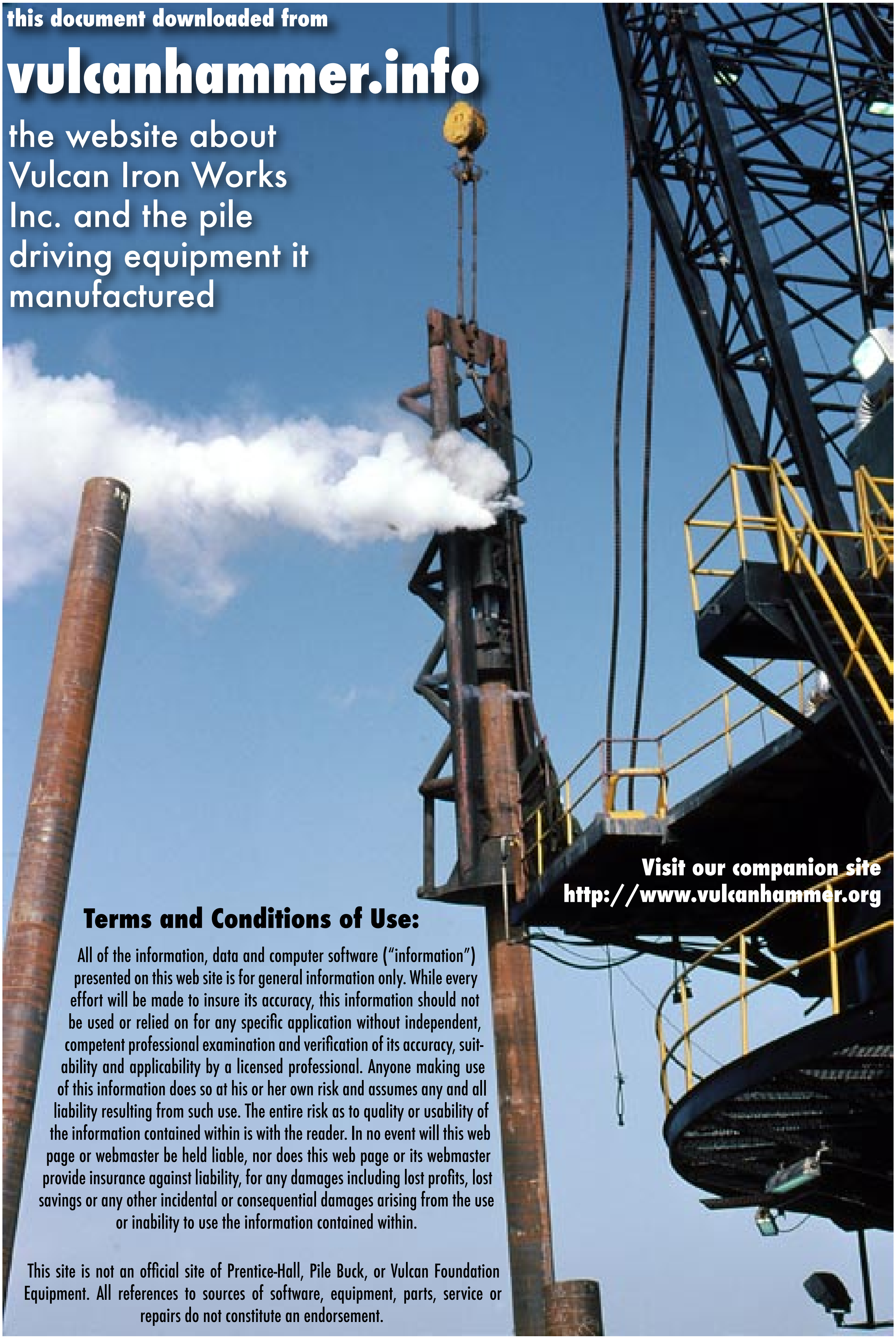


WKV Copy

Chapter 25 Pile Installation by Driving

T. G. Whipple
Division Construction Engineer
Pennzoil Company
Houston, Texas

and

W. R. Vines
Manager, Offshore Construction Engineering
TERA, Inc.
Houston, Texas

25.1 INTRODUCTION

Foundation piles for typical template structures are usually installed by driving with large impact hammers. The planning and timely execution of pile driving operations is critical to the success of an offshore installation, in terms of both cost and foundation adequacy. This chapter discusses elements of platform installation associated with advancing a pile to design grade by impact driving, including handling and connection of pile sections, hammer types and driving operations, and supplemental measures employed when refusal is encountered. These considerations are intended to highlight the importance of planning and analysis in the design phase, and to provide an overview of typical offshore procedures.

25.2 PLANNING CONSIDERATIONS

The key to successful pile installation, as in other phases of offshore construction, is detailed planning of anticipated operations and alternatives. The results of site investigations, structural design, drivability studies that were discussed in earlier chapters, and the details of pile installation discussed in this chapter are the basis for the installation plan that outlines anticipated offshore activities. This section discusses other considerations that influence the pile installation plan.

A primary consideration in the planning of pile installation is the sequence in which the piles are placed and advanced. Soil setup occurs during delays in driving and must be considered when planning splice points. Jacket stability is also of concern at each step of pile installation. Another major consideration is the effect of weather on planned operations.

25.2.1 Installation Sequence

The sequence in which piles or pile add-on sections are installed determines, or is determined by, considerations such as the size of the welding crew, the installation vessel and its equipment, jacket stability, set up, installation time requirements, and cost of installation.

Welding crew size affects the speed of welding operations and consequently affects job progress, unless more welders are available than can be utilized. The installation vessel and its equipment determine the loads that can be lifted, acceptable sea conditions for continuation of work, and hammer sizes available. Jacket stability considerations, as discussed later, may determine allowable loads to be supported by the jacket during installation. Installation time is affected by the number of piles advanced simultaneously and the difficulty of advancement to grade. The cost of installation is closely related to time requirements in most instances.

There are several approaches to sequencing pile installation:

(1) All piles installed simultaneously - pile add-ons are stabbed in all legs, welded simultaneously, and driven before next add-ons are stabbed. This method minimizes welding time for the installation as a whole since all piles are welded simultaneously, but a relatively large welding crew is required. Time required for handling hammers is minimized, but the risk of delays to pile advancement due to weld repairs, hammer breakdown, equipment maintenance, or weather is increased. On some structures, especially small four-pile jackets, there may be sufficient pile batter (inclination) to cause dimensional conflicts between hammer leads and other add-ons. Stability considerations may prevent this approach from being

employed when soils are soft at the mudline.

(2) Advance piles to grade individually - one pile is added-on, welded, and driven until that pile is advanced to final penetration. This method tends to minimize delays to advancement of the pile, and thus decrease setup time and anticipated driving problems. Welding time for the platform as a whole is increased compared to method 1 above. Welding crews are usually not well utilized with this approach unless the piles are large in diameter or the installation vessel is small. Leveling of the jacket may be adversely affected due to eccentric loads associated with this approach.

(3) Advance two piles simultaneously - two piles are worked to design grade together. These piles are usually located in diagonally opposed legs to help balance loads and settlements. For typical 8-pile jackets, the first piles installed are usually interior piles since eccentric loads on mudmats are minimized and jacket leveling is facilitated by single-battered piles. The advancement of two piles in diagonally opposite legs is a common method of pile sequencing with many of the advantages of both of the above procedures.

The circumstances of each installation will dictate which approach or combination of approaches is most likely to result in the most economical pile installation sequence. For example, a common practice is to install several piles of a platform to a depth where restart capability is certain, then advance the piles to design grade singly to reduce the risk of refusal.

25.2.2 Soil Setup

Soil setup describes a phenomenon frequently observed in clay soils and occasionally in other soil types where the resistance to driving is greater after a period of delay (setup time) than during continuous driving. This increased resistance is generally attributed to an increase in soil-pile adhesion due to factors such as reconsolidation of remolded soils during delays.[1] The rate of setup varies in an unpredictable manner from site to site, and is believed to be at least partly related to the dissipation of pore water pressures developed during driving.[2] Setup can occur quickly enough in some soils to affect restart of a pile after delays for connection of add-ons. The installation plan should recognize and provide for the anticipated effects of setup[3] and guard against premature refusal due to setup during normal delays. Observed setup rates and soil sensitivity at nearby sites (or for the first piles installed) are useful in modifying the installation plan to achieve maximum efficiency. This is frequently done by altering the pile installation sequence and/or hammer size to be used on particular add-on sections.

When significant soil setup is anticipated, the selection of penetrations at which add-ons will be spliced into the pile is a major consideration of pile design. In order to reduce the risk of refusal, it is important to plan add-on lengths that minimize the soil resistance encountered when driving is resumed. Long add-ons minimize the number of welds to be made (to save cost), and will reduce the amount of pile embedded in the soil at restart. When driving through stratified soils of clay and sand (or silt), add-on lengths are usually designed to avoid stopping with the pile tip in granular material since maximum driving resistance can generally be anticipated from high end bearing in sands or silts combined with soil setup in clays.

Performance of pile driving hammers is important in any situation where hard driving is expected, but is even more important in circumstances where soil setup can be expected. Adverse effects of poor hammer performance or breakdown can be reduced when an operational backup hammer (in offshore leads) is available on the installation vessel.

25.2.3 Jacket Stability

Bottom bay horizontal bracing and mudmats (if required) provide support for the jacket and for piles supported on the jacket during installation. Mudmats should be sized to assure soil pressures and settlements less than allowable for loads from anticipated sea conditions during installation as well as for jacket and pile loads at each stage of installation. Bottom bay framing should be sized to provide adequate strength against these anticipated soil loads.

Depending on soil conditions, the sequence of pile installation can affect efforts to maintain jacket stability. At sites with very soft seafloor soils, allowable pressures and settlements may dictate special installation sequences or special procedures such as ballasting/deballasting of jacket legs and piles. Near-surface voids or crusts underlain by soft sediments are cause for concern during installation, since stability of the structure on the seafloor can be threatened by punching through the crust during jacket setting or pile driving operations.

Determination of anticipated bearing pressures on the soil includes the effects of possible storm loadings as well as load eccentricity(My/I) and average pressure (P/A) due to the structure dead load for each step during installation. Steps to be considered include placement of jacket and piles, ballast/deballast operations, addition of pile add-ons, and achievement of pile self-support. Figure 25-1 illustrates the bottom bay framing and mudmats for a simplified four-pile jacket along with maximum bearing pressures from the structure dead loads associated with a part of the installation. In this example, the maximum bearing pressures (calculated assuming rigid body movements of the jacket and linear soil stress distribution) occur after lowering of one of the piles to self-support.

25.2.4 Weather

Weather may be the single most unpredictable and potentially costly consideration of platform installation. Weather must be considered in the overall pile installation plan when defining operational limits for pile and hammer handling operations, when analyzing the effect of delays on setup, and in estimating anticipated weather down-time. Weather also affects the day-to-day scheduling during installation when uncertainties in anticipated weather conditions have a potential of delaying planned operations that must be completed once initiated.

Adverse weather effects can be grouped into three catagories:

(1) Excessive vessel motions as a result of wind or sea conditions prevent timely or safe installation operations. Even if work is not stopped, progress can be slowed substantially. Limiting conditions are dependent on vessel characteristics, wind speed, and height, direction and period of waves. Vessel motion is critical for operations that involve placement of piles or equipment. Table 25-1 gives limiting sea states for several vessel types for pile installation. Table 25-2 lists some of the currently available installation vessels. Relative motion between vessels is critical when lifting piles or equipment from support vessels, and presents a safety hazard when transferring personnel. Delays can occur if the work vessel must be repositioned to minimize motion in changing weather conditions.

(2) Anchor slippage presents a hazard to the safety of vessels, pipelines, and nearby structures when forces on the anchoring system grossly exceed its holding power. Less severe slippage may cause delays for resetting of anchors or repositioning of the vessel to minimize forces. Typical anchor line scopes are from six to ten times water depth, with longer scope resulting in lower forces on the anchor. Installation

Table 25-1

Limiting Sea States For Pile Installation *

Vessel Type		
Derrick Barge (350'x100')	Derrick Ship (600'x140')	Semisubmersible (450'x250')
6'	10'	30'

* Limiting sea states shown are very approximate, and assume that vessels are headed into seas with periods different than the natural period of the vessel. Limiting sea states can vary substantially depending on specific vessel characteristics, anchoring, direction and period of seas, and required installation operations.

