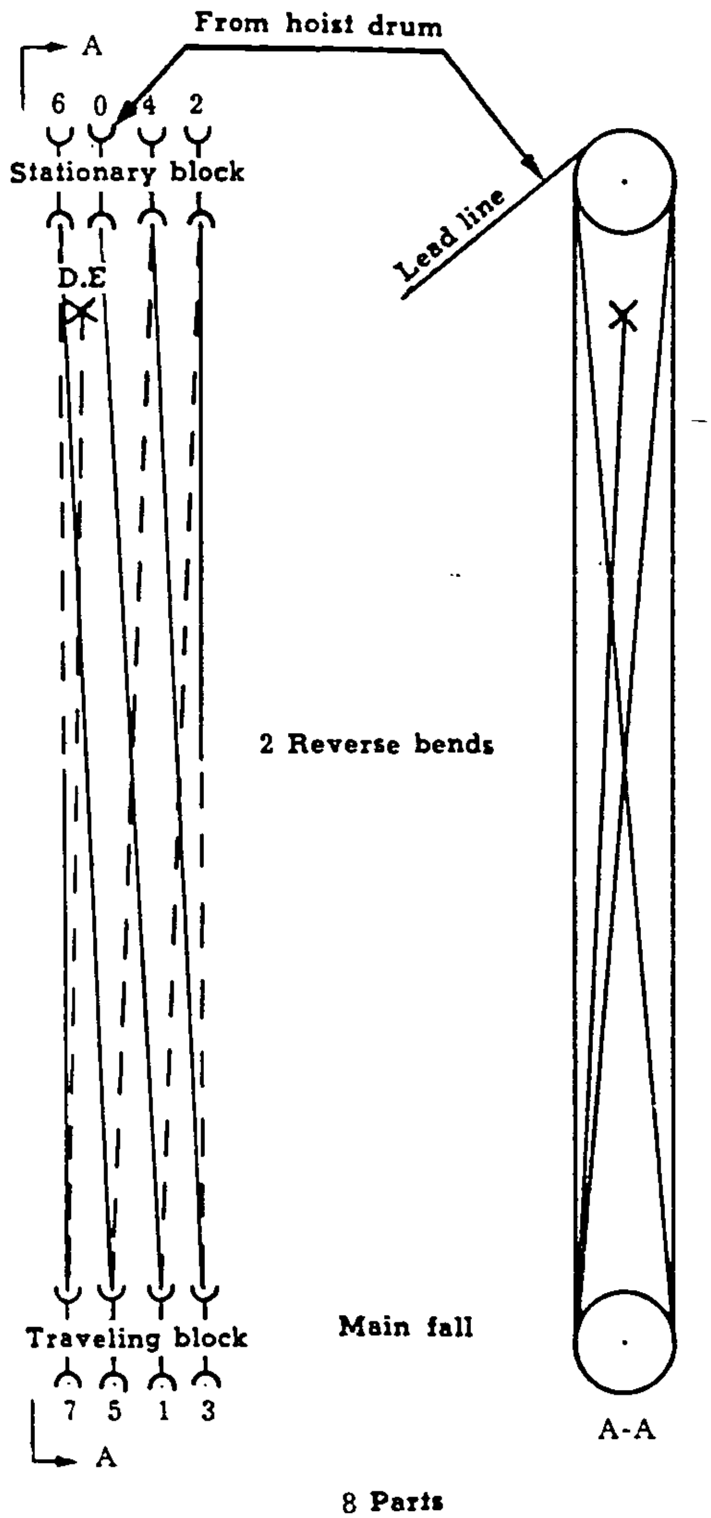
With one (1) four (4) sheave stationary block and one (1) four (4) sheave traveling block, the lead line enters the back of the stationary block and has two (2) reverse bends as shown in the following reeving diagram.

The lead line enters the back of.the stationary block at sheave #0 then down in front of the traveling block and through at sheave #1, up behind the stationary block and through at sheave #2, down behind (making a reverse bend) the traveling block and through at sheave #3, up in front of the stationary block and through at sheave #4, down behind the traveling block and through at sheave #5, up in front of the stationary block and through at sheave #6. down in front (making a reverse bend) of the traveling block and through at sheave #7, then up to the stationary block and becket off.



Unreel and uncoil wire rope carefully

To avoid kinks when unreeling the reel should be mounted on jacks or a turntable so it will revolve as the rope is pulled off. Sufficient tension should be applied to keep slack from accumulating. A board acting as a brake against the reel flange will serve this purpose. Rope in a coil should be unrolled in a straight line away from a man holding the free end.


From hoist drum
A
6 0 4 2
Stationary block
D.E
Lead line
2 Reverse bends
Traveling block
Main fall
7 5 1 3
A
A-A
8 Parts

WIRE ROPE FITTINGS

Fittings should be attached with great care according to the directions given elsewhere, as safety is the most important consideration in operation of all equipment using wire rope. The figures represent the efficiency of the attachment. The approximate percentage of effective rope strength available with each type of fitting depends upon the diameter, construction, and the grade of rope.

Wire Rope Sockets—Spelter Attachment 100%

"Swaged-Sleeve" Thimble Attachment 100%

"Swaged-Sleeve" Loop Attachment 100%

Wedge Sockets — depending on design 80-90%

Clips—(Number of Clips varies with size of Rope)........ 80%

Thimble Splice:

3/8" to 5/8" diam. incl. 90-95%	1¼" to 1½" diam. incl. 80-85%
3/4" to 1 1/8" diam. incl. 85-90%	1 5/8" to 2" diam. incl. 75-80%

2 1/8" and up 70-75%

The efficiency of a loop splice without a thimble is somewhat less than given above for a thimble splice.

Leschen

END PREPARATION
Leschen

Seized End

Tapered and Welded End

Tapered and Speltered End

Tapered and Speltered End with Loop

Fused End with Link

Plain Fused End

Seized End with Loop

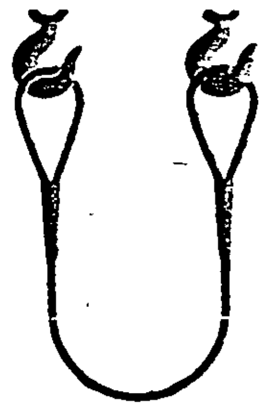
SLING HITCHES IN COMMON USE

A **REGULAR LIFT**

One Leg Vertical. Capacity is 100 pct of single rope

B **CHOKER HITCH**

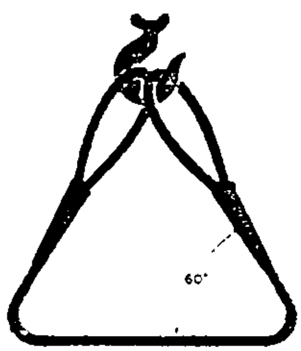
One Leg Vertical. Capacity is 100 pct of single rope



C **BASKET HITCH**

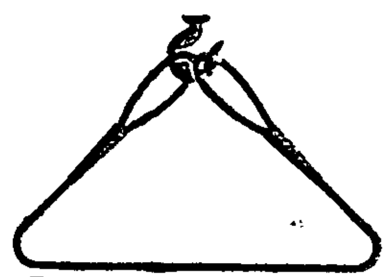
Two Legs Vertical. Capacity is 200 pct of single rope in Regular Lift

Bethlehem



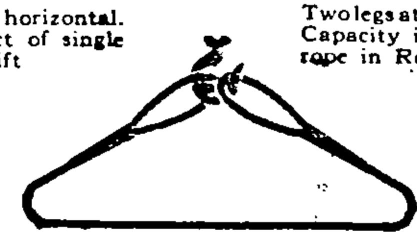
D **BASKET HITCH**

Two legs at 60° with horizontal. Capacity is 174 pct of single rope in Regular Lift



E **BASKET HITCH**

Two legs at 45° with horizontal. Capacity is 142 pct of single rope in Regular Lift



F **BASKET HITCH**

Two legs at 30° with horizontal. Capacity is 100 pct of single rope in Regular Lift

SAFE LOADS

for

Standard Single Wire Rope Slings

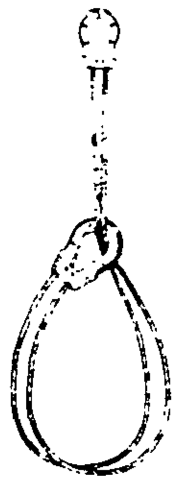
Made of

"HERCULES" (RED-STRAND) WIRE ROPE

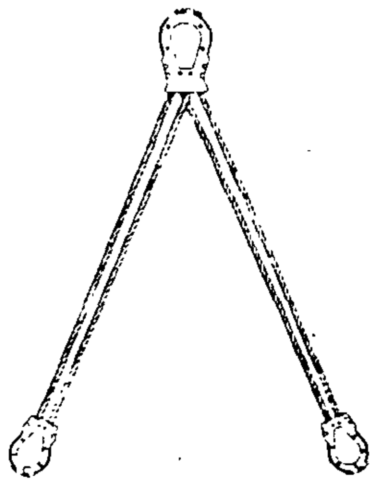
Rope Diameter in Inches	In Tons of 2000 Pounds				
	Vertical Hitch	Anchor Hitch	Basket Hitch		
			Position of Legs		
			 Vertical	 60°	 45°
6x19 Fiber Core					
3/8	.9	.7	1.8	1.5	1.2
1/2	1.5	1.2	3	2.6	2.1
5/8	2.3	1.7	4.6	4	3.2
3/4	3.4	2.8	6.8	5.9	4.8
7/8	4.6	3.8	9.2	7.9	6.5
1	6	4.9	12	10.4	8.5
1 1/8	7.5	6.2	15	13	10.6
6x37 Fiber Core					
1 1/4	8.7	7.2	17.4	15	12.3
1 3/8	10.6	8.7	21.2	18.3	15
1 1/2	12.6	10.4	25.2	21.8	17.8
1 5/8	14.7	12.1	29.4	25.4	20.8
1 3/4	17	14.1	34	29.4	24
1 7/8	19.5	16.1	39	33.8	27.6
2	22.1	18.3	44.2	38.3	31.2

Note 1—The safe loads shown in this table provide a factor of safety—varying with the rope diameters—of from six to seven.
Note 2—The table above is based on an average diminishing splicing efficiency of from 95% to 75%.
Note 3—For slings with wire rope core add 7½% to the safe loads of table above.

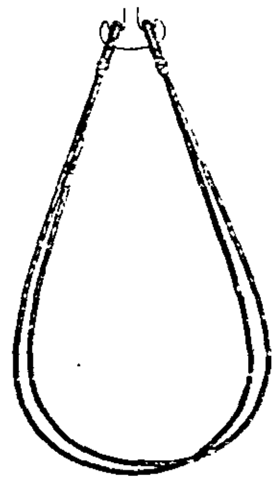
"HERCULES" (RED-STRAND) GROMMET SLINGS
with Equalizing Thimbles



Anchor Hitch



Double Bridle Type



Basket Hitch

Safe Loads in Tons of 2000 Pounds

Diameter of Sling in Inches	Anchor Hitch	Basket Hitch or Two Leg Bridle — Vertical	Basket Hitch or Two Leg Bridle — 60°	Basket Hitch or Two Leg Bridle — 45°
7x19 Construction				
1/4	.7	1.4	1.2	1
3/8	1.5	3	2.6	2.1
1/2	2.6	5.2	4.5	3.7
5/8	4	8	6.9	5.7
3/4	5.6	11.2	9.7	7.9
7/8	7.7	15.4	13.4	10.9
1	10	20	17.4	14.2
1 1/8	12.6	25.2	21.9	17.9
7x37 Construction				
1 1/4	15.5	31	26.9	22
1 3/8	18.7	37.4	32.5	26.5
1 1/2	22	44	38.2	31.2
1 5/8	25.7	51.4	44.6	36.4
1 3/4	29.5	59	51.3	41.8
1 7/8	33.8	67.6	58.7	47.9
2	38.4	76.8	66.7	54.4
2 1/8	43.1	86.2	74.9	61.1
2 1/4	48.1	96.2	83.6	68.2

The safe loads shown in this table provide a factor of safety—varying with the rope diameters—from six to seven.

CHAIN DATA
Bethlehem

WEIGHTS AND BREAKING STRENGTHS OF DREDGE IRON AND HIGH TEST STEEL CHAINS

nominal diam	Dredge Chain (heavy duty)		High Test Steel (high carbon) Chain	
in.	wt per ft, nominal lb	Breaking strength tons	wt per ft, nominal lb	avg Breaking Strength tons
1/4	......	......	0.70	4.27
5/16	......	......	1.10	6.05
3/8	1.67	4.50	1.55	8.55
7/16	2.14	5.75	2.10	11.10
1/2	2.80	7.50	2.65	14.00
9/16	3.55	9.30	3.35	17.50
5/8	4.30	11.55	4.20	20.25
3/4	6.25	16.90	6.00	28.00
7/8	8.30	23.35	8.30	38.25
1	10.65	31.00	10.65	48.50
1 1/8	13.35	39.00	13.35	50.50
1 1/4	16.25	48.00	16.25	60.00
1 3/8	19.60	57.50	19.60	70.50
1 1/2	23.35	68.00	23.35	81.00
1 5/8	27.40	77.50	......	......
1 3/4	31.80	87.50	......	......
1 7/8	36.50	97.50	......	......
2	41.00	111.00	......	......
2 1/8	45.60	125.00	......	......
2 1/4	49.90	141.00	......	......
2 3/8	55.80	156.50	......	......
2 1/2	62.00	174.00	......	......

OPEN WIRE ROPE SOCKETS

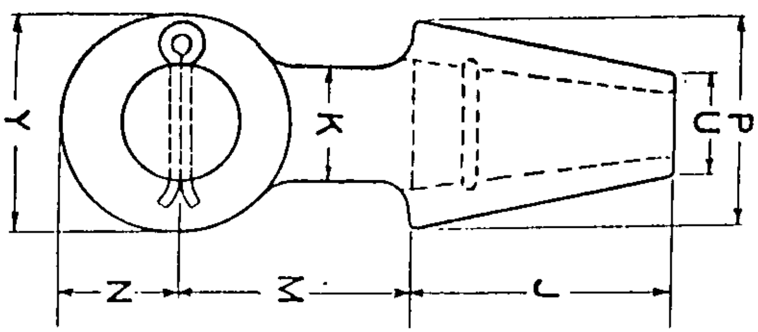
Diam Rope in.	A in.	J in.	K in.	M in.	N in.	O in.	P in.	Q in.	U in.	V in.	Y in.	Pin Length, in.	Pin D, in.	Pin Cotter, in.	Weight approx lb
DROP FORGED STEEL															
3/16	4 5/16	2	3/4	1 9/16	3/4	11/16	1 5/16	5/16	9/16	5/16	1 5/16	1 3/4	11/16	3/16	7/8
1/4	4 5/16	2	3/4	1 9/16	3/4	11/16	1 5/16	5/16	9/16	5/16	1 5/16	1 3/4	11/16	3/16	7/8
5/16	4 5/8	2	13/16	1 3/4	7/8	13/16	1 9/16	3/8	3/4	13/32	1 1/2	2 1/16	13/16	3/16	1 1/8
3/8	4 5/8	2	13/16	1 3/4	7/8	13/16	1 9/16	7/16	3/4	13/32	1 1/2	2 1/16	13/16	3/16	1 1/8
7/16	5 9/16	2 1/2	1	2	1 1/16	1	1 7/8	1/2	15/16	1/2	1 7/8	2 7/16	1	3/16	2 1/4
1/2	5 9/16	2 1/2	1	2	1 1/16	1	1 7/8	9/16	15/16	1/2	1 7/8	2 7/16	1	3/16	2 1/4
9/16	6 3/4	3	1 1/4	2 1/2	1 1/4	1 1/4	2 1/4	5/8	1 1/8	9/16	2 1/4	2 7/8	1 3/16	1/4	3 3/4
5/8	6 3/4	3	1 1/4	2 1/2	1 1/4	1 1/4	2 1/4	11/16	1 1/8	9/16	2 1/4	2 7/8	1 3/16	1/4	3 3/4
3/4	7 15/16	3 1/2	1 1/2	3	1 7/16	1 1/2	2 5/8	13/16	1 1/4	5/8	2 5/8	3 1/4	1 3/8	1/4	6
7/8	9 1/4	4	1 3/4	3 1/2	1 3/4	1 3/4	3 1/8	31/32	1 1/2	3/4	3 1/8	3 7/8	1 5/8	5/16	10
1	10 9/16	4 1/2	2	4	2 1/16	2	3 5/8	1 3/32	1 3/4	7/8	3 3/4	4 1/2	2	3/8	16
1 1/8	11 13/16	5	2 3/8	4 1/2	2 5/16	2 1/4	4	1 7/32	2	1	4 1/8	5	2 1/4	3/8	22
1 1/4	13 3/16	5 1/2	2 3/4	5	2 11/16	2 1/2	4 5/8	1 3/8	2 1/4	1 1/8	4 3/4	5 5/8	2 1/2	7/16	32
1 3/8	13 3/16	5 1/2	2 3/4	5	2 11/16	2 1/2	4 5/8	1 1/2	2 1/4	1 1/8	4 3/4	5 5/8	2 1/2	7/16	32
1 1/2	15 1/8	6	3	6	3 1/8	3	5 1/4	1 5/8	2 3/4	1 3/16	5 3/8	6 3/8	2 3/4	1/2	46
1 5/8	16 1/4	6 1/2	3 1/4	6 1/2	3 1/4	3	5 1/2	1 13/16	3	1 5/16	5 3/4	6 5/8	3	1/2	55
1 3/4	18 1/4	7 1/2	3 7/8	7	3 3/4	3 1/2	6 3/8	1 15/16	3 1/8	1 9/16	6 1/2	7 5/8	3 1/2	1/2	85
1 7/8	18 1/4	7 1/2	3 7/8	7	3 3/4	3 1/2	6 3/8	2 1/16	3 1/8	1 9/16	6 1/2	7 5/8	3 1/2	1/2	85
2	21 1/2	8 1/2	4 1/4	9	4	4	7 3/8	2 3/16	3 3/4	1 13/16	7	8 3/4	3 3/4	1/2	125
2 1/8	21 1/2	8 1/2	4 1/4	9	4	4	7 3/8	2 5/16	3 3/4	1 13/16	7	8 3/4	3 3/4	1/2	125
2 1/4	23 1/2	9	4 3/8	10	4 1/2	4 1/2	8 1/4	2 7/16	4	2 1/8	7 3/4	10	4 1/4	1/2	166
2 3/8	23 1/2	9	4 3/8	10	4 1/2	4 1/2	8 1/4	2 5/8	4	2 1/8	7 3/4	10	4 1/4	1/2	166

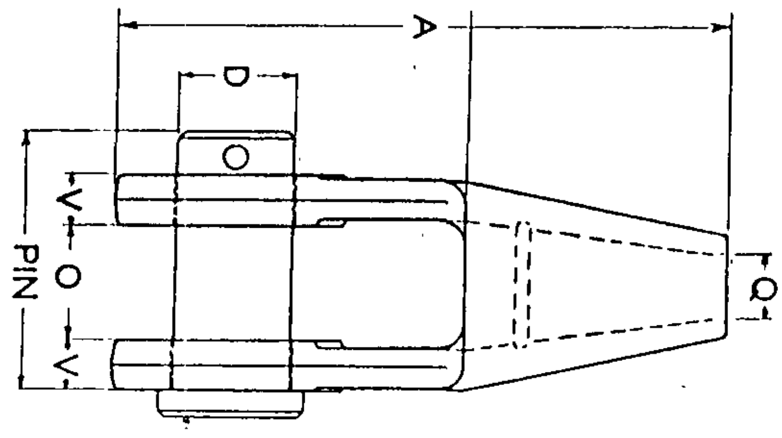
OPEN WIRE ROPE SOCKETS

Bethlehem.

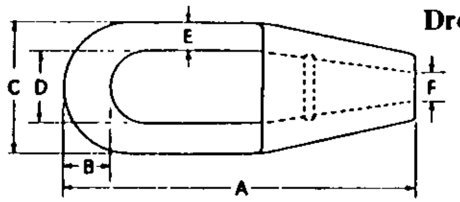
STEEL CASTINGS

2½	26¾	10½	5	11	5¼	5	9	2¾	6⅛	2¼	9	10¾	4¾	⅝	240
2⅝	26¾	10½	5	11	5¼	5	9	2⅞	6⅛	2¼	9	10¾	4¾	⅝	240
2¾	28¾	11½	5¼	11½	5¾	5⅜	10	3	7	2⅜	10	11⅜	5	⅝	305
2⅞	28¾	11½	5¼	11½	5¾	5⅜	10	3⅛	7	2⅜	10	11⅜	5	⅝	305
3	30 9/16	12½	5½	12	6 1/16	5¾	10¾	3¼	7⅝	2½	10½	12¼	5¼	¾	370
3¼	34¾	14	7	14	6¾	6¼	11½	3½	8½	2¾	11½	13¼	5¾	¾	510
3½	36½	15	8	14½	7	7½	13¼	3¾	9¼	3¼	12½	15½	6¾	¾	760
3¾	38¾	16	8¼	15	7¾	7¾	14	4	10	3⅜	14	16	7	¾	890
4	40¼	17	8½	15	8¼	8	14½	4¼	10½	3½	14½	16½	7¼	¾	1020



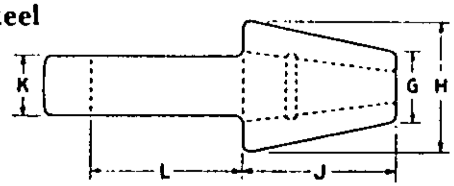


CLOSED WIRE ROPE SOCKETS



Drop Forged Steel

Leschen



The following applies to table on opposite page.

All forged dimensions of less than 4" to have tolerance ± 1/16" except dimensions "B" "G" and "L" which are minimum.

All forged dimensions of 4" and over to have tolerance ± ⅛" except dimensions "B" "G" and "L" which are minimum.