TABLE 25-2

INSTALLATION VESSEL DATA

Owner	Vessel	Type	Location	Size	Max. Crew Size	Crane	
				Length X Width X Depth, ft		Boom Ht, ft	Capacity, tons Fixed/Rotary
ARAMCO	Queen Mary	DLB	3	250x72x16	82	---	250/ -
Bisso Marine	Ajax	DB	1	120x65x11	---	130	255/ -
	Cairo	DB	1	165x70x14	---	106	360/ -
	Cappy Bisso	DB	1	200x70x14	---	160	650/ -
Brownaker Offshore	Borgila Dolphin	SCV	1	Aker H-3	650	---	- /600
Brownaker Service	Borgland Dolphin	SCV	2	Aker H-3	540	220	- /120
Brown & Root,Inc.	BAR 297	DLB	1	350x100x25	231	200	- /650
	BAR 323	DLB	1	400x100x30	238	175	- /250
	BAR 324	DLB	2	400x100x30	238	175	- /250
	Foster Parker	DB	1	350x100x25	122	---	- /550
	George R. Brown	DB	1	350x100x25	122	---	- /550
	H.A.Lindsay	DB	1	300x90x19	117	---	- /350
	Hercules	DB	3	400x140x25	150	245	1600/1200
	Hugh Gordon	DBL	1	400x100x30	230	200	- /800
	L.B.Meaders	DBL	1	400x100x30	250	200	- /800
	Atlas No. 1	DB	1	410x146x25	200	---	2000/1600
	Ocean Builder I	DBL	1	640x120x45	265	---	2000/1600
	Semac I	SDL	2	432x180x91	---	---	------
	Bar 332	DLB	4	352x100x26	185	275	- /925
	Bar 333	DLB	1	250x72x15	75	195	- /275

TABLE 25-2 (continued)

INSTALLATION VESSEL DATA

Owner	Vessel	Type	Location	Size Length X Width X Depth, ft	Max. Crew Size	Crane Boom Ht, ft	Crane Capacity, tons Fixed/Rotary
Constructors Heerema	Moron	PD	5	270x52x18	---	---	500/ -
	Rotterdam	PD	5	140x60x8	---	---	---
Christianson Construction	Barnacle	DB	6	150x50x11	50	180	150/ -
CMM	ORCA	DLB	7	595x99x44	200	240	1800/1600
	Sea Line I	DLB	7	698x125x48	286	300	2000/1600
	Tolteca	DLB	7	682x118x46	231	300	2000/1600
Crowley Maritime	400-7	DB	3	400x100x20	---	125	600/ -
	Cordova	DB	3	400x79x19	40	2@150	2@-/125
	DB-5	DB	6	135x50x11	15	100	100/ -
	DB-7	DB	6	135x50x11	15	120	50/ -
	DB-16	DB	6	140x70x14	10	132	135/ -
	DB-17	DB	6	115x52x10	---	120	50/ -
	DB-25	DB	8	150x54x13	---	155	102/ -
	DB-300	DB	8	255x76x15	---	200	- /275
	PAC 560	DB	6	280x66x20	---	110	33/ -
	PAC 570	DB	6	324x74x22	---	80	25/ -
Dolphin Services	Belford Dolphin	SCV	2	Aker H-3	300	220	120/
ENAP	YAGANA	DLB	5	325x90x22	126	200	500/400

TABLE 25-2 (continued)

INSTALLATION VESSEL DATA

Owner	Vessel	Type	Location	Size Length X Width X Depth, ft	Max. Crew Size	Crane Boom Ht, ft	Crane Capacity, tons Fixed/Rotary
E.T.P.M.	ETPM 202	DB	3	250x82x16	---	---	---
	ETPM 401	DLB	9	265x87x19	160	245	- /350
	ETPM 501	DLB	9	330x100x20	184	245	- /500
	ETPM 502	DLB	3	360x100x26	213	245	- /500
	ETPM 701	DLB	1	428x100x29.5	193	---	- /650
	ETPM 801	DLB	-	351x101x25	200	---	- /800
	ETPM 1601	DLB	2	610x115x50	348	220	2000/1600
Heerema	Balder	SCV	2	449x282x137	350	---	2@ 3000/2000
	Challenger	DB	2	634x96x32	144	---	- /800
	Champion	DB	1	472x100x33	126	---	1150/800
	Hermod	SCV	2	449x282x137	350	---	2@ 3000/2000
	Odin	DB	1	584x140x37	---	---	---/3000/---
	Thor	DB	2	551x129x37	---	---	---/2000/---
Houlder Comex	Uncle John	SCV	2	253x173x19	106	---	2@-/100
Jardine Offshore	Benbecula	DB	-	320x90x23	180	---	600/350
McDermott	Barge No. 1	DB	3	180x70x11	30	171	100/100
	DB No. 7	DLB	3	300x90x19	100	220	250/ -
	DB No. 8	DB	1	300x90x19	77	220	250/250
	DB No. 9	DLB	3	300x90x19	80	200	250/ -

TABLE 25-2 (continued)

INSTALLATION VESSEL DATA

Owner	Vessel	Type	Location	Size Length X Width X Depth, ft	Max. Crew Size	Crane Boom Ht, ft	Crane Capacity, tons Fixed/Rotary
McDermott (con't)	DB No. 11	DLB	9	300x90x22	80	220	250/250
	DB No. 12	DB	1	400x100x29	156	240	860/620
	DB No. 14	DLB	3	400x93x29	240	240	600/600
	DB No. 15	DLB	9	400x100x29	156	240	600/600
	DB No. 16	DB	1	400x100x29	156	275	860/620
	DB No. 17	DLB	1	400x106x29	156	275	600/500
	DB No. 18	DB	1	350x100x25	120	245	750/500
	DB No. 19	DB	3	300x100x21	80	245	700/500
	DB No. 20	DLB	10	401x100x29	154	275	750/600
	DB No. 21	DLB	11	401x100x29	154	275	750/600
	DB No. 22	DB	1	448x131x45	210	275	1100/800
	DB No. 23	DB	1	401x100x29	156	275	750/600
	DB No. 24	DB	9	250x80x16	60	217	350/250
	DB No. 25	DLB	10	400x106x29	224	275	600/600
	DB No. 100	SCV	2	406x275x130	548	300	2000/1600
	Derrick Barge 101	DB	3	475x171x120	270	350	2000/1600
Micoperi	M12	DB	12	164x72x14	13	105	300/ -
	M26	DB	2	600x116x50	157	285	2000/1600
	M30	DLB	3	400x80x27	119	165	750/ -
	Pearl Marine	DB	2	667x122x53	178	---	2000/1600
	Railto	DB	12	263x82x20	---	180	- /225

TABLE 25-2 (continued)

INSTALLATION VESSEL DATA

Owner	Vessel	Type	Location	Size	Max. Crew Size	Crane	
				Length X Width X Depth, ft		Boom Ht, ft	Capacity, tons Fixed/Rotary
Nippon Steel	Kuroshio	DLB	11	404x94x26	186	200	800/600
	Kuroshio II	DLB	11	459x111x30	212	230	1100/800
NKK	Kokan Pioneer I	DLB	11	456x98x30	242	---	720/550
Raymond Int'l	Cayuga	DB	13	155x50x13	---	160	75/ -
	Chelsea	DB	13	135x50x12	---	135	62/ -
	Cree	DB	1	155x50x12	---	125	80/ -
	LB-3	DB	13	200x46x10	---	---	---
	LB-22	DB	5	200x46x10	---	---	100/ -
	LB-23	DB	5	200x46x10	---	---	50/ -
	Loretta	J	3	139x80x10	---	220	150/ -
	Polaris	DLB	1	450x128x30	---	---	---
	Regulus	J	3	231x98x18	60	140	- /350
	Vega	DB	3	301x93x22	120	140	- /350
Saipem	Castoro 2	DLB	13	450x107x30	192	---	- /800
	Castoro 3	DB	13	250x86x16	---	---	- /150
	Castoro 4	DB	5	250x86x16	66	---	- /300
Santa Fe	Cherokee	DB	1	350x100x25	184	---	1000/600
	Choctaw II	DLB	1	400x106x54	260	230	750/600
	Seminole	DB	6	150x55x10	---	150	150/100
	Tonkawa	DLB	1	250x72x16	63	205	175/ -

TABLE 25-2 (continued)

INSTALLATION VESSEL DATA
CONTINUE

Owner	Vessel	Type	Location	Size Length X Width Depth #	Max Crew Size	Boom Ht, #	Crane-Specs -Capacity, tons Fixed/Rotary
Selco Pte	Selco L-10	DB	4	120x50x10	---	125	- /120
	Selco L-60	DB	4	155x75x15	12	240	- /300
Smith-Rue	Derrick Barge 5	DB	6	225x78x15	---	190	400/325
	Derrick Barge 24	DB	6	150x54x13	---	165	130/100
	Super Scoop	DB	6	176x70x12	---	145	200/150
Stolt-Nielsen	Seaway Swan	SCV	2	325x221	150	---	- /200
Teledyne Movible	Movible DB-1	DB	1	240x70x15	50	180	- /150
	Movible DB-2	DB	1	350x100x25	100	200	800/650
	Movible DB-3	DB	-	350x100x25	100	200	800/650
Trans Ocean Contr.	Samson	DLB	1	230x60x15	50	140	300/100
Oglands Rederi	Sarita	DLB	1	677x122x51	200	---	2000/1600
Wilhelmsen	Treasure Finder	SCV	2	Aker H-3	500	---	---
	Treasure Hunter	SCV	2	Aker H-3	560	---	120/ -
Williams-McWilliams	Exxon DB-1	DLB	1	300x90x19	---	---	- /250
	W-701	DLB	1	180x60x12	40	135	- /160

LOCATION
1-GOM
2-N.Sea
3-Mid East
4-Far East
5-S.A.
6-Pacific U.S.
7-Mexico
8-Alaska
9-Africa
10-SE Asia
11-Australia
12-Mediterranean
13-Caribbean

VESSEL TYPE
DLB - Derrick Lay Barge
DB - Derrick Barge
SCV - Semi Constr Vessel
SDL - Semi Derrick Bay Vessel
PD - Pile Driver

vessel positioning should anticipate the effects of anchor slippage during rough weather to avoid damage to the platform and other nearby structures.

(3) Direct effects of weather conditions present safety hazards to crew and equipment during periods of electrical storms and high winds. Welding operations may be delayed due to rain, high winds, or rough sea conditions. Jacket stability is threatened by storm conditions occurring before sufficient pile penetration or ballast is provided.

Both direct and indirect economic impacts result from adverse weather conditions. Direct impacts include standby time for the installation spread, and loss of or damage to equipment and structures. Indirect costs include those associated with delays in commissioning of the platform, remedial measures required due to soil setup, and jacket leveling required.

25.3 PILE HANDLING

Pile sections are usually transported to the installation site by barge, and must be lifted into place. Piles may be transported on the same barge as the jacket or on separate material barges, depending on size of the structure and quantity of piling. Although unusual structures may dictate special pile handling procedures and/or equipment[4], each installation contractor has preferred methods for use under normal conditions.

25.3.1 Positioning Sequence

Pile positioning consists of moving a pile section from the transport vessel to the location and attitude required for connection to the piling string. Additional movement of piles may be required to sort add-on sections or to move sections from a material barge to the deck of the installation vessel; these time-consuming steps can usually be minimized with proper load-out planning. After pile handling equipment has been placed, the pile section is lifted by the crane (Figure 25-2a). The crane block supports the weight of the pile and controls the position of the top of the section. Control of the lower end is provided by one or more tugger lines, which are relatively light cables controlled manually or connected to compressed air winches (air tuggers) mounted on the crane. Sudden sliding or swinging of the lower end of the pile section is a safety hazard during initial lifting operations (Figure 25-2b). Also, ovaling or buckling damage to internal stabbing guides may occur at this stage. Damage to the stabbing guides makes the stabbing operation more difficult and possibly decreases clearance inside the pile (an important consideration if wells are to be drilled through the pile or conductor, or if jetting is subsequently required).

After the pile is lifted clear of the barge deck, the crane rotates and positions the pile for placement as shown in Figure 25-2c. During stabbing operations, the bottom end of the lifted pile section is placed in the jacket leg (initial section) or aligned with the top of the pile string already in place as shown in Figure 25-2d.

Add-ons are placed with the use of internal or external stabbing guides which also hold the pile in place during welding operations. Stabbing guides help align pile sections during placement, and align the sections during connection. Internal stabbing guides (Figure 25-3a) are normally used unless inside clearance is a controlling factor, since external guides (Figure 25-3b) require more time to set up and make welding more difficult. Internal guides cause problems in the stabbing operation if damaged (as mentioned above) or if not properly matched to the top of the existing pile string. When mechanical connectors are used instead of welding, special alignment systems are sometimes used.[4]

Installation vessel motion is a critical factor in the stabbing operation. Pile placement is a more critical operation than hammer placement or driving because of the small target and clearances afforded by the pile top or jacket leg. An experienced crane operator can compensate for relatively large pitching motions of the vessel, but significant roll can make stabbing a time-consuming, dangerous, or impossible operation. Vessel motion is dependent on stability characteristics of the vessel as well as sea conditions including direction, period, and amplitude of waves and swells.