Grooves—Size of Socket	No. of Grooves	Approx. Depth
¼"— ¾"	1	1/16"
⅞"—1½"	2	⅛"
1⅝"—2⅝"	3	3/16"

Basis of Design—Max. Stress in sockets not to exceed 48,000 lbs. per sq. in. when rope used with them is stressed to its listed catalog strength.

Material—A.I.S.I. Grade C-1035—Normalized.

Leschen Sockets conform to Wire Rope Institute standards.

Sizes larger than listed on opposite page are available in steel castings.

Closed Sockets designed especially for use on 19-wire, 37-wire and 61-wire galvanized strand can be supplied. These sockets are made with a longer bowl than standard sockets, so that the fewer wires in the strand will be held definitely in place.

Continued on opposite page

CLOSED WIRE ROPE SOCKETS—Continued from opposite page

All Dimensions in Inches

Rope Diam.	A	B*	C	D	E	F	G*	H	J	K	L*	Approx. Weight Each in Pounds	Approx. Wgt. of Zinc‡ Required in Pounds
1/4	4 1/4	7/16	1 7/16	13/16	5/16	5/16	9/16	1 5/16	2	1/2	1 3/4	1/2	10
5/16	4 5/8	9/16	1 11/16	15/16	3/8	3/8	3/4	1 9/16	2	5/8	2	7/8	.15
3/8	4 5/8	9/16	1 11/16	15/16	3/8	7/16	3/4	1 9/16	2	5/8	2	7/8	17
7/16	5 1/2	11/16	2	1 1/8	7/16	1/2	15/16	1 7/8	2 1/2	7/8	2 1/4	1 1/2	.25
1/2	5 1/2	11/16	2	1 1/8	7/16	5/16	15/16	1 7/8	2 1/2	7/8	2 1/4	1 1/2	.30
9/16	6 3/8	13/16	2 5/8	1 3/8	5/8	5/8	1 1/8	2 3/8	3	1	2 1/2	3	.45
5/8	6 3/8	13/16	2 5/8	1 3/8	5/8	11/16	1 1/8	2 3/8	3	1	2 1/2	3	.55
3/4	7 5/8	1 1/16	3	1 5/8	11/16	13/16	1 1/4	2 3/4	3 1/2	1 1/4	3	4 1/2	90
7/8	8 7/8	1 1/4	3 5/8	1 7/8	7/8	31/32	1 1/2	3 1/4	4	1 1/2	3 1/2	7	1.50
1	10	1 3/8	4 1/8	2 1/4	15/16	1 3/32	1 3/4	3 3/4	4 1/2	1 3/4	4	11	2.25
1 1/8	11 1/8	1 1/2	4 1/2	2 1/2	1	1 7/32	2	4 1/8	5	2	4 1/2	16	3.15
1 1/4	12 5/16	1 5/8	5	2 3/4	1 1/8	1 3/8	2 1/4	4 3/4	5 1/2	2 1/4	5	22	4
1 3/8	12 5/16	1 5/8	5	2 3/4	1 1/8	1 1/2	2 1/4	4 3/4	5 1/2	2 1/4	5	22	4 50
1 1/2	14 1/8	1 13/16	5 3/8	3 1/8	1 1/8	1 5/8	2 3/4	5 1/4	6	2 1/2	6	28	6 65
1 5/8	15 3/8	2 1/8	5 3/4	3 1/4	1 1/4	1 13/16	3	5 1/2	6 1/2	2 3/4	6 1/2	36	7.50
1 3/4	17 1/2	2 3/16	6 3/4	3 17/32	1 1/2	1 15/16	3 1/8	6 3/8	7 1/2	3	7 13/16	58	11
1 7/8	17 1/2	2 3/16	6 3/4	3 17/32	1 1/2	2 1/16	3 1/8	6 3/8	7 1/2	3	7 13/16	58	14
2	19 3/4	2 7/16	7 5/8	3 25/32	1 5/8	2 3/16	3 3/4	7 3/8	8 1/2	3 1/4	8 13/16	80	16.80
2 1/8	19 3/4	2 7/16	7 5/8	3 25/32	1 5/8	2 5/16	3 3/4	7 3/8	8 1/2	3 1/4	8 13/16	80	18
2 1/4	21 5/8	2 5/8	8 1/2	4 9/32	1 3/4	2 7/16	4	8 1/4	9	3 5/8	9 3/4	105	22.50
2 3/8	21 5/8	2 5/8	8 1/2	4 9/32	1 3/4	2 5/8	4	8 1/4	9	3 5/8	9 3/4	105	23.50

*Denotes minimum dimensions.

‡The melting point of zinc is 787°F. Its boiling point is 1724°F. Recommended pouring temperature is about 830°F.

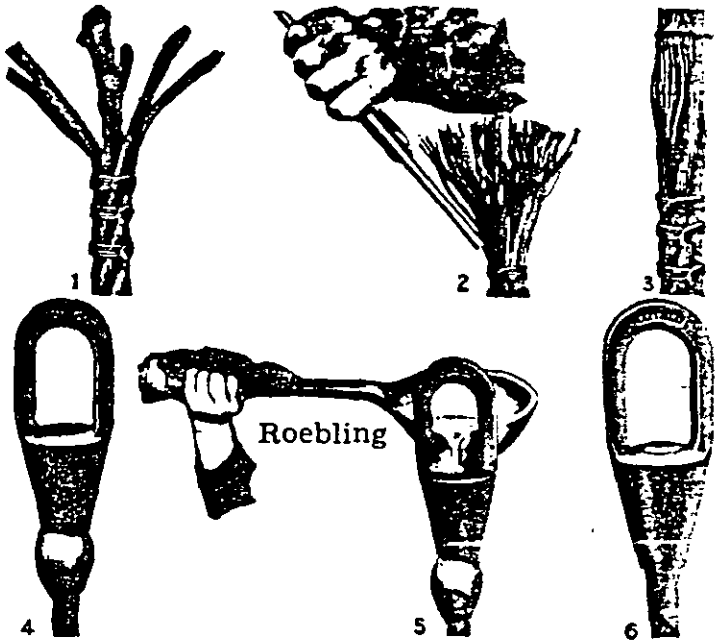
How to Attach a Wire Rope Socket

1. Measure from end of rope a distance equal to length of socket basket. Serve at this point with not less than three seizings. Open up rope. If core is of fiber cut it off as close to first seizing as possible. Do not cut off a metallic core, i.e. independent wire rope core or wire strand core.

2. Wires in main rope strands and also in a metallic core should be separated to form a brush. Partially straightening the wires may be necessary and with larger ropes a pipe may be needed. Cut away any non-metallic material as close to first seizing as possbile. Cleanse all brushed out wires carefully with kerosene or other suitable solvent to as near first seizing as possible. Shake off excess and wipe dry.

Dip wire brush for three quarters of its length into a bath of equal parts of commercial muriatic acid and water. Take extreme care that acid does not touch any other part of rope. The acid bath must be kept clean. Oil or grease on the surface of the bath from previous dippings should be skimmed off. Thorough cleaning is essential to successful socketing. Keep wire brush in acid bath until clean dull gray steel color is observed, usually one-half to one minute. On removing brush from acid, keeping it pointing downward, shake off excess acid thoroughly. Avoid handling brush in an upright position until excess acid has been shaken off. Otherwise, acid may drain into the rope.

3. Put temporary seizing on end of brush so that socket can be slipped over all of the wires.



4. Slip socket over wires. Socket should preferably have been preheated to approximately 200°F to avoid too rapid chilling of the zinc during the pouring operation. Cut temporary top seizing wire and distribute all wires evenly in basket and flush with top. Be sure socket is in line with axis of rope. Place fire clay, putty or asbestos wicking around bottom of socket.

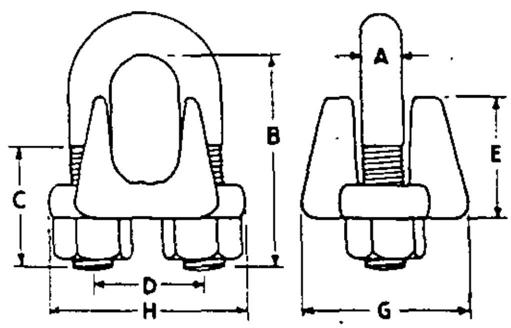
5. Use zinc not lower in quality than "high grade" per A.S.T.M. Specification B-6-49. Do not use babbitt or other anti-friction metal. Heat zinc to an approximate range of 850°F to 900°F keeping toward the low side of the range for smaller ropes and toward the high side for larger ropes. Skim off any dross which may have accumulated on the surface of the zinc bath. Pour pure molten zinc into the socket basket.

6. Remove all seizings. After cooling apply lubricant to rope adjacent to socket to replace lubricant removed by heat of socketing. Socket is then ready for service.

Weight of zinc in standard socket attachment
for fiber core rope

Standard open and closed sockets			
Size Socket	Approx. Weight of Zinc	Size Socket	Approx. Weight of Zinc
1/4"	.140#	1 7/8"	10.00#
3/8"	.185	2	18.00
1/2"	.320	2 1/8"	17.30
5/8"	.580	2 1/4"	21.00
3/4"	.930	2 3/8"	20.3
7/8"	2.00	2 1/2"	30.0
1	2.50	2 5/8"	29.0
1 1/8"	3.25	2 3/4"	38.0
1 1/4"	5.00	2 7/8"	37.0
1 3/8"	4.70	3	49.0
1 1/2"	6.75	3 1/4"	69.0
1 5/8"	8.50	3 1/2"	87.0
1 3/4"	10.50	3 3/4"	101.0

CROSBY WIRE ROPE CLIPS



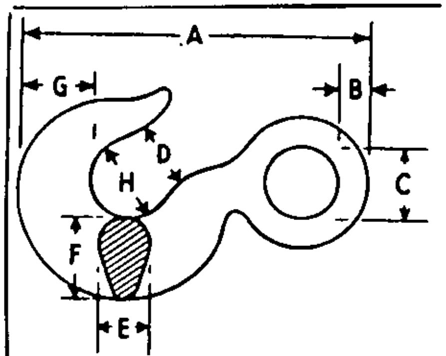
*Drop Forged Steel and Hot-Dip Galvanized

Leschen

*Sizes 2¼-inch and larger are Cast Steel—all other sizes are drop forged.

Diameter of Rope in Inches	Dimensions in Inches A	B	C	D	E	G	H	Min. No. of Clips Required	Net Weight Per 100 Clips in Pounds
1/8	3/16	23/32	7/16	15/32	29/64	13/16	29/32	2	5
3/16	1/4	31/32	5/8	19/32	1/2	15/16	1 1/32	2	9
1/4	5/16	1 1/32	21/32	3/4	21/32	1 3/16	1 7/16	2	18
5/16	3/8	1 13/32	7/8	7/8	23/32	1 5/16	1 11/16	2	30
3/8	7/16	1 5/8	1	1	15/16	1 5/8	1 15/16	2	47
7/16	1/2	2	1 3/16	1 1/8	1 3/32	1 13/16	2 1/4	2	71
1/2	1/2	2	1 3/16	1 3/16	1 1/4	1 29/32	2 9/32	3	73
9/16	9/16	2 3/8	1 1/2	1 5/16	1 11/32	2 1/16	2 31/64	3	101
5/8	9/16	2 3/8	1 1/2	1 5/16	1 3/8	2 1/16	2 1/2	3	101
3/4	5/8	2 3/4	1 9/16	1 1/2	1 9/16	2 5/16	2 13/16	4	157
7/8	3/4	3 3/16	1 3/4	1 3/4	1 3/4	2 11/16	3 1/4	4	242
1	3/4	3 1/2	1 7/8	1 7/8	1 29/32	2 13/16	3 15/32	4	264
1 1/8	3/4	3 7/8	2 3/16	2	2 3/32	3	3 17/32	5	332
1 1/4	7/8	4 1/4	2 5/16	2 5/16	2 9/32	3 1/4	4	5	448
1 3/8	7/8	4 5/8	2 1/2	2 3/8	2 15/32	3 3/8	4 1/16	6	488
1 1/2	7/8	4 15/16	2 5/8	2 19/32	2 23/32	3 17/32	4 5/16	6	544
1 5/8	1	5 5/16	2 13/16	2 3/4	2 7/8	3 5/8	4 3/4	6	702
1 3/4	1 1/8	5 3/4	3 3/16	3 1/16	3 1/8	4 1/16	5 9/32	6	928
2	1 1/4	6 7/16	3 7/16	3 3/8	3 13/32	4 7/16	5 13/16	7	1204
2 1/4	1 1/4	7 1/8	3 13/16	3 7/8	3 15/16	4 9/16	6 3/8	7	1481
2 1/2	1 1/4	7 11/16	4 3/16	4 1/8	4 7/16	4 11/16	6 5/8	8	1660
2 3/4	1 1/4	8 5/16	4 1/2	4 3/8	4 7/8	5	7 1/16	9	2256
3	1 1/2	9 3/16	5	4 3/4	5 11/32	5 5/16	7 5/8	9	3200

HOOKS FOR WIRE ROPE



Leschen
Drop Forged Steel—Weldless

For safe usage see table on page. 108.

Hook No.	Dimensions in Inches A	B	C	D	E	F	G	H	Approx. Ultimate Str. in Lbs.	Approx. Wgt. Each in Lbs.
5	5 1/8	7/16	7/8	1	11/16	1 1/16	15/16	1 1/4	9500	1
10	6 1/8	1/2	1 1/8	1 5/16	3/4	1 3/8	1 3/16	1 5/8	13000	1 6
15	6 19/32	9/16	1 3/16	1 5/16	7/8	1 1/2	1 5/16	1 5/8	17000	2 1
20	6 7/8	5/8	1 1/4	1 5/16	1	1 5/8	1 3/8	1 5/8	21000	2.5
25	7 11/16	11/16	1 3/8	1 1/2	1 1/16	1 7/8	1 9/16	2	25000	3.5
30	8 7/8	13/16	1 3/4	1 5/8	1 1/4	2	1 3/4	2	32000	5 1
40	10 11/32	7/8	2	1 7/8	1 7/16	2 5/16	2	2 5/16	45000	8
50	11 3/4	1	2 1/4	2 1/8	1 5/8	2 5/8	2 5/16	2 5/8	54000	12
65	13 3/8	1 5/16	2 1/2	2 7/16	1 13/16	2 15/16	2 9/16	2 15/16	69000	18.5
75	14 1/2	1 7/16	2 3/4	2 9/16	2 1/4	3 1/8	2 3/4	3 1/8	90000	25
90	16 7/16	1 11/16	3	3	2 5/8	3 5/8	3	3 3/8	115000	38
110	17 7/16	1 13/16	3 1/4	3 5/16	2 3/4	3 13/16	3 1/8	3 5/8	140000	44
125	19 1/4	2	2 1/2	3 3/4	3	4 1/4	3 1/2	4 1/16	170000	59
150	21 3/4	2 1/8	4	4 1/2	3 1/2	4 3/4	4	4 3/4	200000	78
175	25 1/4	2 3/8	4 1/2	5	4	5 3/4	4 3/4	5 3/4	270000	129
175SP	25 1/4	2 3/8	4 1/2	5	4	5 3/4	4 3/4	5 3/4	325000	129

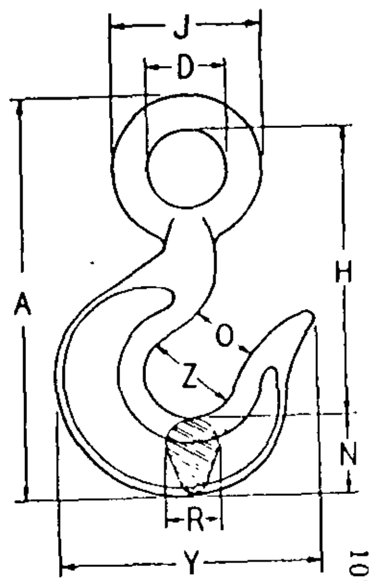
Eye Hoist Hooks

Bethlehem

Drop Forged Steel

DATA AND DIMENSIONS

Size No.	Safe Load tons	A in.	D in.	H in.	J in.	N in.	O in.	R in.	Y in.	Z in.	Weight lb
22	0.6	4 3/8	3/4	3 1/4	1 1/2	3/4	1	9/16	2 7/8	1 1/4	0.5
23	.7	4 15/16	7/8	3 21/32	1 3/4	27/32	1 1/16	5/8	3 3/16	1 3/8	.75
24	.85	5 13/32	1	4	2	29/32	1 1/8	11/16	3 15/32	1 1/2	1
25	1.5	6 1/4	1 1/8	4 9/16	2 1/4	1 1/8	1 1/4	13/16	4 3/32	1 5/8	1.5
26	2.1	6 7/8	1 1/4	4 15/16	2 1/2	1 5/16	1 3/8	15/16	4 17/32	1 3/4	2.25
27	2.6	7 5/8	1 3/8	5 9/16	2 3/4	1 3/8	1 1/2	1 1/8	4 7/8	2	3
28	3.1	8 19/32	1 1/2	6 3/16	3 1/8	1 19/32	1 11/16	1 1/4	5 3/4	2 1/4	4.75
29	3.7	9 1/2	1 5/8	6 15/16	3 3/8	1 11/16	1 7/8	1 3/8	6 3/8	2 1/2	6
30	5	10 11/32	1 3/4	7 15/32	3 5/8	1 15/16	2 1/16	1 1/2	7	2 3/4	8.5
31	5.8	11 27/32	2	8 19/32	4 1/4	2 1/8	2 1/4	1 5/8	7 7/16	3	11.25
32	6.8	13 9/32	2 3/8	9 9/16	4 7/8	2 15/32	2 1/2	1 3/4	8 7/16	3 1/4	16
33	8.5	14 13/16	2 3/4	10 3/4	5 1/2	2 11/16	3	1 7/8	9 3/8	3 3/4	20.3
34	10.6	16 13/16	3 1/8	12 3/16	6 3/8	3	3 3/8	2 1/4	11	4 1/4	33
34A	12.5	17 5/8	3 1/4	12 7/8	6 1/2	3 1/8	3 5/8	2 3/8	11 7/8	4 1/2	41
35	15	19 1/16	3 1/2	14 1/16	7	3 1/4	4	2 5/8	13	5	50
35 1/2	19	19 1/4	3 1/2	13 3/4	7 1/2	3 1/2	3 3/4	2 7/8	12	4 1/16	59
36A	25	22 9/16	4	15 15/16	8 1/2	4 3/8	4 1/2	2 3/4	15	5 3/4	86
37	22.5	21 3/4	4	15 5/8	8 1/4	4	4 1/2	3 1/4	13 13/16	4 3/4	78
38	30	25 1/4	4 1/2	18 1/8	9 1/4	4 3/4	5	3 5/8	16	5 3/4	129



Hook Sizes for 6x19 and 6x37 Wire Ropes

— with —

Fiber Core or Wire Rope Core

Leschen

Diameter of Rope in Inches	HOOK NUMBER For Thimble Splice Connection			HOOK NUMBER For Socket Connection		
	"HERCULES" (Red-Strand) Wire Rope	Plow Steel Wire Rope	Mild Plow Steel Wire Rope	"HERCULES" (Red-Strand) Wire Rope	Plow Steel Wire Rope	Mild Plow Steel Wire Rope
1/4	5	5	5	5	5	5
5/16	10	5	5	10	5	5
3/8	15	10	10	15	10	10
7/16	20	15	10	20	15	15
1/2	25	20	15	25	20	20
9/16	30	25	20	30	25	25
5/8	30	30	25	40	30	30
3/4	40	40	40	50	40	40
7/8	50	50	40	65	65	50
1	65	65	50	75	75	65
1 1/8	75	75	65	90	90	75
1 1/4	90	90	75	110	90	90
1 3/8	110	90	90	125	110	110
1 1/2	125	110	110	150	125	125
1 5/8	125	125	110	175	150	125
1 3/4	150	125	125	175	175	150
1 7/8	175	150	150	175SP	175	175
2	175	175	150	175SP	175SP	175
2 1/8	175	175	...	......	......	...
2 1/4	175SP	175	...	......	......	...
2 1/2	......	175SP	...	......	......	...

SIZES OF EYE HOIST HOOKS FOR THIMBLE AND SOCKET ATTACHMENTS TO WIRE ROPE

Bethlehem

Diam of Rope	Size Nos. of Hoist Hooks for use with Ropes made of			
	Plow Steel		Improved Plow Steel	
	using attachment to wire rope by			
in.	Thimble	Socket	Thimble	Socket
1/4	22	22	22	22
5/16	23	24	24	24
3/8	25	25	25	25
7/16	25	25	25	26
1/2	26	26	26	28
9/16	27	27	27	28
5/8	28	28	28	29
3/4	30	30	30	30
7/8	30	31	31	32
1	32	33	33	33
1 1/8	33	34	33	34
1 1/4	34	35	34	35
1 3/8	34	35	35	35
1 1/2	35	35 1/2	35	35 1/2
1 5/8	35	35 1/2	35 1/2	37
1 3/4	35 1/2	37	35 1/2	36A
1 7/8	35 1/2	36A	37	38
2	37	38	36A	....
2 1/8	37	....	36A	....
2 1/4	36A	....	38	....
2 1/2	38	....	....	....

Tables figured for wire ropes of the 6 x 19 Class.
Sockets develop catalog strength of rope.
Thimble efficiency varies with the rope diameter.