25.3.2 Handling Methods

There are many ways to provide lifting points for positioning of pile sections. Some of the more commonly used variations are holes or padeyes near the pile top through which shackles are pinned; choking with slings; bridle arrangements; and elevators.

Shackles pinned through holes at the pile top (Figure 25-4a) are simple to attach, even in relatively cramped stacks of pile sections. A pair of holes, diametrically opposed, are cut several inches from the pile top. Shackles attached to wire rope slings are pinned through the holes. To disconnect from the add-on to be driven, the slings are unshackled from the pile top from a personnel basket or bosun's chair. Unless a specially-designed basket is used, this operation can be difficult and unsafe, especially in rough seas or when shackles are heavy. The normal pile cutoff allowance includes the portion with holes. Holes and shackles should be sized to assure that the pile section (or pile string, if it must be lifted to remove stops) can be safely lifted.

Shackles pinned through padeyes welded near the pile top (Figure 25-4b) is another simple way to provide lifting points. Attachment and detachment of shackles are usually similar to that described above. If the hammer leads can fit over the padeyes or if the padeyes are positioned below the leads, the lifting slings can be detached from the crane and left in place on the pile during driving. For pile sections that may not require driving, the padeyes can be used as stops to support the pile string on the jacket while making add-ons. Welding of the padeyes is generally done in the fabrication yard to avoid long delays offshore. Padeyes must be cut off before lowering of subsequent pile sections. When removing padeyes, the pile must not be gouged; if tight shims or grout packers will be passed, the padeye stub should be ground smooth. Care should be exercised when designing padeyes to account for changes in load direction during initial lifting of the section.

Long lead sections of pile are sometimes lifted by choking with wire rope slings (Figure 25-4c). Since no holes or padeyes are required, little time is lost after placement in preparation for driving. This method is generally limited to initial sections of piles for platforms in relatively shallow water, where the initial section is long enough to support the weight of pile and hammer. A support lug can be welded below the choke point to provide support for piles that are too short to reach the seafloor, but scaffolds are required for welding of add-ons. Long sections of pile can be handled by choking since the lift point away from the end of the pile reduces bending stress during lifting. However, control of the pile section is not as satisfactory as with other methods where the pile is lifted essentially from the top. Choke points should be kept well above the center of gravity of the section to increase control.

A bridle arrangement (Figure 25-4d) with wire rope slings shackled to a padeye near the bottom of the section allows fast, safe disconnection of the lifting slings from the pile since the shackle can be reached from the platform. The bridle, to which lifting slings are attached, is free to twist or slide along the section and provides control over movement of the pile top. This arrangement usually requires more preparation on the deck of the installation vessel, and makes control of the pile somewhat more difficult. Since the bridle is connected at the bottom of the section, it is used only for add-ons.

An elevator, or collar that bears against stops near the pile top (Figure 25-4e), is another method of rigging piles for safe disconnection. A circular box-section elevator, with padeyes for sling attachment, is clamped around the pile section below the stops. After the section is lifted and in place, the elevator slides down the pile for unclamping. Hydraulically operated clamps are also available to perform essentially the same function without the need for stops.[4] Elevators are particularly well-suited for handling large pile sections.

25.4 CONNECTION OF PILE SECTIONS

Pile add-ons are connected into the piling string by welding or with special-purpose mechanical connectors. This section discusses the equipment, processes, and problems particular to welding and inspection of welds for offshore pile installation as well as the currently available alternative connection methods. For a more general treatment of welding and inspection practices for offshore construction as a whole, refer to Chapter 27.

25.4.1 Welded Pile Connections

Welding is the most common method of connecting pile sections together. As in onshore fabrication, shielded metal arc (SMAW) and flux cored arc welding (FCAW) are the two usual processes. SMAW makes use of the familiar consumable stick electrode while FCAW is a semi-automatic process which uses a continuous innershield electrode. Because of the necessity of changing electrodes in SMAW, FCAW is a faster process, but the equipment used is more difficult to set up properly. FCAW is more prone to produce cracking in high strength applications than SMAW, especially in windy or wet conditions, becaue of faster colling of weld bead. Also, the electrode reels used in FCAW require storage procedures similar to low hydrogen SMAW electrodes to prevent pickup of moisture which can reult in porosity and cold cracking of welds.

Welding time required for connection of pile add-ons is dependent mainly on the wall thickness and the number of welders available. Welders are usually spaced at a minimum of 3 ft around the pile perimeter to reduce interference and burn unjuries. Welding time is influenced by welder experience and performance, bevel width, wall thickness, specific process used, and weather (discussed below). Bevel width must be large enough to allow access and visibility, and to prevent slag inclusions but as small as practical to reduce welding time. A total angle of 45 to 60 degrees, depending on wall thickness, is typical for large piles.

Table 25-3 lists typical welding times for the SMAW process for several common wall thicknesses. Repairs to welds for correction of defects substantially increase connection time requirements. The use of high strength steels (sometimes used to reduce required wall thickness) requires more time-consuming procedures, including preheating in some cases. In offshore installations, preheating cannot be as well-controlled as in shop conditions since torches are used to heat the joint, resulting in somewhat uneven temperatures.

Environmental conditions such as wind speed and rain affect the probability of defects and determine the feasibility of continuing welding operations. Code provisions require windbreaks when winds are in excess of

TABLE 25-3

TYPICAL WELDING TIME REQUIRED

Wall Thickness (in.)	Typical Welding Time (hrs.)
1	3
1.5	4
2	8
3	16
4	24

5 mph; in practice, the level of experience of the welding crew may allow production of acceptable welds at higher windspeed without benefit of windbreaks. To avoid cracks due to cooling from evaporation, welding should not be attempted during periods of rain. Even though the weld may be protected from direct contact with rain, moisture from higher elevations on the pile add-on can eventually reach the weld. In rough sea conditions, splashing of seawater may halt welding before the installation vessel is forced to pull off location. Welds should be allowed to cool (to a temperature commensurate with the welding procedures used) before lowering pile sections to prevent cracking.

25.4.2 Inspection of Welds

Inspection of welds that connect pile add-ons normally consists of both visual inspection and nondestructive testing. As in other phases of the construction process, it is to the mutual benefit of contractor and operator for the welding supervisors to understand the criteria of the inspection team for acceptance and rejection of welders, procedures, and welds. The primary goal of inspection is to maintain at least a specified minimum level of weld quality.

Due to the possibly severe economic penalties, both for in-service failure of a weld and for repair of defects during offshore construction operations, acceptance criteria for field welds becomes very important. If an overly strict criteria is used, the cost of installation is increased. If acceptance criteria or level of inspection is relaxed to a degree not compatible with design assumptions, the risk of in-service failures is increased.

Inspection of pile sections during fabrication helps assure efficiency of field welding activities. In order to help prevent weld defects, particular attention is paid to pipe diameter, wall thickness, and out-of-roundness; bevel shape; and material laminations at the bevel. Inspection of fit-up and alignment prior to the start of welding is an important factor in reducing repairs and increasing confidence in weld quality.

Ultrasonic examination is the usual nondestructive testing technique used for pile add-on welds. The extent of ultrasonic testing ranges from a random sampling to full inspection (100% of welds). In general, more nondestructive testing is required for offshore work than for onshore fabrication.[5]

25.4.3 Mechanical Connectors

The economic penalties for welding time and possible repairs, which include pile refusal in some cases as well as spread time, have prompted the development of alternate pile connection methods. The gravity follower used with skirt piles is one alternate method. Specialized mechanical connectors are finding more frequent application as offshore platforms are constructed in deeper water.

Gravity followers (Fig. 25-5a) are used when driving skirt piles to avoid the necessity of underwater cutting when separating follower and pile. The follower itself must usually consist of welded add-ons that are separated at the surface when extracting the follower. The only connection between the pile and the follower is through gravity, and as a result no tensile forces can be transmitted from one to the other. Energy losses occur during driving at the follower-pile interface.

Mechanical pile connectors are designed to provide a positive connection without welding. Two basic types are currently available. The breechblock,[6] shown in Fig 25-5b, connects sections with a mated set of interlocks which are twisted together in a system similar to the locking mechanism of a bolt-action rifle. Hydraulic connectors[7] use hydraulically-controlled lugs as shown in Fig. 25-5c to connect the pile sections. Alignment guides are used to reduce delays in mating the sections. Mechanical connectors are most frequently used on followers for installation of skirt piles in deep water to minimize setup and time required to retrieve follower sections. Although time requirements and the risk of refusal are reduced, the cost of mechanical connectors is such that total costs of pile connection are of the same order of magnitude of welded connections.

25.5 HAMMERS AND ACCESSORIES

Pile driving hammers supply the impact forces with which a pile is driven. Hammers available for offshore applications have progressed from maximum rated energies of about 200,000 ft-lb in 1970 to 1,800,000 ft-lb in 1980. During this period, hydraulically operated hammers and hammers capable of driving in an underwater environment have also become available.

25.5.1 General

There are three major classifications of hammers by power source:

(1) Air/Steam Hammers

(2) Diesel Hammers

(3) Hydraulic Hammers

Hammers can be further classified as (1) single acting, in which the power source is used only to raise the ram, which then drops under the force of gravity; and (2) double acting, in which the power source is used to move the ram both upwards and downwards. Most large air/steam and diesel hammers are single acting, while currently available hydraulic hammers are double acting.

The mating of hammers to piles is accomplished with pilecaps (anvils). Pilecaps can be specially fabricated for specific pile sizes, but are usually stepped to allow use with a wide range of pile diameters. Conical transitions in the piling string can be used to mate normal pile-cap sizes to large diameter piles to avoid the need for special pilecaps. In addition to transferring impact forces from the ram to the pile, the pilecap acts to center the hammer over the pile to minimize eccentricity of the hammer blow.

Cushions are used with virtually all pile driving hammers. In air/steam and diesel hammers, the cushion is packed into a recess in the top of the pilecap. Cushion materials commonly used for air/steam hammers include Bongassi hardwood, asbestos, and sandwich-type cushions made up of combinations of steel plate, asbestos, wire rope, aluminum and/or micarta. In hydraulic hammers, a cushioning effect is provided by compressed gas in the ram cylinder or by a specially designed flexible pilecap. The cushion serves to protect the ram and the pile from damage due to impact stresses.

An offshore cage, or leads, is a support frame commonly used to handle hammers and to align the hammer and pile during driving. In the case of hammers with underwater capabilities, the housing of the hammer fulfills a similar role in supporting and aligning the hammer while also acting as a barrier to seawater. Leads act as a protective cage for storage and transportation, and support the hammer during lifting and placement. Once the hammer is in place on the pile, the leads help align the hammer with the pile and reduce disturbance to the hammer from installation vessel motion. In case a pile "runs" (increases penetration without driving), the leads allow a skilled operator to prevent the hammer from following the pile and damage the jacket structure. Figure 25-6 illustrates a typical hammer in offshore leads.

25.5.2 Air/Steam Hammers

Hammers powered by steam or compressed air are currently the most common type used for driving foundation piles for typical offshore structures. Hammers in the 500,000 to 1,000,000 ft-lb range are commonly available. At least one model is capable of operating underwater.[8] Table 25-4 lists rated energy, maximum stroke, weight data, rated operating pressure, steam and air consumption rates, supply hose requirements, and rated blows per minute for typical air/steam hammers with rated energies of 50,000 ft-lb or more.

Figure 25-7 illustrates the major features of two types of single-acting air/steam hammers, one with a fixed piston and one with a fixed cylinder. Air/steam hammers can be visualized as a one-cylinder steam engine. The ram is guided by a lubricated column or columns as it is pushed upward by compressed air or steam. At the top of a stroke, the valved inlet is closed and the exhaust port, through which the compressed air or steam escapes, is opened. The ram then decelerates and falls under the influence of gravity, guided by the lubricated columns. Forces such as friction along the guide columns, cylinder walls, and packing, and air or steam pressure in the cylinder act against gravity during drop of the ram, slowing its impact velocity. Immediately after impact the valved inlet opens to admit air or steam and the ram is raised to the top of its stroke. Timing of the cycle, exhaust of air or steam, and lubrication are all important factors in the efficient operation of the hammer.

Performance data (hammer efficiencies and cushion properties) for steam hammers is presented in Section 25.6.7.