SHACKLES—ANCHOR PATTERN

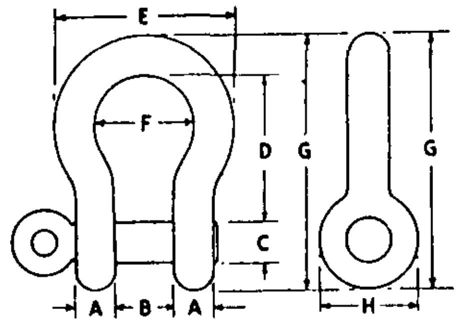
Drop Forged Steel—Weldless

All Dimension in Inches								*Minimum Breaking Load in Pounds	Approximate Wgt. Each in Pounds
A	B	C	D	E	F	G	H		
1/4	31/64	5/16	1 1/8	1 9/32	25/32	1 7/8	11/16	3,550	.12
5/16	17/32	3/8	1 7/32	1 15/32	27/32	2 1/8	13/16	5,300	.18
3/8	21/32	7/16	1 7/16	1 25/32	1 1/32	2 17/32	31/32	7,950	.30
7/16	23/32	1/2	1 11/16	2 1/32	1 5/32	2 29/32	1 1/16	10,850	.49
1/2	13/16	5/8	1 7/8	2 5/16	1 5/16	3 11/32	1 5/16	14,150	.74
5/8	1 1/16	3/4	2 13/32	2 15/16	1 11/16	4 3/16	1 9/16	22,100	1.44
3/4	1 1/4	7/8	2 27/32	3 1/2	2	4 31/32	1 7/8	31,800	2.16
7/8	1 7/16	1	3 5/16	4 1/32	2 9/32	5 3/4	2 1/8	43,250	3.37
1	1 11/16	1 1/8	3 3/4	4 11/16	2 11/16	6 1/2	2 3/8	56,550	5.26
1 1/8	1 27/32	1 1/4	4 1/4	5 5/32	2 29/32	7 5/16	2 5/8	66,800	7 03
1 1/4	2 1/32	1 3/8	4 11/16	5 3/4	3 1/4	8 1/8	3	82,500	9.55
1 3/8	2 1/4	1 1/2	5 1/4	6 3/8	3 5/8	9	3 5/16	99,800	12.57
1 1/2	2 3/8	1 5/8	5 3/4	6 7/8	3 7/8	9 7/8	3 5/8	118,700	17.25
1 3/4	2 7/8	2	7	8 1/2	5	11 7/8	4 5/16	161,600	27.75
2	3 1/4	2 1/4	7 3/4	9 3/4	5 3/4	13 3/8	5	211,100	41.12
2 1/4	3 3/4	2 1/2	9 1/4	11	6 1/2	15 5/8	5 1/4	270,000	56.50
2 1/2	4 1/8	2 3/4	10 1/2	12 1/4	7 1/4	17 3/8	6	338,000	83.50
ƒ2 3/4	4 1/2	3	11 1/2	12 5/8	7 1/8	18 3/4	6	405,000	115
ƒ3	5	3 1/4	13	13 7/8	7 7/8	20 7/8	6 1/2	485,000	145

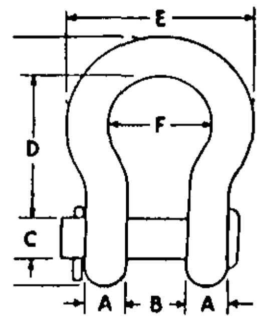
Leschen

ƒFurnished in Round Pin only.

*The Minimum Breaking Loads shown are in accordance with Bureau of Ships ad interim Specifications 42 C 19 (INT) Amendment 1 of July 1, 1945.



Screw Pin
Galvanized



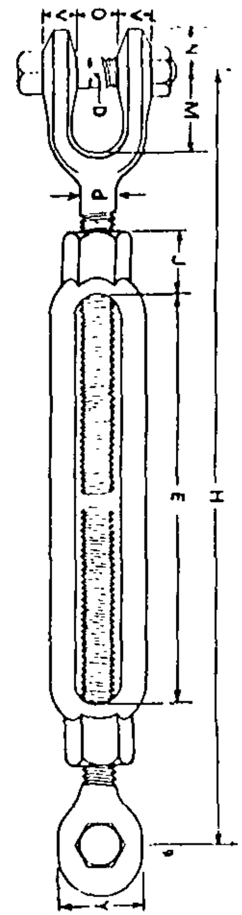
Round Pin
Self Colored

TURNBUCKLES
Jaw and Jaw

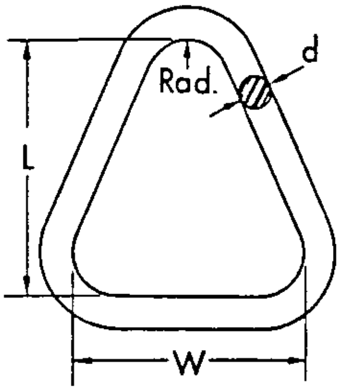
Bethlehem

Drop Forged Steel—Weldless

Size		DATA AND DIMENSIONS								wt approx	Breaking strength
diam d	Take-up E	D	H	J	M	N	O	V	Y		
in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	lb	tons
1/4	4	1/4	7 3/8	7/16	3/4	3/8	13/32	9/32	5/8	0.35	0.80
5/16	4 1/2	1/4	8 3/4	1/2	1	3/8	15/32	9/32	11/16	.58	1.35
3/8	6	5/16	10 11/16	19/32	1 1/32	7/16	1/2	5/16	13/16	1.08	2
1/2	† 9	3/8	14 5/8	3/4	1 1/4	9/16	5/8	13/32	1	2.28	3.75
5/8	† 9	1/2	16 11/16	1 1/8	1 9/16	25/32	3/4	1/2	1 5/16	3.47	6
3/4	‡12	5/8	20 9/16	1 1/8	1 13/16	31/32	15/16	9/16	1 5/8	6.33	9
7/8	*12	3/4	22 3/16	1 5/16	2 1/8	1 3/32	1 1/8	11/16	1 7/8	10.1	12.5
1	*12	7/8	23 11/16	1 1/2	2 1/2	1 7/32	1 3/16	25/32	2 1/8	12.9	16.5
1 1/4	†18	1 1/8	33 11/16	2	3 3/8	1 17/32	1 3/4	1	2 5/8	28.1	26.5
1 1/2	†18	1 3/8	34 7/8	2 1/4	3 1/2	1 13/16	2 1/16	1 5/32	3 1/4	44.5	39
1 3/4	†24	1 5/8	43 7/8	2 5/8	4 3/16	2 1/16	2 3/8	1 1/4	3 1/2	64.6	52.5
2	*24	1 7/8	46 11/16	3	4 11/16	2 17/32	2 1/2	1 9/16	4 3/16	89.1	69
2 1/2	*24	2 1/4	51 5/16	3 3/4	5 9/16	3 15/32	2 7/8	1 7/8	5 5/8	161	112
2 3/4	*24	2 3/4	54 1/4	4 1/8	5 9/16	4	3 1/2	1 5/8	6 1/8	216	147



TRIANGULAR LINK

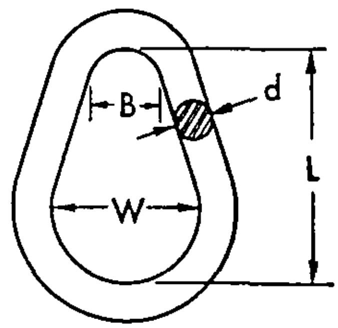


Bethlehem

DATA AND DIMENSIONS

Size No.	Dimensions d in.	W in.	L in.	Rad. in.	Safe Load lb	wt lb
32 T	¾	7	8	1	3,200	3.5
60 T	⅞	7	8	1	6,000	4.75
90 T	1	7½	8	1¼	9,000	6.4
130 T	1¼	7½	8	1¼	13,000	10.5
170 T	1½	8	9	1½	17,000	16.5
220 T	1¾	8	9¼	1⅝	22,000	24
300 T	2	8½	9½	2	30,000	32
380 T	2¼	8½	9½	2	38,000	42
470 T	2½	9	10	2½	47,000	54
560 T	2¾	9	12	2¾	56,000	76
680 T	3	9	12	3	68,000	92
780 T	3¼	9½	12	3	78,000	110
900 T	3½	11	14	3½	90,000	145
1000 T	3¾	12	14	4	100,000	172
1080 T	4	14	16	4	108,000	218
1440 T	4½	15	17	4½	144,000	296
1880 T	5	16	18	5	188,000	390

PEAR SHAPED LINK



Bethlehem

DATA AND DIMENSIONS

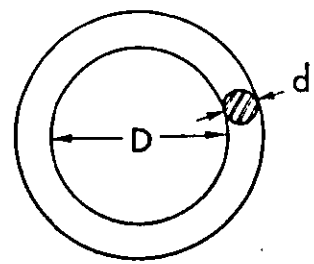
Size No.	Dimensions d in.	B in.	W in.	L in.	Safe Load lb	wt lb
7 P	⅜	¾	1¼	2½	700	¼
15 P	½	1	2	3	1,500	½
22 P	⅝	1¼	2¼	4	2,200	1
32 P	¾	1½	3	5	3,200	2
42 P	⅞	1¾	3½	5½	4,200	3
54 P	1	2	4	6	5,400	4¼
85 P	1¼	2½	5	8	8,500	8½
120 P	1½	3	6	9	12,000	14
170 P	1¾	3½	7	10	17,000	21¾
220 P	2	4	8	12	22,000	33¼
300 P	2¼	4½	9	14	30,000	48½
360 P	2½	5	10	15	36,000	65
440 P	2¾	5½	11	16	44,000	86
520 P	3	6	12	18	52,000	112
600 P	3¼	6½	13	19	60,000	140
750 P	3½	7	14	21	75,000	177
840 P	3¾	7½	15	22½	84,000	222
1000 P	4	8	16	24	100,000	296
1260 P	4½	9	18	27	126,000	377
1560 P	5	10	20	30	156.000	526

Bethlehem

DATA AND DIMENSIONS

Size No.	Dimensions d in.	Dimensions D in.	Safe Load lb	wt approx lb	Size No.	Dimensions d in.	Dimensions D in.	Safe Load lb	wt approx lb
10	½	2½	1,000	½	180	2	8	18,000	28
16	⅝	2¾	1,600	1	230	2¼	9	23,000	40
25	¾	3	2,500	1½	320	2½	9	32,000	50
35	⅞	4	3,500	2½	380	2¾	10	38,000	68
45	1	4	4,500	3½	480	3	10	48,000	82
70	1¼	5	7,000	7	540	3¼	12	54,000	113
100	1½	6	10,000	12	640	3½	12	64,000	133
140	1¾	7	14,000	19	700	3¾	14	70,000	175
					760	4	15	76,000	213

RING

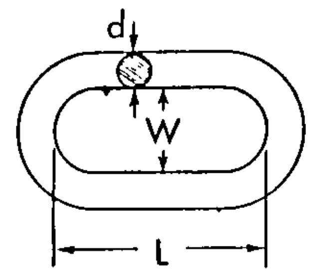


Bethlehem

DATA AND DIMENSIONS

Size No.	Dimensions d in.	W in.	L in.	Safe Load lb	wt approx lb
7	3/8	1	2½	700	¼
15	½	1¼	3	1,500	½
22	5/8	1¾	4	2,200	1
32	¾	2	5	3,200	2
42	7/8	2½	5	4,200	2¾
54	1	3	6	5,400	4
85	1¼	3½	8	8,500	8½
120	1½	4½	9	12,000	14
170	1¾	5	10	17,000	22
220	2	6	12	22,000	33
300	2¼	6	14	30,000	47
360	2½	7	14	36,000	61
440	2¾	8	15	44,000	81
520	3	8	16	52,000	102
600	3¼	9	18	60,000	133
750	3½	9	18	75,000	156

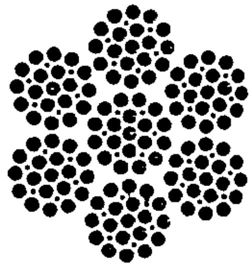
LINK

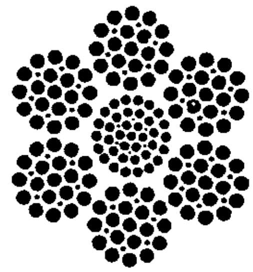


6 x 19 CLASS WIRE ROPE

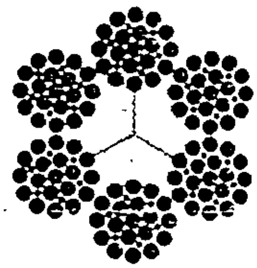
6 Strands—Nominally 19 Main Wires per Strand

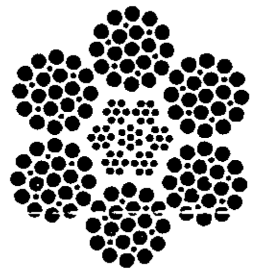
Purple Strand (Improved Plow Steel), Plow Steel Grade
Regular or Lang Lay—Fiber Core or IWRC or WSC



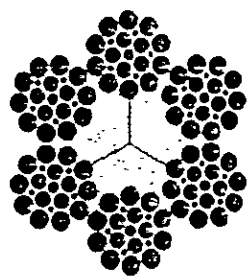


6 x 25 filler wire Type W construction
with WSC (1 x 25) with WSC (1 x 43)

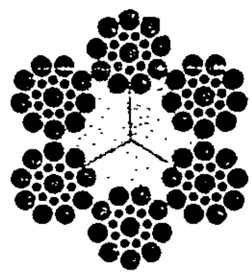




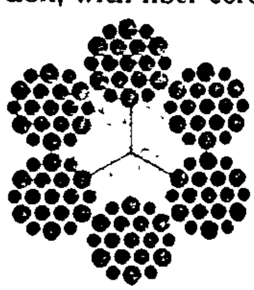
6 x 25 filler wire Type W construction
with fiber core with IWRC



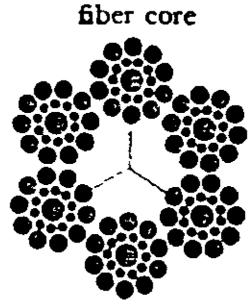
6 x 21 filler wire Type U construction, with fiber core



6 x 19 Seale construction, with fiber core



6 x 19 Warrington with fiber core



6 x 21 Seale construction, with fiber core

6 x 19 CLASS WIRE ROPE

6 x 25 filler wire Type W rope with fiber core or IWRC

6 x 19 Class—6 Strands, Nominally 19 Main Wires per Strand

Purple Strand (Improved Plow Steel)

Regular Lay or Lang Lay—Fiber Core or IWRC

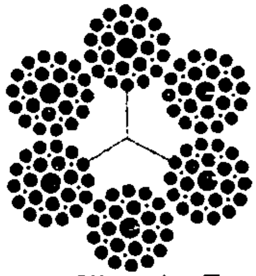
Data below apply to the following constructions: 6 x 25 filler wire Type W, 6 x 21 filler wire Type U, 6 x 19 Warrington, 6 x 19 Seale, 6 x 21 Seale, 6 x 17 Seale.

Diam	Circum approx	Weight per ft approx	Weight per ft approx	min Breaking Strength in tons of 2,000 lb		
		Fiber Core	IWRC WSC	Improved Plow Steel	Plow Steel	IWRC or WSC Improved Plow Steel
in.	in.	lb	lb	Fiber Core	Fiber Core	
¼	¾	0.10		2.74	2.39	
5/16	1	.16		4.26	3.71	
⅜	1⅛	.23	0.25	6.10	5.31	6.56
7/16	1⅜	.31	.34	8.27	7.19	8.9
½	1⅝	.40	.44	10.7	9.35	11.5
9/16	1¾	.51	.56	13.5	11.8	14.5
⅝	2	.63	.70	16.7	14.5	18.0
¾	2⅜	.90	.99	23.8	20.7	25.6
⅞	2¾	1.23	1.36	32.2	28.0	34.6
1	3⅛	1.60	1.76	41.8	36.4	45.0
1⅛	3½	2.03	2.24	52.6	45.7	56.6
1¼	3⅞	2.50	2.75	64.6	56.2	69.5
1⅜	4⅜	3.03	3.34	77.7	67.5	83.5
1½	4¾	3.60	3.96	92.0	80.0	98.9
1⅝	5⅛	4.23	4.66	107.0	93.4	115.0
1¾	5½	4.90	5.39	124.0	108.0	134.0
1⅞	5⅞	5.63	6.20	141.0	123.0	152.0
2	6¼	6.40	7.04	160.0	139.0	172.0
2⅛	6⅝	7.23	7.96	179.0	156.0	193.0
2¼	7⅛	8.10	8.91	200.0	174.0	215.0
2½	7⅞	10.00	11.0	244.0	212.0	263.0
2¾	8⅝	12.10	13.31	292.0	254.0	314.0

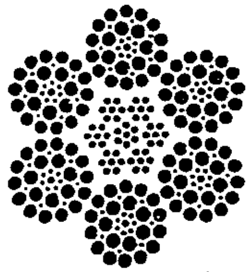
6 x 37 CLASS WIRE ROPE

6 Strands—Nominally 37 Wires per Strand

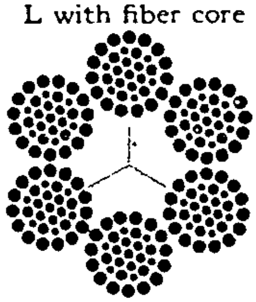
Purple Strand (Improved Plow Steel) or Plow Steel Grade
Regular or Lang Lay—Fiber Core or IWRC



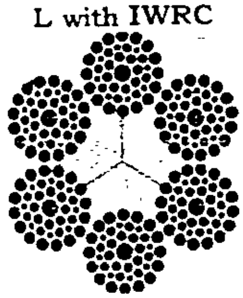
6 x 29 filler wire Type L with fiber core



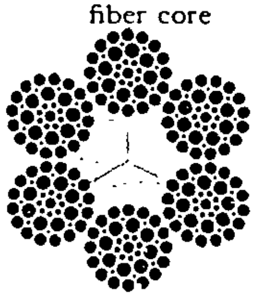
6 x 36 filler wire Type L with IWRC

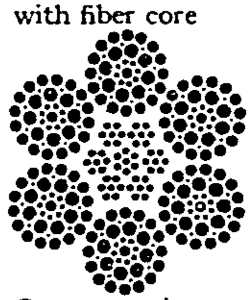


6 x 34 (2 Operations) Type M with fiber core

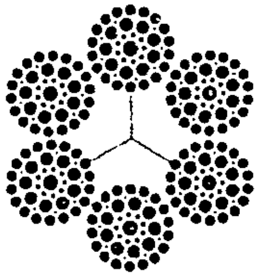


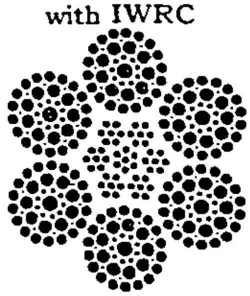
6 x 41 Warrington-Seale Type M with fiber core





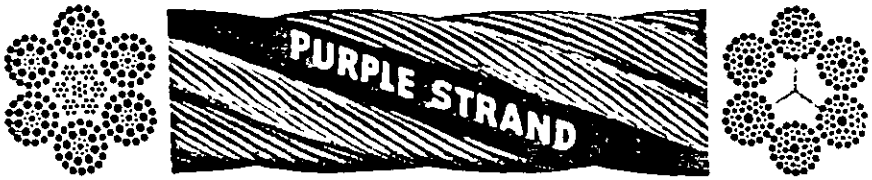
6 x 41 filler wire Type Q construction
with fiber core   with IWRC





6 x 46 filler wire Type R construction
with fiber core   with IWRC

6 x 37 CLASS WIRE ROPE



6 x 37 Class—6 Strands, nominally 37 Wires per Strand

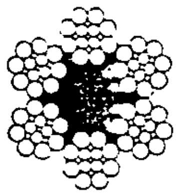
Purple Strand (Improved Plow Steel) Grade

Lang Lay or Regular Lay—IWRC—Fiber Core

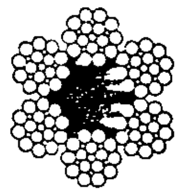
Diam	Circumference approx	Weight per ft, approx lb		min Breaking Strength in tons of 2,000 lb	
				Improved Plow Steel	
in.	in.	Fiber Core	IWRC	Fiber Core	IWRC
1/4	3/4	0.10	....	2.59	....
5/16	1	.16	....	4.03	....
3/8	1 1/8	.22	0.24	5.77	6.20
7/16	1 3/8	.30	.33	7.82	8.41
1/2	1 5/8	.39	.43	10.2	11.0
9/16	1 3/4	.49	.54	12.9	13.9
5/8	2	.61	.68	15.8	17.0
3/4	2 3/8	.87	.96	22.6	24.3
7/8	2 3/4	1.19	1.31	30.6	32.9
1	3 1/8	1.55	1.71	39.8	42.8
1 1/8	3 1/2	1.96	2.16	50.1	53.9
1 1/4	3 7/8	2.42	2.67	61.5	66.1
1 3/8	4 3/8	2.93	3.23	74.1	79.7
1 1/2	4 3/4	3.49	3.84	87.9	94.5
1 5/8	5 1/8	4.09	4.50	103	111
1 3/4	5 1/2	4.75	5.23	119	128
1 7/8	5 7/8	5.45	6.00	136	146
2	6 1/4	6.20	6.82	154	166
2 1/8	6 5/8	7.00	7.70	173	186
2 1/4	7 1/8	7.85	8.64	193	208
2 3/8	7 1/2	8.74	9.61	214	230
2 1/2	7 7/8	9.69	10.66	236	254
2 3/4	8 5/8	11.72	12.90	284	306
3	9 3/8	13.95	15.35	335	360
3 1/8	9 7/8	15.14	16.65	362	389
3 1/4	10 1/4	16.37	18.00	390	420
3 1/2	11	19.00	20.90	449	483

Constructions of Leschen Wire Rope

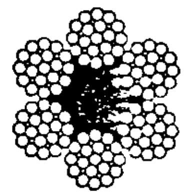
ROPES OF THE 6X19 CLASSIFICATION



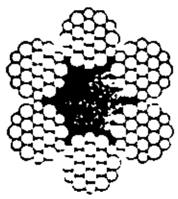
6x13 Filler Wire
(6x17 Filler Wire)



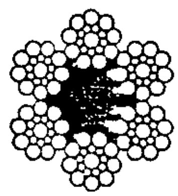
6x16 Filler Wire
(6x21 Filler Wire)



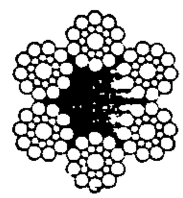
6x19 Filler Wire
(6x25 Filler Wire)