25.5.3 Diesel Hammers

Figure 25-8 shows the components of a typical, single-acting diesel hammer, which is basically a one-cylinder diesel engine. On the downstroke, the ram falls under the force of gravity guided by the cylinder which forms the hammer housing. Exhaust ports are closed, and fuel is injected into the cylinder just prior to impact. The extreme compression of the trapped air causes an increase in temperature such that the diesel fuel explodes just after impact. The force of the combustion pushes the ram to the top of its stroke, completing the cycle. In diesel hammers, the stroke of the ram can be controlled by the amount of fuel injected. The timing of the combustion is very critical to hammer performance. If the fuel is ignited too soon, the ram impact velocity is significantly reduced. Table 25-4 lists the rated energy, stroke and weight data, fuel consumption, and rated blows per minute for typical diesel hammers with more than 50,000 ft-lbs of rated energy.

25.5.4 Hydraulic Hammers

Figure 25-9 illustrates two hydraulic hammer types. The HBM hammers feature a ram which contains hydraulic fluid and nitrogen acting as a cushion to limit the impact force delivered to the pile.[9,10] The characteristics of the impact force can be altered by remote control of the amount of hydraulic fluid in the ram cylinder. The Menck hydraulic hammers utilize a solid steel ram and a flexible steel pilecap designed to limit impact forces to approximate those produced by a steam hammer of the same size using a Bongassi hardwood cushion.[11] Both hammer types are double-acting and are capable of operating underwater, with some models designed to be guided through skirt pile guides. Hydraulic fluid under high pressure is used to force a piston or set of pistons, and in turn the ram, up and down. Remote diesel or electric powered pumps supply the required hydraulic energy to move the piston (or pistons). Table 25-4 lists the rated energy, stroke and weight data operating pressure, oil flow requirements, and rated blows per minute for typical hydraulic hammers with more than 50,000 ft-lbs of rated energy.

25.5.5 Comparison of Hammer Types

Considerations that affect the selection of hammer type for a particular application include availability, restrictions imposed by the installation vessel or by construction methods, and required hammer size (broadly expressed in terms of rated energy). Availability may depend on geographic location and/or contractor chosen for installation. Steam and diesel hammers are widely available, while hydraulic hammers have limited availability in some areas. The installation vessel may impose restrictions on hammer weight that can be lifted or on support capabilities such as power or steam supply capacity. Diesel hammers tend to be lighter than steam or hydraulic hammers, and do not require an external power source, so support requirements are less than for steam or hydraulic hammers. Most installation vessels are equipped with steam boilers to power air/steam hammers, but hydraulic hammers require fairly large power pumps for operation.

Some construction methods require special hammer types. For example, hammers with underwater driving capabilities are required when driving skirt piles without the use of followers. Hammer size requirements may eliminate some hammer types from consideration due to inappropriate ranges of rated energy. Air/steam hammers with up to 1,800,000 ft-lbs rated energy are available, and models with 300,000 to 500,000 ft-lb are common. Diesel hammers are manufactured with up to 280,000 ft-lbs rated energy, with models of 80,000 to 150,000 ft-lbs widely available. Hydraulic hammers are available with up to 1,200,000 ft-lbs rated energy.

Air/steam hammers are the most commonly used type for driving foundation piles of offshore structures. Diesel hammers are used mostly for driving of conductors (drive pipes) from a small vessel or with a drilling rig, although the larger models can be used to drive main foundation piles. Hydraulic hammers are not presently common, but their use is increasing, especially for underwater driving in deepwater applications.

25.5.6 Selection of Hammer Size

The selection of hammer size strongly affects cost effectiveness of pile installation by determining the success of driving to planned penetrations, the rate of pile advancement, and the cost of pile installation equipment (hammers, remedial advancement equipment, and possibly installation vessel) required. The choice of hammer sizes to be used at various stages of installation affects the pile schedule and add-on lengths and thus impacts the pile handling and connection time requirements. The major influence is the ability of the hammer to drive the pile as planned, without damaging the pile, thus avoiding the need for remedial advancement measures.

Selection of hammer size is based on experience with similar situations and on analysis of driving for each particular site. Prior experience in a given area yields broad guidelines for hammer size selection, and suggests ranges of parameters for use in the analysis. Expected ranges of driving resistance are estimated based on soil boring information and comparisons of predicted and measured blow counts at nearby or similar sites.[12,13] Drivability studies indicate the capabilities of different driving systems to overcome the estimated driving resistance for a particular pile-soil system. The most commonly used analysis tool for this purpose is a one-dimensional stress wave equation computer program.[14,15]

In a wave equation analysis, the hammer, cushion, pilecap and pile are modeled as a series of masses and springs. The ram of a steam hammer is normally represented by a mass with an initial velocity; the cushion is modeled as a weightless spring; and the pilecap is represented as a mass of infinite stiffness. The pile is divided into segments, each represented by a mass and a spring. The soil resistance is distributed along the side of each element below the mudline and at the tip of the pile. An elastic-plastic (bi-linear) spring model is used for static soil resistance while the dynamic soil resistance is modeled as linear viscous damping. Wave equation programs and drivability studies are discussed in detail in Chapter 23.

Parameter selection is critical to sucessful modeling of hammers, cushions, piles, and soil for predicting drivability. The parameters that characterize the driving system are the rated energy and efficiency of the hammer, and the stiffness and weight of driving accessories (cushion, pilecap, and followers). The choice of these parameters affects the character of the force pulse imparted to the pile and influences predicted pile drivability. Pile schedule, material type, and length together with soil stratigraphy and strength determine (1) how the force pulse is modified as it travels the length of the pile, and (2) the associated pile motions in the soil.

The modeling of driving system parameters for steam hammers has been studied in detail using wave equation analyses and field measurements of the force input to piles during driving operations.[16,17] From parametric studies of driving system models, some general tendencies are apparent:

- Ram weight and shape influences primarily the width of the force pulse with secondary effects on the magnitude of the force.
- Stroke is a primary factor in the magnitude of maximum forces.
- Cushion stiffness influences both the magnitude and width of the force pulse.
- Pilecap and follower weights have secondary although not negligible influence on the magnitude and width of the force pulse.

Statistical descriptions of estimated driving system parameters from field measurements are presented in Section 25.6.7. These descriptions indicate the extreme variability of driving system performance and can be used to estimate driving system parameters for widely varying conditions.

Initial selection of hammer sizes for analysis with drivability studies is usually based on experience with similar installation circumstances. In the absence of such experience, preliminary wave equation analyses can be used to choose hammer sizes for study. Figure 25-10 gives broad guidelines for preliminary hammer size selection as a function of soil type, expected driving resistance, and pile area in the form of blow count vs. resistance curves. These curves are based on the nominal driving system, pile, and soil parameters shown in the figure, and are not intended to model any specific driving systems or site conditions. However, the information shown does indicate typical influences of hammer size, pile area, and soil type on the driving resistance that can be overcome. Since the effect of variation in driving system, pile, and soil parameters is not easily accounted for otherwise, specific hammer-pile-soil systems should be individually analyzed with a wave equation analysis.

25.6 DRIVING OPERATIONS

This section discusses hammer placement, pile penetration under self-weight and hammer weight, typical driving characteristics for several soil types, and presents pile installation case histories for various regions of the world. Criteria and consequences of pile refusal, and quality control of pile driving are also discussed.

25.6.1 Hammer Placement

Placement of the pile driving hammer in fair weather is a routine operation which is carried out with little difficulty. Unfavorable weather conditions can complicate the situation, since the ability of an installation vessel to drive piles in rough weather is limited by the response characteristics of the vessel and the height, direction, and period of seas or swells. As vessel motions increase, the time required to properly position the hammer over the pile, and the risk of damage to the pile and hammer are also increased.

The hammer is supported by the offshore leads which are lifted by the crane of the installation vessel (Figure 25-11a). A tugger line connected to the lower portion of the leads aids in controlling movements of the hammer. The bell of standard offshore leads acts as a stabbing guide when positioning the hammer over a pile and is especially helpful in rough weather. Once the leads are positioned over the pile (Figure 25-11b), the hammer (resting in the bottom of the leads) is lowered until the pilecap contacts the pile top. The impact force experienced by the pile during initial placement is dependent on the relative velocity between hammer and pile and generally increases with vessel motion. When properly positioned, the pilecap aligns the hammer and tends to "lock" it into place. The leads are then lowered in preparation for pile driving (Figure 25-11d). Failure to align the pilecap concentrically with the pile will result in the pile top spanning more than one step of the pile cap, causing damage to the pile top when driving begins. Pile tops must be squarely cut to avoid damage to the pile top or loss of energy transfer. To position the pilecap properly, it is sometimes necessary to tighten the tugger line and swing the top of the offshore leads to align the hammer with the batter of the pile (Figure 25-11c). When the pilecap locks, the hammer and pile top will move together, possibly resulting in large arresting forces.

Each add-on should be designed to prevent bending or buckling failure for placement and in-place (operating) conditions. Static in-place stresses are typically calculated using an empirical procedure similar to that described in API RP 2A,[18] in which the full weight of hammer and leads in place is assumed to be supported by the pile. Dynamic stresses associated with driving are estimated from wave equation analyses. It should be noted that stress concentrations due to wall thickness changes or due to high end bearing resistance can control the wall thickness required for pile driving. When sizing add-on length or wall thickness for pile-hammer combinations with which installation experience is limited, more refined analysis of placement and in-place conditions may be appropriate to avoid pile damage. A rational analysis of placement and operating stresses will involve the effects of pile add-on length, wall thickness schedule, batter, weight and dimensions of hammer and leads, and impact stresses associated with initial placement and arrest of lateral motion as well as driving stresses.

25.6.2 Pile Penetration without Driving

One of the hazards associated with early stages of pile installation occurs when the bearing capacity of the pile at a particular penetration is less than the weight of the pile. In such cases, the pile will "run", or penetrate the soil in an essentially uncontrolled manner, until the load can be supported. This tendency is common in areas with relatively soft soils at or near the mudline. If competent soils are present at the mudline and the bearing capacity of the pile increases fairly rapidly with depth, the tendency for a pile to run is reduced. However, if pile bearing capacity is near the weight of the pile and increases slowly, or if there is a decrease in capacity at some depth (for example a soft clay beneath a relatively strong crust), running may be a hazard. Some penetration under pile self-weight is normal. This initial penetration can be estimated by comparing the weight of the pile to the calculated pile capacity (using remolded soil shear strength in clays).

If the bearing capacity of the pile is nearly equal to its self-weight and a hammer is placed on top of the pile, the pile may plunge without warning. The same situation can occur during apparently normal driving if the pile tip penetrates into a void or substantially weaker material and reduces bearing capacity below that required to support pile and hammer weight. The sudden removal of support for the hammer can be dangerous, since the hammer weight will cause a dynamic load to be applied to the crane of the installation vessel. For a hammer that is near the limit of lifting capability for a particular crane, such a load could result in damage to the boom. If the braking system cannot control the hammer's fall, damage will occur to the jacket. If the pile top plunges past the top of the jacket leg, remedies are time consuming, costly, and can be dangerous.

25.6.3 Effect of Stratigraphy on Blow Counts

Driving records show that soil stratigraphy markedly affects driving characteristics of a given pile-hammer combination. This section discusses the general driving behavior patterns of piles for clays, sands, and silts.

Piles driven into clay soils generally meet resistance to driving that is dominated by skin friction, with minimal tip resistance. The result is that sudden blow count changes due to soil resistance are unusual. Another characteristic of clays is a reduced soil-pile adhesion during driving (as discussed in Section 25.2.3). As a result, blow counts may remain constant or even decrease with penetration in a highly sensitive clay. Predictability of pile driving in clay soils with current analytical techniques is relatively good, with the most significant uncertainties related to the degree of remolding reduction of adhesion that occurs during driving and the rate of setup during delays.

Resistance to driving in sands is characterized by much higher tip resistance than in clays. The resistance encountered when driving through sands is closely related to the density of the material. Density increases resulting from driving a closely-spaced group of piles (or conductors) may progressively increase the resistance encountered during driving of the piles. The tip resistance encountered in dense sands may in some cases be limited only by the crushing strength of individual sand particles underneath the pile wall. Thus, a resistance higher than that predicted by static pile capacity computations may be encountered. End bearing resistance is markedly influenced by the formation of a soil plug during driving, which is more common in sands than in clays. A reduction in skin friction during driving is not usually experienced in sands, and thus, setup is not normally a problem, but blow counts are typically higher than in clays. Exceptions have been noted in calcareous sands. Predictability of pile driving in sands by present analytical methods is less reliable than in clays due to 1) uncertainties in prediction of driving resistance, especially in predicting plug formation, and 2) inadequate mathematical modeling of soil behavior for sands. Further discussion of problems with prediction of pile drivability in sands is available in the literature.[19]

Driving in silts is even less predictable than that in sands. In silts a fairly high tip resistance can be expected and setup is not usually significant. Experience has shown more variability in blow counts between piles of a particular platform in silts than in either clays or sands, with a corresponding decrease in the reliability of drivability predictions.