6x19 Warrington

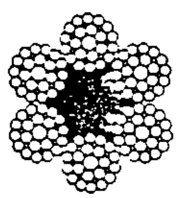


6x19 Seale

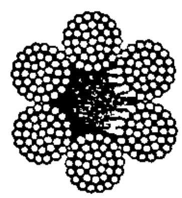


6x21 Seale

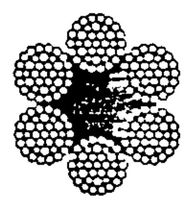
ROPES OF THE 6X37 CLASSIFICATION



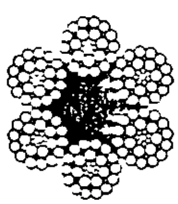
6x22 Filler Wire
(6x29 Filler Wire)



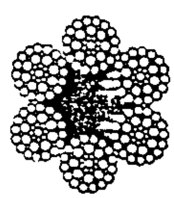
6x37 Filler Wire
(6x43—2 Operations)



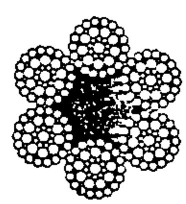
6x37
2 Operations



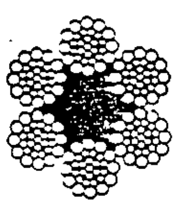
6x29 Filler Wire
(6x36 Filler Wire)



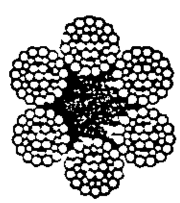
6x33 Filler Wire
(6x41 Filler Wire)



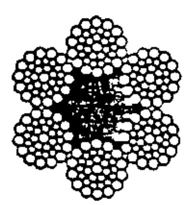
6x37 Filler Wire
(6x46 Filler Wire)



6x31
Seale



6x37 Seale

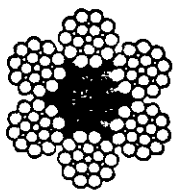


6x36
6x43 Filler Wire Seale

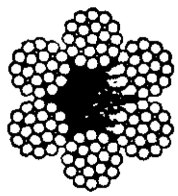
ROPES OF THE 6X19 CLASSIFICATION

*"HERCULES" (RED-STRAND) WIRE ROPE

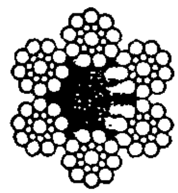
Fiber Core, Wire Rope Core or Wire Strand Core



Filler Wire (6x21) Fiber Core



Filler Wire (6x25) Fiber Core



Seale (6x19) Fiber Core

This data applies also to ropes made 6x13, 6x17 Filler Wire and 6x21 Seale.

PREFORMED OR NON-PREFORMED

Diameter in Inches	Breaking Strength in Tons of 2000 Pounds		Approximate Weight Per Foot in Pounds	
	Fiber Core	Steel Core†	Fiber Core	Steel Core†
3/16	1.55	1.67	.05	.06
1/4	2.74	2.95	.10	.11
5/16	4.26	4.58	.16	.18
3/8	6.10	6.56	.23	.25
7/16	8.27	8.89	.31	.34
1/2	10.7	11.5	.40	.44
9/16	13.5	14.5	.51	.56
5/8	16.7	17.9	.63	.69
3/4	23.8	25.6	.90	.99
7/8	32.2	34.6	1.23	1.35
1	41.8	44.9	1.60	1.76
1 1/8	52.6	56.5	2.03	2.23
1 1/4	64.6	69.4	2.50	2.75
1 3/8	77.7	83.5	3.03	3.33
1 1/2	92	98.9	3.60	3.96
1 5/8	107	115	4.23	4.65
1 3/4	124	133	4.90	5.39
1 7/8	141	152	5.63	6.19
2	160	172	6.40	7.04
2 1/8	179	192	7.23	7.95
2 1/4	200	215	8.10	8.91
2 1/2	244	262	10	11
2 3/4	292	314	12.10	13.31

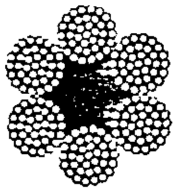
When galvanized, deduct 10 per cent from the bright rope strengths shown above.

*Reg. U. S. Pat. Off. †Wire Rope Core or Wire Strand Core.

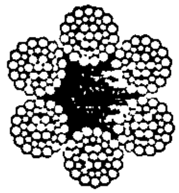
ROPES OF THE 6X37 CLASSIFICATION

*"HERCULES" (RED-STRAND) WIRE ROPE

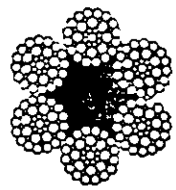
Fiber Core, Wire Rope Core or Wire Strand Core



Filler Wire
(6x43)
Fiber Core



Seale
(6x37)
Fiber Core



Filler Wire
(6x41)
Fiber Core

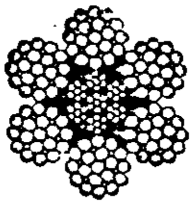
This data applies also to ropes made 6x29 Filler Wire, 6x37-2 Operations 6x36 Filler Wire, 6x46 Filler Wire, 6x31 Seale, 6x43 Filler Wire Seale, 6x49 Filler Wire Seale and 6x61-Multiple Operation.

PREFORMED OR NON-PREFORMED

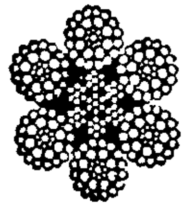
Diameter in Inches	Breaking Strength In Tons of 2000 Pounds		Approximate Weight Per Foot in Pounds	
	Fiber Core	Steel Core†	Fiber Core	Steel Core†
1/4	2.59	2.78	.10	.11
5/16	4.03	4.33	.16	.18
3/8	5.77	6.20	.22	.24
7/16	7.82	8.41	.30	.33
1/2	10.2	11	.39	.43
9/16	12.9	13.9	.49	.54
5/8	15.8	17	.61	.67
3/4	22.6	24.3	.87	.96
7/8	30.6	32.9	1.19	1.31
1	39.8	42.8	1.55	1.71
1 1/8	50.1	53.9	1.96	2.16
1 1/4	61.5	66.1	2.42	2.66
1 3/8	74.1	79.7	2.93	3.22
1 1/2	87.9	94.5	3.49	3.84
1 5/8	103	111	4.09	4.50
1 3/4	119	128	4.75	5.23
1 7/8	136	146	5.45	6
2	154	165	6.20	6.82
2 1/8	173	186	7	7.70
2 1/4	193	207	7.85	8.64
2 3/8	214	230	8.74	9.62
2 1/2	236	254	9.69	10.66
2 3/4	284	305	11.72	12.89
3	335	360	13.95	15.35
3 1/8	362	389	15.14	16.65
3 1/4	390	419	16.37	18
3 1/2	449	483	19	20.90

When galvanized, deduct 10 per cent from the bright rope strengths shown above

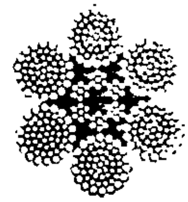
*Reg. U. S. Pat. Off. †Wire Rope Core or Wire Strand Core.



6x19



6x37



6x61

Lang Lay with I.R.W.C.

Preformed or Non-Preformed

Hoist Roebling **Boom Hoist**

Diam., Inches	Approximate Weight per Foot, Pounds			Breaking Strength in Tons of 2000 Pounds		
	6 x 19	6 x 37	6 x 61	6 x 19	6 x 37	6 x 61
3/8	.25	.....	.....	6.56	.....	.....
7/16	.34	.....	.....	8.89	.....	.....
1/2	.44	.43	.....	11.5	11.0	.....
9/16	.56	.54	.....	14.5	13.9	.....
5/8	.69	.67	.....	17.9	17.0	.....
3/4	.99	.95	.....	25.6	24.3	.....
7/8	1.35	1.31	.....	34.6	32.9	.....
1	1.76	1.71	.....	44.9	42.8	.....
1 1/8	2.23	2.16	.....	56.5	53.9	.....
1 1/4	2.75	2.66	.....	69.4	66.1	.....
1 3/8	3.33	3.22	.....	83.5	79.7	.....
1 1/2	3.96	3.84	.....	98.9	94.5	.....
1 5/8	4.65	4.50	.....	115.0	111.0	.....
1 3/4	5.39	5.23	.....	133.0	128.0	.....
1 7/8	6.19	6.00	.....	152.0	146.0	.....
2	7.04	6.82	6.82	172.0	165.0	165.0
2 1/8	7.95	7.70	7.70	192.0	186.0	186.0
2 1/4	8.91	8.64	8.64	215.0	207.0	207.0
2 3/8	9.92	9.62	9.62	239.0	230.0	230.0
2 1/2	11.00	10.66	10.66	262.0	254.0	254.0
2 5/8	.....	11.75	11.75	.....	279.0	279.0
2 3/4	.....	12.89	12.89	.....	305.0	305.0
3	.....	15.35	15.35	.....	360.0	360.0
3 1/8	.....	16.65	16.65	.....	392.0	392.0
3 1/4	.....	18.01	18.01	.....	419.0	419.0
3 1/2	.....	20.90	20.90	.....	483.0	483.0
3 3/4	.....	.....	23.98	.....	.....	549.0
4	.....	.....	27.28	.....	.....	620.0

The above ropes can also be furnished Regular Lay.

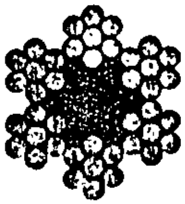
Preformed "Blue Center" Steel Standard Hoisting Rope

6 x 19 Classification

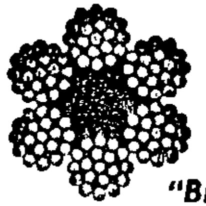
Diam., Inches	Weight, Pounds Per Foot		Breaking Strength, Tons		
	Bright, I.W.R.C.	Bright and Galv., Fiber Core	Bright, I.W.R.C.	Bright, Fiber Core	Galv., Fiber Core
¼	.11	.10	2.94	2.74	2.47
5/16	.18	.16	4.58	4.26	3.83
⅜	.25	.23	6.56	6.10	5.49
7/16	.34	.31	8.89	8.27	7.44
½	.44	.40	11.5	10.7	9.63
9/16	.56	.51	14.5	13.5	12.1
⅝	.69	.63	17.9	16.7	15.0
11/16	....	.76	....	....	18.1
¾	.99	.90	25.6	23.8	21.4
13/16	....	1.06	....	....	25.0
⅞	1.35	1.23	34.6	32.2	29.0
15/16	....	1.41	....	....	33.2
1	1.76	1.60	44.9	41.8	37.6
1⅛	2.23	2.03	56.5	52.6	47.3
1¼	2.75	2.50	69.4	64.6	58.1
1⅜	3.33	3.03	83.5	77.7	69.9
1½	3.96	3.60	98.9	92.0	82.8
1⅝	4.65	4.23	115.0	107.0	96.3
1¾	5.39	4.90	133.0	124.0	112.0
1⅞	6.19	5.63	152.0	141.0	127.0
2	7.04	6.40	172.0	160.0	144.0
2⅛	7.95	7.23	192.0	179.0	161.0
2¼	8.91	8.10	215.0	200.0	180.0
2⅜	9.92	9.02	239.0	222.0	200.0
2½	11.0	10.00	262.0	244.0	220.0
2¾	13.31	12.10	314.0	292.0	263.0

6 x 19 Standard Hoisting Rope—Probably more of this type of rope than any other is in use at the present time, as its ability to resist abrasion and bending is better proportioned to a wider range of uses than any other of the general rope constructions.

STANDARD WIRE ROPE

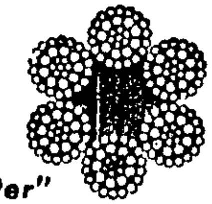


6x7

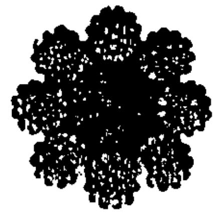


6x19

"Blue Center" Steel



6x37

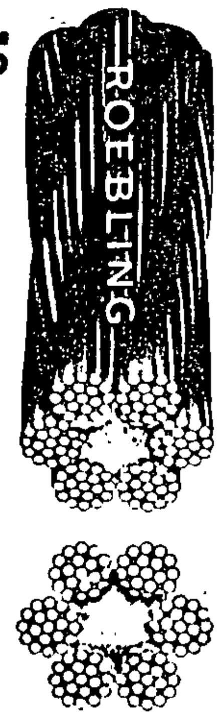


8x19

Diam., Inches	Approximate Weight per Foot, Pounds				Breaking Strength in Tons of 2000 Pounds "Blue Center" Steel			
	6 x 7	6 x 19	6 x 37	8 x 19	6 x 7	6 x 19	6 x 37	8 x 19
1/4	.094	.10	.10	.09	2.64	2.74	2.59	2.35
5/16	.15	.16	.16	.14	4.10	4.26	4.03	3.65
3/8	.21	.23	.22	.20	5.86	6.10	5.77	5.24
7/16	.29	.31	.30	.28	7.93	8.27	7.82	7.09
1/2	.38	.40	.39	.36	10.3	10.7	10.2	9.23
9/16	.48	.51	.49	.46	13.0	13.5	12.9	11.6
5/8	.59	.63	.61	.57	15.9	16.7	15.8	14.3
3/4	.84	.90	.87	.82	22.7	23.8	22.6	20.5
7/8	1.15	1.23	1.19	1.11	30.7	32.2	30.6	27.7
1	1.50	1.60	1.55	1.45	39.7	41.8	39.8	36.0
1 1/8	1.90	2.03	1.96	1.84	49.8	52.6	50.1	45.3
1 1/4	2.34	2.50	2.42	2.27	61.0	64.6	61.5	55.7
1 3/8	2.84	3.03	2.93	2.74	73.1	77.7	74.1	67.1
1 1/2	3.38	3.60	3.49	3.26	86.2	92.0	87.9	79.4

6x19 STANDARD HOISTING ROPE

This classification includes ropes of 6 Strands, 16 to 25 Wires to the Strand, and a Fiber Core.



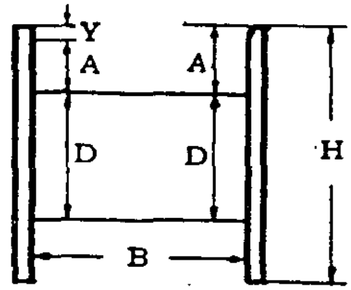
ROYAL BLUE Wire Rope

A PRODUCT OF ROEBLING

PREFORMED—6 x 37 CLASSIFICATION—STEEL CORE

Diameter: Fraction Inches	Diameter: Size Prefix	Weight Per Foot, Pounds	Breaking Strength, Tons
1/4	04	.11	3.20
5/16	05	.18	4.98
3/8	06	.24	7.14
7/16	07	.33	9.67
1/2	08	.43	12.6
9/16	09	.54	15.9
5/8	10	.67	19.5
3/4	12	.96	27.9
7/8	14	1.31	37.8
1	16	1.71	49.1
1 1/8	18	2.16	61.9
1 1/4	20	2.66	76.1
1 3/8	22	3.22	91.7
1 1/2	24	3.84	109.
1 5/8	26	4.5	127.
1 3/4	28	5.23	147.
1 7/8	30	6.	168.
2	32	6.82	190.
2 1/8	34	7.7	214.
2 1/4	36	8.64	238.
2 3/8	38	9.61	265.
2 1/2	40	10.66	299.
2 5/8	42	11.75	321.
2 3/4	44	12.89	351.
2 7/8	46	14.08	382.
3	48	15.35	414.
3 1/8	50	16.65	448.
3 1/4	52	18.01	482.
3 1/2	56	20.9	555.

Shipping Reel and Drum Capacities



The length of wire rope, in feet, that can be spooled onto a drum or reel, can be figured as below. There are two cases to be considered: (1), when the drum is filed just flush with the drum flanges; and (2), when the drum or reel is not completely filled.

Bethlehem

Let A = depth of rope space on drum, in inches

B = width between drum flanges, in inches

D = diameter of drum barrel, in inches

H = diameter of drum flanges, in inches

K = factor from table below for size of line selected

Y (in case 2 only) = depth not filled on drum or reel, in inches

FACTORS K USED IN CALCULATING DRUM AND REEL CAPACITIES

Rope Dia.	K	Rope Dia.	K	Rope Dia.	K
1/16"	49.8	1/2"	.925	1-3/8"	.127
3/32"	23.4	9/16"	.741	1-1/2"	.107
1/8"	13.6	5/8"	.607	1-5/8"	.0886
5/32"	8.72	11/16"	.506	1-3/4"	.0770
3/16"	6.14	3/4"	.428	1-7/8"	.0675
7/32"	4.59	13/16"	.354	2"	.0597
1/4"	3.29	7/8"	.308	2-1/8"	.0532
5/16"	2.21	1"	.239	2-1/4"	.0476
3/8"	1.58	1-1/8"	.191	2-3/8"	.0419
7/16"	1.19	1-1/4"	.152	2-1/2"	.0380

CASE 1—The dimension A equals (H—D) ÷ 2, for use in formula (a) below.

The length of rope in feet = (A + D) x A x B x K (a)

Example: find the length in feet of a 9/16-in. diam rope required to fill a drum having dimensions as follows: B = 24 in., D = 18 in., H = 30 in., A = (30-18) ÷ 2 = 6 in.

Use the value of K (0.741, for a 9/16-in. rope) from the table below.

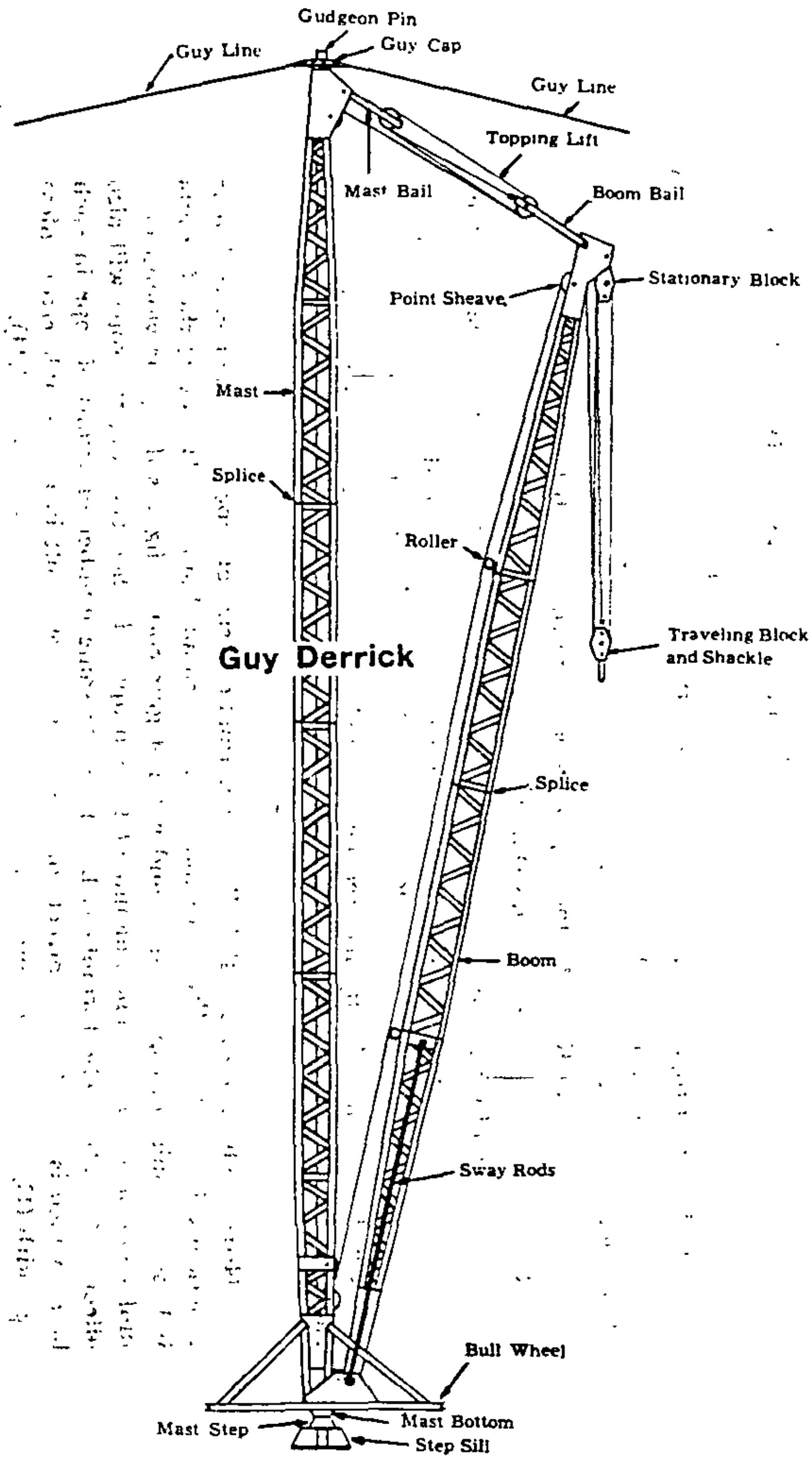
Substitute figures in equation (a) and

Length of rope in feet = (6 + 18) x 6 x 24 x 0.741 = 24 x 6 x 24 x 0.741 = 2560.9

Therefore, the theoretical capacity of a drum the size specified, using 9/16-in. rope, is 2560.9 feet. As a matter of practice, this would be rounded off to 2560 feet.

CASE 2—When the drum or reel lacks Y inches of being filled, the new value of dimension A to be used in equation (a) is, A = (H—D—2Y) ÷ 2.

NOTE: The values of "K" allow for normal oversize of ropes, and the fact that it is practically impossible to "thread-wind" ropes of small diameter. However, the formula is based on uniform rope winding and will not give correct figures if rope is wound non-uniformly on the reel. The amount of tension applied when spooling the rope will also affect the length. The formula is based on the same number of wraps of rope in each layer, which is not strictly correct, but which does not result in appreciable error unless the width (B) of the reel is quite small compared with the flange diameter (H).



Gudgeon Pin
Guy Cap
Guy Line
Guy Line
Topping Lift
Mast Bail
Boom Bail
Stationary Block
Point Sheave
Mast
Splice
Roller
Traveling Block and Shackle
Guy Derrick
Splice
Boom
Sway Rods
Bull Wheel
Mast Step
Mast Bottom
Step Sill