In most cases, piles are driven through a soil stratigraphy which is not uniform. Changes in blow count can be expected as the pile tip penetrates into a new soil strata. In some cases, changes in driving system performance can be confused with changes in soil strata or resistance. Rates of set-up in clay soils can be increased by the draining effect of nearby or interlayered granular materials. Resistance to driving can be very high after long delays when the pile tip is located in a granular material overlain mostly by clays.

Records of pile installation at nearby or similar sites are helpful when planning pile driving operations. Most records are proprietary, but installations in many parts of the world have been documented in the literature. Table 25-5 lists some case histories by location.

25.6.4 Pile Refusal

Theoretically, refusal with a particular driving system is the point at which no further penetration of the pile can be achieved by driving with that system. Practically, refusal must be defined in terms of a minimum rate of penetration where further advancement of the pile by driving is no longer feasible because of the time required, or possible damage to hammer or pile. Each hammer-pile-soil combination will have a different rate of increase in blow count as driving resistance increases; this relationship can be predicted in drivability studies. Useful alternatives to driving for advancing the pile depend on soil conditions, pile make-up and installation vessel capabilities. Anticipated drivability and available advancement alternatives along with economics should be considered when defining refusal to maximize the cost effectiveness of pile installation.

The application of refusal criteria can have an effect on the penetration at which refusal is defined. Refusal can be defined for a particular hammer as (1) the first unit of penetration for which a blow count rate maximum is exceeded, (2) a given unit of penetration for which a blow count rate maximum is exceeded, or (3) the penetration at which a maximum blow count total is recorded for less than a given number of units of penetration. Option 1 is useful when easy driving is anticipated prior to encountering a strata where pile damage might occur. Option 2 is commonly used when a relatively thin strata of dense material is to be penetrated. Option 3 is desirable when refusal is expected in a massive layer of dense material. Typically, different criteria is specified for restarting a pile after delays. In addition, a maximum number of blows is usually specified for a minimal penetration (for example, 800 blows for less than 6 inches of penetration) to protect the pile and driving system from damage in case of sudden refusal. A minimum driving system performance level is sometimes specified along with refusal blow count criteria. In contracts where remedial measures are considered extra

TABLE 25-5

CASE HISTORIES

thor	Location	Hammers Used	Pile Size	Penetration
	Gulf of Mexico			
yfield, et al[20]	Mississippi Canyon Area	HBM 3000	84"Ø	450 ft
ng[21]		Menck 3000	96"-168"Ø	150 ft
		Vulcan 560 & 5100	125"Ø	340-360 ft
ick, et al[22]	Main Pass Area	Vulcan 560 & 5150	144"Ø	200 ft
	Main Pass Area	Vulcan 560 & 5150	72"Ø	500 ft
	Main Pass Area	Vulcan 030	22"Ø	450 ft
x & Christy[23]	South Marsh Island Area	HBM 500	24"Ø	310 ft
rora[3]	High Island Area	Vulcan 060	42"Ø	385 ft
	High Island Area	Vulcan 060	48"Ø	232 ft
	Eugene Island Area	MKT OS-60	48"Ø	275 ft
	East Cameron Area	Vulcan 040	42"Ø	207 ft
	Hign Island Area	Vulcan 020, 040, 060 & Menck 3000	42"Ø	267 ft
	High Island Area	Vulcan 040 & 560	42"Ø	219 ft
	Vermilion Area	Vulcan 040 & 560	48"Ø	324 ft
	Ship Shoal Area	Vulcan 020	42"Ø	219 ft
	West Cameron Area	Vulcan 040 & 060	48"Ø	240 ft

TABLE 25-5 (continued)

CASE HISTORIES

uthor	Location	Hammers Used	Pile Size	Penetration
tockard[24]	Offshore Louisiana	Vulcan 040	42"Ø	200 ft
	Offshore Louisiana	Vulcan 020 040 & 060	48"Ø	325 ft
	Offshore Louisiana	Vulcan 040 & 060	42"Ø	280 ft
	Offshore Louisiana	Vulcan 340 & 560	48"Ø	340 ft
	Offshore Texas & Louisiana	Vulcan 340 & 560	42'Ø	280 ft
	Offshore Texas	Menck 750 & 1800	36"Ø	240 ft
	Offshore Texas	Conmaco 020	30"Ø	140 ft
	Offshore Texas	Vulcan 020	30"Ø	170 ft
	U. S. West Coast			
ınningham & ıughton[25]	Santa Barbara Channel	Vulcan 340, 3100 & Menck 4600	48"Ø	340-375 ft
		Vulcan 340 & 3100	54"Ø	260 ft
	U. S. East Coast			
ng[26]	Offshore North Carolina	Vulcan 040, 060 & 560	42"Ø	220-275 ft

TABLE 25-5 (continued)

CASE HISTORIES

ıthor	Location	Hammers Used	Pile Size	Penetration
	North Sea			
ırning, et al[27]	Heather Field	Vulcan 560, Menck 4600, 8000, 12500 & MRBU 6000	60"Ø	102-140 ft
›ung,et al[28]	Thistle Field	Vulcan 560	54"Ø	98 ft
ıvivier & ːnstock[29]	Piper Field	Vulcan 060, 560 & Kobe 150	60"Ø	110-130 ft
ıtton, et al[30]	Forties Field	Menck 3000 & 7000	54"Ø	240 ft
.rsch, et al[31]	Forties Field	Menck 2500, 2500SL & 7000	54"Ø	150 ft
	Forties Field	Menck 7000	54"Ø	240 ft
x, et al[32]	West Sole Field	Vulcan 200L	30"Ø	40-60 ft
	West Sole Field	MKT OS-60 & Menck 1500	36"Ø	55 ft
jayvergiya, al[33]	Rough Field	Menck 1500, 2500 & 2500SL	36"Ø	160 ft
	Rough Field	Menck 2500, 2500SL, 7000 & 8000	36"Ø	130 ft
	India			
ırwal, et al[34]	Arabian Sea	Vulcan 560	48"Ø	193-270 ft
	Arabian Sea	Vulcan 560	48"Ø	160-260 ft

TABLE 25-5 (continued)

CASE HISTORIES

uthor	Location	Hammers Used	Pile Size	Penetration
		Arabian Gulf		
ettgast[35]	Offshore Abu Dhabi	Delmag D-55	42"Ø	14-42 ft
agaya, et al[36]	Offshore Qatar	Menck 1500 & 3000	30"Ø	278 ft
		Menck 1500 & 3000	36"Ø	306 ft
	New Zealand			
ennie & Fried[37]	Maui Field	Menck 3000 & 4600	48"Ø	230 ft
		Delmag D-46, D-62 & Menck 1800	26"Ø	240 ft

work, the definition of refusal can affect the total installation cost dramatically by determining when extra work begins.

A widely quoted and generally practical rate of penetration for defining refusal is 300 blows per foot.[18] At this blow count, a typical hammer would drive a pile one foot in 6 to 8 minutes. For most hammer-pile-soil systems, wave equation analyses and experience both indicate that very little additional driving resistance can be overcome with higher blow counts.

When a pile reaches refusal at less than design penetration, an important and sometimes difficult decision must be made as to pile adequacy. In many cases, this question is evaluated in terms of how near the penetration at refusal is to the design penetration, but adequacy of the foundation is more rationally decided on the basis of acceptable risk (in terms of factor of safety against failure), confidence in capacity predictions, and cost of remedial measures. Required pile bending and shear capacity near the mudline may also affect acceptability. In considering the question of adequacy, the possible detrimental effects of remedial advancement techniques must also be considered, since load bearing capacity (not penetration) is the rational criteria for evaluating pile acceptability. Remedial measures are discussed in Section 25.7.

The high cost of advancing a pile after refusal is reached has resulted in much emphasis being placed on evaluation of pile adequacy in recent years.[31,38,39,40] Pile installation monitoring data (discussed in Section 25.6.6) quantifies driving system performance (hammer efficiency and cushion properties) and thus allows more meaningful interpretation of recorded blow counts. The result is more information available for assessing the adequacy of a refused pile. Although the evaluation of pile bearing capacity is still based mainly on static pile capacity predictions, the soil stratigraphy shown by the original exploratory boring can be checked and better estimates can be made of driving resistance when monitoring data is available.[41] Setup can be evaluated if instrumented piles are redriven after periods of delay. The additional confidence in pile capacity predictions is recognized by reduced conservatism when determining acceptable factors of safety.

25.6.5 Quality Control

Quality control of driving operations normally consists of visual inspection of installation procedures to assure compliance with the installation plan, and sometimes includes measurements of pile and/or hammer behavior with specialized equipment (discussed in Section 25.6.7). The intent of quality control measures is 1) to increase confidence in foundation adequacy by confirming that piles are installed in a manner consistent with design assumptions, and 2) to provide a record of pile installation for reference when planning subsequent pile installations for the same platform or at nearby locations, and when planning modifications to the platform.

Visual inspection of driving operations verifies pile length, diameter, and wall thickness schedule; hammer sizes used, performance, and general operating condition; blow counts experienced during driving; delays during installation; and circumstances accompanying refusal, if it occurs. The length and wall thickness schedule of individual pile add-ons, and cutoff lengths are checked to assure that the installed pile schedule corresponds to that shown on the plans. Hammer sizes for each add-on are compared to those shown in the installation plan to avoid overstressing of the pile due to hammer weight and to confirm correct rated hammer energy. Recorded blow counts and delays to driving are verified to insure accuracy. This information is also used in planning strategies for efficient installation of subsequent piles of the platform. If refusal is encountered, factors such as delays, hammer operating condition, and sudden changes in blow count are carefully documented in an effort to determine the cause of refusal.

Pile driving records should be maintained for all pile (and conductor) installations. Complete driving records contain the following information for each pile:

1. Dimensional data (pile diameter and wall thickness schedule),
2. Measured pile add-on lengths and cutoffs,
3. Blow count record, including
   a. Penetration under pile self-weight,

b. Penetration under weight of hammer,

c. Hammers used (referenced to penetration),

d. Blow count for each foot of driving,

e. Delays to driving (length of time and cause), referenced to penetration,

4. Comments or records of hammer and pile performance, including

a. Monitoring data, if available,

b. Observed hammer stroke and speed of operation (blows per minute),

c. Steam pressure,

d. Pile damage, if any,

5. Soil plug and internal water level elevations before and after driving of each pile section, and

6. Documentation of remedial measures and grouting.

Figure 25-12 illustrates a blow count record in graphical form showing pile schedule, hammers used, delays, and soil stratigraphy.

25.6.6 Performance Monitoring

Methods have been developed in recent years to monitor pile and hammer performance during pile driving using specialized measurement and recording systems. Current measurement and evaluation techniques are based on research efforts developed at several universities in the 1960's. Although directed to bearing capacity predictions for onshore applications, the research established that hammer efficiency and cushion properties varied substantially during the course of normal pile driving operations. Performance monitoring for offshore pile installations began in the North Sea in the 1970's.

Performance monitoring has three primary benefits. First, driving system performance is quantified, which allows better control over the installation process since ineffective components can be repaired or replaced before a critical situation arises. Thus, the risk of refusal due to an inefficient driving system is reduced. Second, additional input is provided for assessment of pile adequacy since driving system performance is eliminated as a variable when interpreting blow count records to evaluate soil stratigraphy and driving resistance distribution.[42] Third, the records of performance form a data base that is available for use in predictions of drivability for future pile installations and in studying the reliability of such predictions.

The most valuable applications for performance monitoring are in situations where difficult driving is expected, or where the reliability of analytical predictions is uncertain. These uncertainies may be due to limited experience in a new area of exploration or with new equipment, or due to variable soil conditions.[43] In such situations, monitoring reduces the risk of refusal associated with inefficient driving systems and provides a more complete picture of pile adequacy if refusal is encountered.

25.6.7 Hammer Performance Statistics

This section contains statistical descriptions of measured performance parameters for steam hammers. These statistics were compiled by TERA, Inc. from performance monitoring data gathered on 32 platform installations from 1977 to 1980.[44] Statistics are presented for hammer efficiency, cushion stiffness, and cushion coefficient of restitution. Typical histograms of hammer efficiency, cushion stiffness, and cushion coefficient of restitution are presented to indicate the distribution of measured data. The values of parameters that form the data base were estimated by matching measured force-time characteristics with predicted characteristics from wave equation analyses of hammer blows. These statistics and histograms are useful for estimating driving system parameters for use in drivability studies.

The data base on which the statistics and histograms are based includes a broad range of hammer sizes, cushion materials, and pile configurations. Data for 13 hammer types ranging from 60,000 to 750,000 ft.-lbs. of rated energy is in the data base. Cushion types are bongassi hardwood, asbestos and steel plate sandwich, and wire rope and steel plate sandwich cushions. Pile sizes range from 36 to 144 inches in diameter, with wall thicknesses from 1 to 4 inches. A total of 2439 interpreted blows are included in these statistics.

Figure 25-13 shows a histogram of hammer efficiency (percent of rated energy) for all measurements in the data base and statistics describing the distribution. Hammer efficiency is defined here as a function of ram velocity only, by the following relationship:

$$V = (2ghe)^{1/2}$$

or

$$e = V^2/2gh,$$

where V = ram velocity at impact,

g = acceleration of gravity,

h = nominal stroke,

e = efficiency,

and thus is independent of cushion properties. Few significant statistical differences are noted between the hammer types in the data base.

Figure 25-14 shows a typical histogram of cushion stiffness for bongassi hardwood. Similarly, Figures 25-15 and 25-16 show typical histograms for asbestos-concrete/steel plate and wire rope/steel plate sandwich cushions, respectively. Statistics describing the distribution of stiffness for these cushions with several hammer sizes are given in Table 25-6. Typical time histories of stiffness for the three cushions are shown in Figure 25-17 to illustrate differences in cushion behavior with time.

A histogram of cushion coefficient of restitution for all measurements is shown in Figure 25-18. Statistics for several hammer and cushion types are listed in Table 25-7.

(25.6.8)

TABLE 25-6

CUSHION STIFFNESS STATISTICS

Cushion Type	Hammer Type	Mean (Kips/in)	Standard Deviation (Kips/in)	Coefficient of Variation
Bongassi	Menck 1800	27,400	8,700	0.32
"	" 3000	47,500	15,300	0.32
"	" 4600	56,600	14,700	0.26
"	" 7000	185,000	54,900	0.30
Asbestos/				
Steel	Vulcan 020	7,790	800	0.10
"	" 040	21,400	9,900	0.46
"	" 060	37,200	8,900	0.24
"	" 560	52,800	25,000	0.47
"	" 5150	60,700	19,000	0.31
Wire rope/				
Steel	" 040	26,100	13,800	0.53
"	" 060	48,500	10,300	0.21
"	" 560	119,000	78,400	0.66
"	" 5100	141,000	61,900	0.44
"	" 5150	183,000	51,000	0.28

(25.6.8)

TABLE 25-7

CUSHION COEFFICIENT OF RESTITUTION

Cushion Type	Mean	Standard Deviation	Coefficient of Variation
Bongassi	0.68	0.10	0.15
Asbestos/Steel	0.71	0.09	0.13
Wire rope/Steel	0.74	0.09	0.12

25.7 SUPPLEMENTAL PROCEDURES

When advancement of a pile by driving alone becomes impractical and more penetration is required to obtain a satisfactory factor of safety against failure, there are several alternative installation methods that can be used to obtain additional pile penetration and/or capacity.[45,46,47] There are four basic alternatives to driving alone: (1) removal of the soil plug, (2) removal of soil below the pile tip, (3) use of insert piles, and (4) use of belled footings. Other supplemental measures, such as electrosmosis[48], have been developed but are not in widespread use offshore.

The use of supplemental procedures normally implies significantly higher cost and sometimes results in less confidence in foundation adequacy compared to piles installed by driving alone. However, when supplemental procedures are carefully planned and executed with strict quality control, the probability of degredation of pile capacity is reduced. In some cases, creative use of alternative foundation installation procedures that take advantage of unique site conditions or structure configuration can result in cost savings compared to pile driving alone.

The need for supplemental pile advancement procedures is assessed in the design phase as a part of pile drivability studies. Indications of a possible hard driving situation call for planning of remedial measures in case of pile refusal at a penetration where pile capacity is unacceptable. Depending on the probable need and the economic penalties for delays, required equipment may be specified to be mobilized to the installation site with the installation vessel to if remedial measures are required.

Attainment of additional pile load capacity, not simply penetration, is the reason for implementing remedial measures; achievement of a design penetration by arbitrary means will not necessarily result in satisfactory foundation performance. Therefore, the pile tip elevation required to obtain a particular level of confidence for a specified load capacity is dependent on the method of installation. The following paragraphs describe common supplemental advancement techniques, situations in which they are useful, and associated risks.

25.7.1 Soil Removal Methods

Jetting and drilling are two commonly used soil removal methods. In jetting, a pump delivers high-pressure water through a drill string and nozzle to loosen the soil. An air lift return is usually employed when removal of jetted material from the pile is desired. Airlifting consists of introducing compressed air above the jet nozzle to lift the loosened soil up a return pipe and out of the pile. Jetting is commonly used in sands and clays.

Drilling is usually accomplished with a crane-supported hydraulic power swivel, a pile top drilling rig, or a skid-mounted rig. Drilling is used when hard clays, cemented materials or rock must be removed from the pile. Occasionally, jetting and drilling with a power swivel are used in combination to remove strong clays.

25.7.2 Removal of Soil Plug

The most common supplemental advancement procedure is to remove the soil plug inside the pile by controlled jetting or drilling (Figure 25-19a), followed by redriving of the pile. This technique is effective when end bearing resistance due to plugging action is a significant portion of the the driving resistance. Airlift jetting is more economical for removal of most clays and sands; drilling is used when hard clays, cemented materials or rock must be removed from the pile.

If end bearing is not a major contributor to soil resistance or if the pile tip bears on a material which provides high bearing resistance regardless of the soil plug (such as rock), removal of the soil plug is not likely to be an effective supplemental advancement technique. Similarly, if much of the pile is embedded in clays where setup is anticipated, part or all of the reduction in driving resistance from removal of the soil plug may be eliminated by increased resistance due to setup during delays for jetting or drilling operations.

Behavior of the soil plug when driving is resumed is not presently predictable. Driving records indicate that at deep penetrations it is possible for increases in end bearing resistance (due to either soil compaction or plug formation) to occur fairly soon after redriving of the pile begins, causing refusal. In such cases several cycles of plug removal and driving are commonly required to advance the pile to the desired penetration. After redriving is completed, the plug length inside the pile should be checked. If computations indicate that the plug alone is not long enough to mobilize full end bearing, a grout plug should be installed in the pile.

Controlled removal of the soil plug within the pile poses relatively few risks of decreasing pile capacity if strict control is exercised. Removal of the plug by jetting closer to the pile tip than 10 ft. is generally not recommended because of the risk of a reduction in pile capacity. Special site conditions may present risks when the soil plug is removed. For example, in granular strata with excess porewater pressures, the removal of the soil plug overburden may cause a flow of water and soil into the pile which would reduce the density or strength of the material supporting the pile.

25.7.3 Removal of Soil Below Pile Tip

In cases where end bearing resistance is too high even after removal of the soil plug, controlled removal of material below the pile tip may be a reasonable supplemental advancement alternative. Drilling of a pilot hole smaller in diameter than the pile (Figure 25-19b) is the normally recommended procedure for removal of the material. Pilot hole diameters are usually about 75 percent of pile diameter in clay and about 50 percent of pile diameter in sand[49] to minimize reductions in skin friction resistance. The pile is driven as soon as possible after drilling is completed to reduce the time the hole remains open. As in the case when only the soil plug is removed, several cycles of drilling and driving may be necessary to achieve the desired penetration. If driving is still not possible after removal of this material, an insert pile (discussed below) may be required. A grout plug is placed in the hole after redriving to assure full end bearing capacity.

Close supervision and carefully planned procedures are necessary to assure a straight hole that remains stable.[45] Alignment of the hole is determined by centralizer effectiveness, drill string stiffness, pile batter, and length. Hole stability is affected by hole diameter, soil type, drilling mud characteristics, and return velocity. A stable, straight hole is necessary to prevent loss of skin friction due to the pile wall being located in the void created by drilling.

Jetting or uncontrolled drilling below the pile tip is not recommended. Although these methods can allow relatively easy redriving of the pile, tests have shown that little confidence can be placed in the load capacity.[46] Uncontrolled removal of soil from below the pile tip results in unpredictable changes in the supporting soils and, in many cases, a reduction in pile capacity.

25.7.4 Insert Piles

Insert piles are used to increase total pile penetration when further driving of the main pile is not feasible, and/or to increase pile moment capacity, especially near the mudline. Inserts are frequently used after other supplemental methods have proven inadequate.

Insert piles are smaller than and driven (or drilled and grouted) inside a main pile as illustrated in Fig. 25-19c. If insert drivability is a problem, the soil plug is usually removed from the main pile prior to driving of the insert. Alternatively, inserts may be grouted in an oversized drilled hole. Centralizers are used to provide a uniform annulus for the grouted connection between main and insert piles.

25.7.5 Belled Footings

Underreamed or belled footings (Figure 25-19d) can be practical in some applications to increase pile capacity without additional driving. After driving is completed, a footing is drilled and underreamed, and reinforced concrete is used to fill the footing. Relatively high foundation loads can be supported in this manner with minimal pile penetration.[49] Placement of reinforcement and concrete for deep footings must be carefully executed and monitored to assure the desired configuration.

References

1. Randolph, M. F., Carter, J. P. and Wroth, C. P. Driven Piles in Clay - The Effects of Installation and Subsequent Reconsolidation. Geotechnique 29(4): 361-393, 1979.

2. Bozozuk, M., Fellenius, B. H. and Samson, L. Soil Disturbance from Pile Driving in Sensitive Clay. Canadian Geotechnical Journal 15(3): 346-361, 1978.

3. Aurora, R. P. Case Studies of Pile Setup in the Gulf of Mexico. 12th Annual Offshore Technology Conference Paper No. 3824, 2: 281-290, 1980.

4. Boyadjieff, G., House, H. and Roussel, H. A. New Offshore Pile Aligning System. 9th Annual Offshore Technology Conference Paper No. 2839, 2: 257- 266, 1977.

5. Recommended Practice for Ultrasonic Examination of Offshore Structural Fabrication and Guidelines for Qualification of Ultrasonic Technicians. API RP2X, First Edition, March 1980.

6. Drouin, A. H. A New Method for Fast and Economical Connection of Large-Diameter Pipe. Proceedings of the 49th Annual Fall Meeting of the Society of Petroleum Engineers of AIME Paper No. SPE 5040, 1974.

7. Riemert, L. E. and Hettinger, F. L. New Mechanical Pile Connector. Ocean Industry 14(3): 53-56, 1979.

8. Gendron, G. J., Holland, H. A. N. and Ranft, E. V. Underwater Pile Driving at Maui Field. 10th Annual Offshore Technology Conference Paper No. 3270, 3: 1887-1894, 1978.

9. Jansz, J. W. North Sea Pile Driving Experience with a Hydraulic Hammer. 9th Annual Offshore Technology Conference Paper No. 2840, 2: 267-274, 1977.

10. Jansz, J. W. Underwater Piledriving, Today's Experiences and What is About to Come. Proceedings of the Second International Conference on the Behavior of Off-Shore Structures Paper No. 35, 1: 447-474, 1979.

11. Rusen, J. New Slender Hydraulic Hammer Saves Time Offshore. Petroleum Engineer International 12(52): 114, 1980.

12. Toolan, F. E. and Fox, D. A. Geotechnical Planning of Piled Foundations for Offshore Platforms. Proceedings of the Institute of Civil Engineers, Part 1, 62: 221-294, 1977.

13. Kindel, C. E. Mechanism of Soil Resistance for Driven Pipe Piles. Ports '77 (ASCE) 2: 251-268, 1977.

14. Smith, E. A. L. Pile Driving Analysis by the Wave Equation. Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers 86(SM4): 35-61, 1960.

15. Holloway, D. M., Audibert, J. M. E. and Dover, A. R. Recent Advances in Predicting Pile Drivability. 10th Annual Offshore Technology Conference Paper No. 3273, 2: 1917-1924, 1978.

16. Samson, C. H., Hirsch, T. J. and Lowery L. L. Jr. Computer Study of Dynamic Behavior of Piling. Journal of the Structural Division, Proceedings of the American Society of Civil Engineers 89(ST4): 413-449, 1963.

17. Naughton, H. R. and Miller, T. W. The Prediction and Subsequent Measurement of Pile Driving Behavior at the Hondo Platform in Santa Barbara. European Offshore Technology Conference Paper No. 11, 1: 95-100, 1978.

18. Recommended Practice for Planning Designing, and Constructing Fixed Offshore Platforms. API RP 2A, 12th Edition, January 1981.

19. Mes, M. J. Pile Driveability is Unpredictable in Sand or Silt Foundation Strata. Petroleum Engineer 48(12): 76-92, 1976.

20. Mayfield, J. G., Strohbeck, E. E., Olivier, J. C. and Wilkins, J. R. Installation of the Pile Foundation for the Cognac Platform. 11th Annual Offshore Technology Conference Paper No. 3497, 2: 1221-1232, 1979.

21. Lang, G. R. Jr. Predicting the Drivability of Large Diameter Offshore Piling. 12th Annual Offshore Technology Conference Paper No. 3827, 3: 301-312, 1980.

22. Irick, J. T., Birdwell, J. R., Henkhaus, E. J., Hayes, D. A. and Mannina, G. A. Design and Installation of the Main Pass 72 Mud Slide Resistant Platform. 12th Annual Offshore Technology Conference Paper No. 3878, 4: 223-230, 1980.

23. Cox, B. E. and Christy, W. W. Underwater Pile Driving Test Offshore Louisiana. 8th Annual Offshore Technology Conference Paper No. 2478, 1: 611-616, 1976.

24. Stockard, D. M. Case Histories - Pile Driving in the Gulf of Mexico. 11th Annual Offshore Technology Conference Paper No. 3443, 2: 737-746, 1979.

25. Cunningham, G. R. and Naughton, H. R. Design and Installation of the Piling Foundation for the Hondo Platform in 850 Feet of Water in the Santa Barbara Channel. 9th Annual Offshore Technology Conference Paper No. 2843, 2: 291-298, 1977.

26. Ling, S. C. Four Offshore Piled Structures Installed on Outer Continental Shelf of the Atlantic Coast of the United States. 10th Annual Offshore Technology Conference Paper No. 3088, 1: 345-352, 1978.

27. Durning, P. J., Rennie, I. A., Thompson, J. M., and Ruckstuhl, E. J. Installing a Piled Foundation in Hard, Overconsolidated North Sea Clay for the Heather Platform. European Offshore Petroleum Conference Paper No. 46, 1: 375-382, 1978.

28. Young, A. G., Sullivan, R. A. and Rybicki, C. A. Pile Design and Installation Features of the Thistle Platform. European Offshore-Petroleum Conference Paper No. 12, 1: 101-110, 1978.

29. Duvivier, S., and Henstock, P. L. Installation of the Piled Foundations and Production Modules on Occidental's Piper A Platform. Proceedings of the Institution of Civil Engineers, Part 1, 66: 407-436, 1979.

30. Sutton, V. J., Rigden, W. J., James, E. L., St. John, H. D. and Poskitt, R. J. A Full Scale Instrumented Pile Test in the North Sea. 11th Annual Offshore Technology Conference Paper No. 3489, 2: 1117-1133, 1979.

31. Hirsch, T. J., Koehler, A. M. and Sutton, V. J. R. Selection of Pile Driving Equipment and Field Evaluation of Pile Bearing Capacity During Driving for the North Sea Forties Field. 7th Annual Offshore Technology Conference Paper No. 2247, 2: 37-49, 1975.

32. Fox, D. A., Parker, G. F. and Sutton, V. J. R. Pile Driving into North Sea Boulder Clays. 2nd Annual Offshore Technology Conference Paper No. 1200, 1: 535-548, 1970.

33. Vijayvergia, V. N., Cheng, A. P. and Kolk, H. J. Effect of Soil Set Up on Pile Drivability in Chalk. Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers 103(GT10): 1069-1082, 1977.

34. Agarwal, S. L., Rawat, P. C. and Paintal, S. S. Problems Encountered in Installation of Offshore Piles for Fixed Platforms. 10th Annual Offshore Technology Conference Paper No. 3274, 3: 1925-1936, 1978.

35. Settgast, R. H. Marine Pile Load Testing in Carbonate Rocks. 12th Annual Offshore Technology Conference Paper No. 3868, 4: 131-141, 1980.

36. Tagaya, K., Heerema, E. P., Uchino, T. and Kusaka, T. Pile Driveability Test on Actual Offshore Platform in Calcareous Clay for Qatar NGL Offshore Project. 11th Annual Offshore Technology Conference Paper No. 3440, 2: 713-720, 1979.

26. Ling, S. C. Four Offshore-Piled Structures Installed on Outer Continental Shelf of the Atlantic Coast of the United States. 10th Annual Offshore Technology Conference Paper No. 3088, 1: 345-352, 1978.

27. Durning, P. J., Rennie, I. A., Thompson, J. M., and Ruckstuhl, E. J. Installing a Piled Foundation in Hard, Overconsolidated North Sea Clay for the Heather Platform. European Offshore Petroleum Conference Paper No. 46, 1: 375-382, 1978.

28. Young, A. G., Sullivan, R. A. and Rybicki, C. A. Pile Design and Installation Features of the Thistle Platform. European Offshore-Petroleum Conference Paper No. 12, 1: 101-110, 1978.

29. Duvivier, S., and Henstock, P. L. Installation of the Piled Foundations and Production Modules on Occidental's Piper A Platform. Proceedings of the Institution of Civil Engineers, Part 1, 66: 407-436, 1979.

30. Sutton, V. J., Rigden, W. J., James, E. L., St. John, H. D. and Poskitt, R. J. A Full Scale Instrumented Pile Test in the North Sea. 11th Annual Offshore Technology Conference Paper No. 3489, 2: 1117-1133, 1979.

31. Hirsch, T. J., Koehler, A. M. and Sutton, V. J. R. Selection of Pile Driving Equipment and Field Evaluation of Pile Bearing Capacity During Driving for the North Sea Forties Field. 7th Annual Offshore Technology Conference Paper No. 2247, 2: 37-49, 1975.

32. Fox, D. A., Parker, G. F. and Sutton, V. J. R. Pile Driving into North Sea Boulder Clays. 2nd Annual Offshore Technology Conference Paper No. 1200, 1: 535-548, 1970.

33. Vijayvergia, V. N., Cheng, A. P. and Kolk, H. J. Effect of Soil Set Up on Pile Drivability in Chalk. Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers 103(GT10): 1069-1082, 1977.

34. Agarwal, S. L., Rawat, P. C. and Paintal, S. S. Problems Encountered in Installation of Offshore Piles for Fixed Platforms. 10th Annual Offshore Technology Conference Paper No. 3274, 3: 1925-1936, 1978.

35. Settgast, R. H. Marine Pile Load Testing in Carbonate Rocks. 12th Annual Offshore Technology Conference Paper No. 3868, 4: 131-141, 1980.

36. Tagaya, K., Heerema, E. P., Uchino, T. and Kusaka, T. Pile Driveability Test on Actual Offshore Platform in Calcareous Clay for Qatar NGL Offshore Project. 11th Annual Offshore Technology Conference Paper No. 3440, 2: 713-720, 1979.

37. Rennie, I. A. and Fried, P. An Account of the Piling Problems Encountered and the Innovative Solutions Devised During the Installation of the Maui "A" Tower in New Zealand. 11th Annual Offshore Technology Conference Paper No. 3442, 2: 723-740, 1979.

38. Scanlan, R. H. and Tomko, J. J. Dynamic Prediction of Pile Static Bearing Capacity. Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers 95(SM2): 583-604, 1969.

39. Rausche, F., Moses, F., and Goble, G. G. Soil Resistance Predictions from Pile Dynamics. Journal of the Soil Mechanics and Foundations Division, Proceeding of the American Society of Civil Engineers 98(9): 917-937, 1972.

40. Ruiter, J. de and Berigen, F. L. Pile Foundations for Large North Sea Structures. Marine Geotechnology 3(3): 267-314, 1979.

41. Vines, W. R. Stress Measurements for Offshore Pile Driving. Symposium on Deep Foundations (ASCE), 475-494, 1980.

42. Rausche, F. Pile Driving Measurements on the Heather Platform Installation. European Offshore Technology Conference Paper No. 51, 1: 423-430, 1978.

43. Vines, W. R. and Verner, E. A. Monitoring Driving System Performance During Offshore Pile Installation. Soundings, a publication of McClelland Engineers, Houston, Texas, 1(3): 10-12, 1979.

44. Pile Monitoring Statistics. Internal Report No. 80-801, TERA, Inc., December 1980.

45. Sullivan, R. A. and Ehlers, C. J. Practical Planning for Driving Offshore Pipe Piles. 4th Annual Offshore Technology Conference, Paper No. 1600, 1: 807-822, 1972.

46. McClelland, B., Focht, J. A., Jr. and Emrich, W. J. Problems in Design and Installation of Offshore Piles. Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers 95(SM6): 1491-1514, 1969.

47. Gerwick, B. C. Jr. Techniques for Increasing the Capacities of Pin Piles in New and Existing Offshore Construction. 3rd Annual Offshore Technology Conference Paper No. 1473, 2: 543-552, 1971.

48. Esrig, M. I. Increasing Offshore Pile Driveability Through Electroosmosis. 10th Annual Offshore Technology Conference Paper No. 3269, 3: 1877-1885, 1978.

49. Ehlers, C. J. and Bowles, W. R. Underreamed Footings Support Offshore Platforms in the North Sea. 5th Annual Offshore Technology Conference Paper No. 1895, 2: 693-700, 1973.

CHAPTER 25

CAPTIONS

25-1 Example calculation of soil bearing pressure at various stages of pile installation.

25-2 Pile positioning sequence: A) add-on lifted from barge deck; B) crane rotates to position add-on; C) stabbing point maneuvered into pile top; D) add-on lowered into pile string.

25-3 Typical stabbing guides: A) internal stabbing guide; B) external stabbing guide.

25-4 Pile lifting methods: A) shackles through holes near pile top; B) padeyes; C) choking with slings; D) bridle; E) elevator.

25-5 Mechanical connectors: A) breechblock connector; B) detail of hydraulic connector.

25-6 Typical offshore hammer with leads.

25-7 Steam hammers types: A) moving piston; B) fixed piston.

25-8 Single-acting diesel hammer operation: A) tripping at start of stroke; B) fuel injection; C) compression and impact; D) diesel fuel explosion; E) top of stroke.

25-9 Hydraulic Hammer types: A) Hydroblock type; B) Menck type.

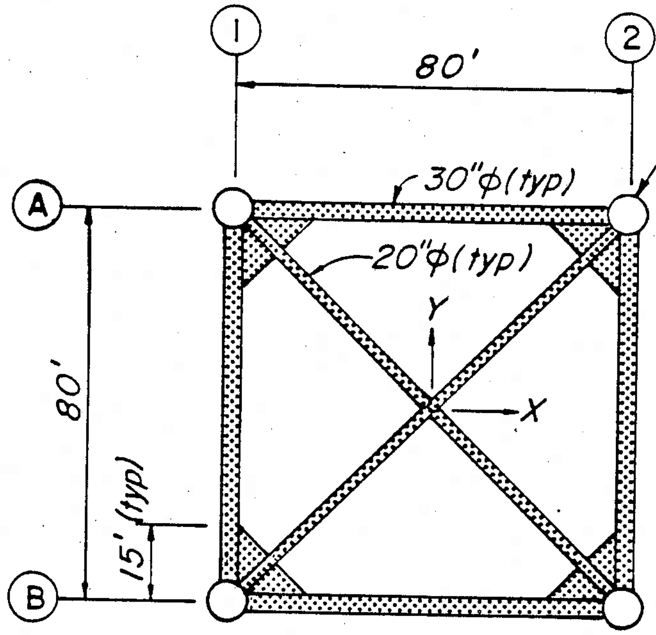
25-10 Drivability barameter study: A) 80,000 ft-lb hammer, clay profile; B) 80,000 ft-lb hammer, sand profile; C) 180,000 ft-lb hammer, clay profile; D) 180,000 ft-lb hammer, sand profile; E) 300,000 ft-lb hammer, clay profile; F) 300,00 ft-lb hammer, sand profile; G) 500,000 ft-lb hammer, clay profile; H) 500,000 ft-lb hammer, sand profile; I) 1,500,000 ft-lb hammer, clay profile; J) 1,500,000 ft-lb hammer, sand profile; K) assumptions.

25-11 Hammer placement sequence: A) hammer lifted from barge deck; B) hammer positioned overpile by booming out or in; C) pilecap seated by rocking hammer; D) leads lowered after hammer in place.

25-12 Typical blowcount record showing recorded blowcount at 1-ft increments of penetration, length of delays, soil stratigraphy, pile add-on schedule, and hammers used.

25-13 Histogram and statistics of hammer efficiency for all hammers in data base.

25-14 Histogram of cushion stiffness for bongassi hardwood cushions used with Menck 3000 hammers.



PLAN VIEW @ MUDLINE

GIVEN

- C.G. @ center of jacket
- Weight of jacket (on bottom) = 400^k
- Weight of pile sections (in place):
  - P1 100^k @ $e_x = e_y = 30'$
  - P2 120^k @ $e_x = e_y = 15'$
- Piles reach self support during lowering of P2 sections.
- No diaphragms in jacket leg.
- Bottom bay framing designed to resist soil loads.
- Ignore 20"φ due to flexibility.

FOR FRAMING + MUDMATS,

$$A = 4(190) + 4(78) = 1072 Ft^2$$

$$I_x = I_y = 608{,}200 + 182{,}900 + 747{,}600 + 16{,}300 = 1{,}555{,}000 Ft^4$$

$$S_x = S_y = \frac{1{,}555{,}000}{40} = 38{,}875 Ft^3$$

	Step	ΣP	$\Sigma M_x = \Sigma M_y$	P/A	$\frac{(M_x + M_y)c}{I}$	Bearing Pressure, ksf Max.	Min.
1.	JACKET ONLY	400	0	0.37	0	0.37	0.37
2.	ADD P1/A1	500	3000	0.47	0.15	0.62	0.32
3.	ADD P1/B2	600	0	0.56	0	0.56	0.56
4.	ADD P2/A1	720	1800	0.67	0.09	0.76	0.58
5.	ADD P2/B2	840	0	0.78	0	0.78	0.78
6.	LOWER B2	620	4800	0.58	0.25	0.83	0.33
7.	LOWER A1	400	0	0.37	0	0.37	0.37

Fig. 25 - 1

SOIL BEARING PRESSURE EXAMPLE

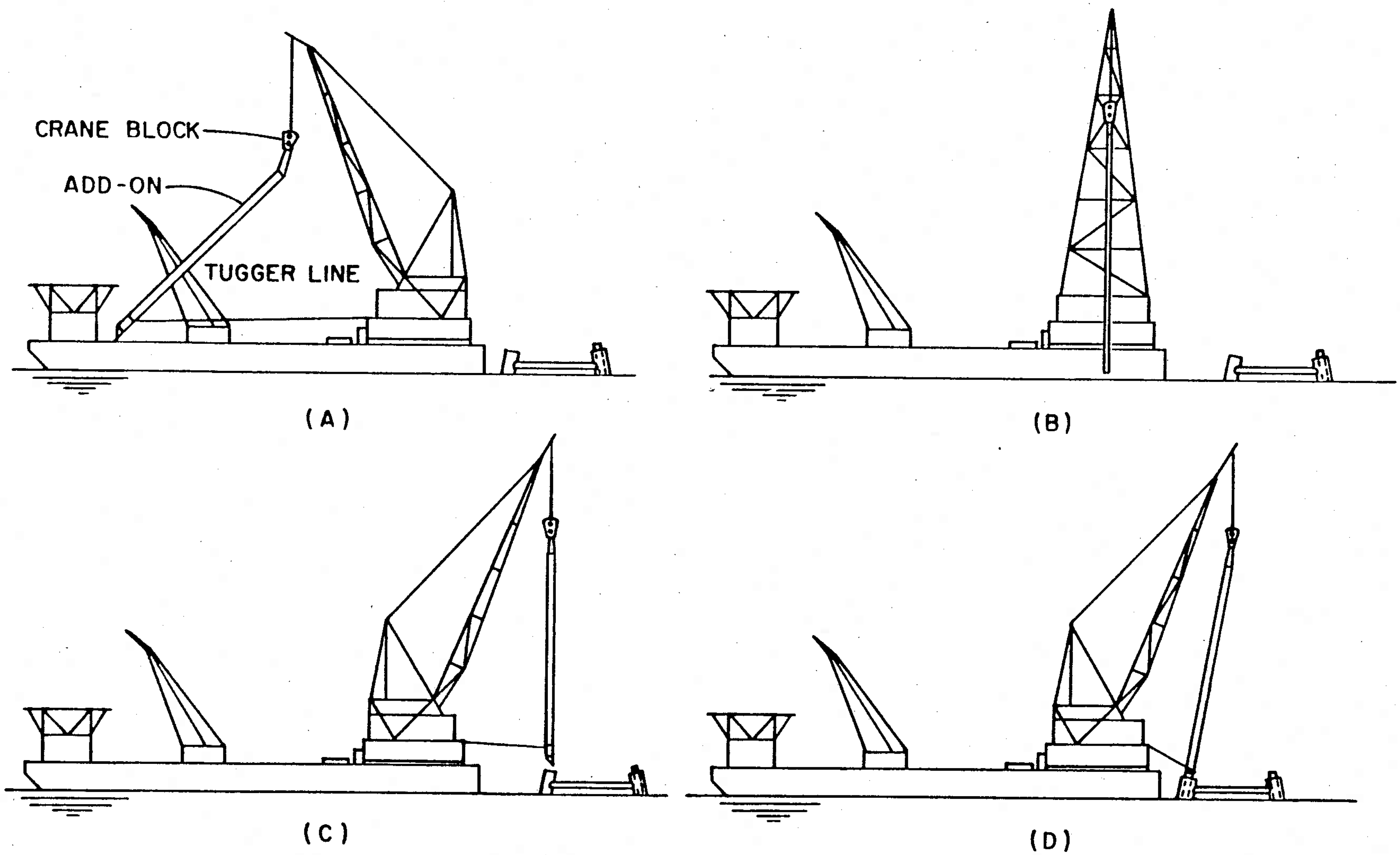


Fig. 25-2

PILE POSITIONING

TERA, INC.

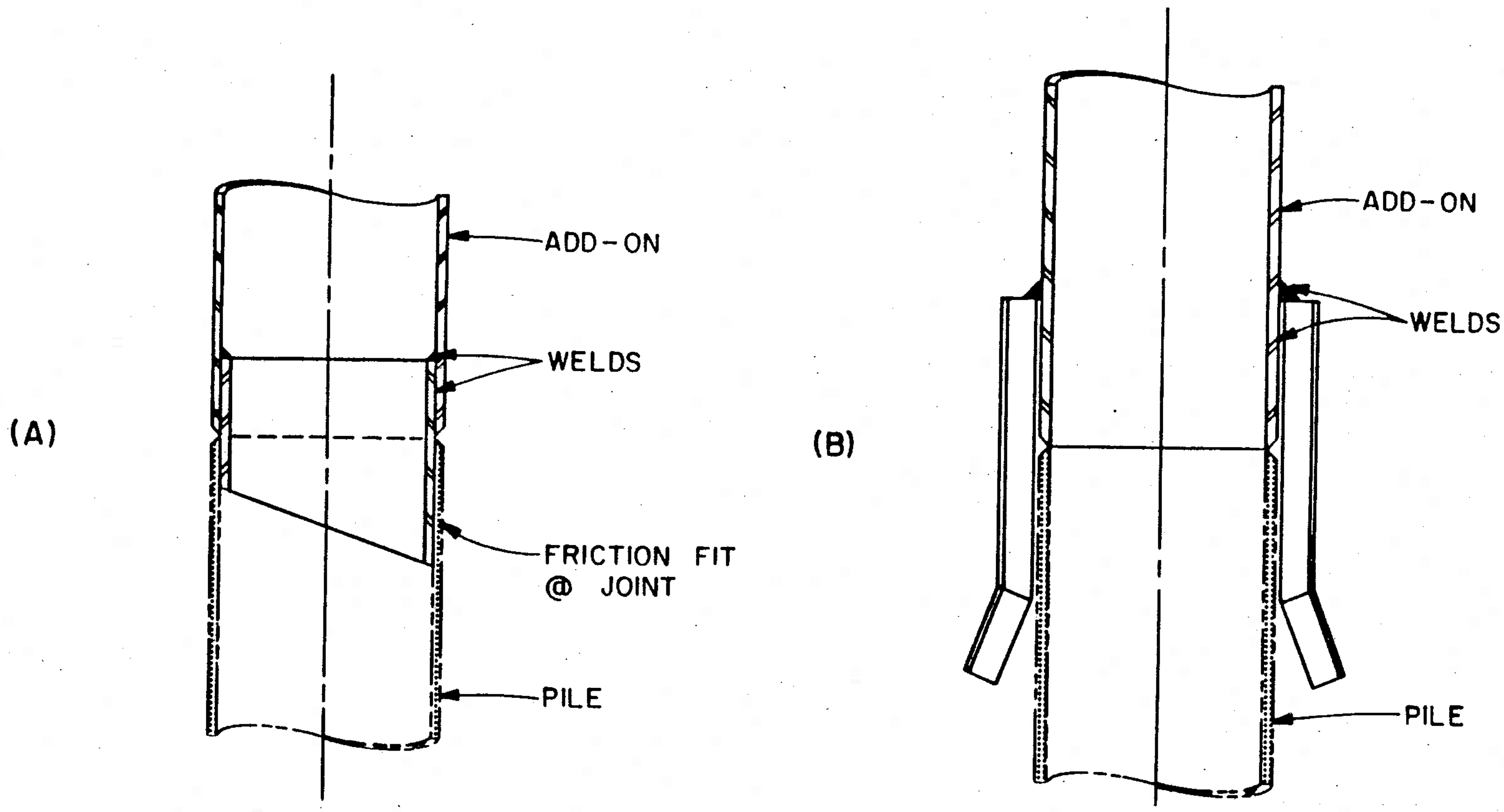


Fig. 25-3

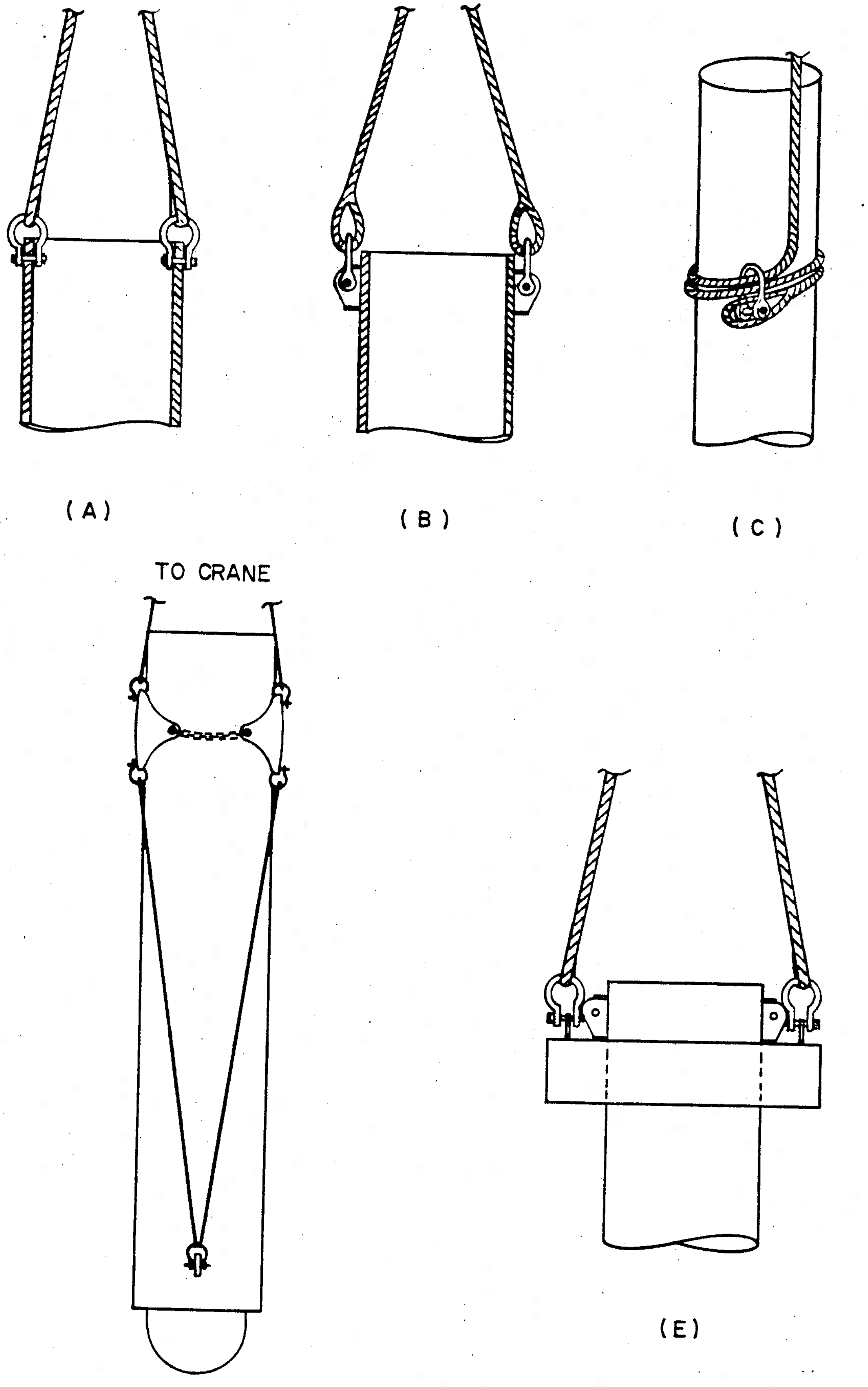
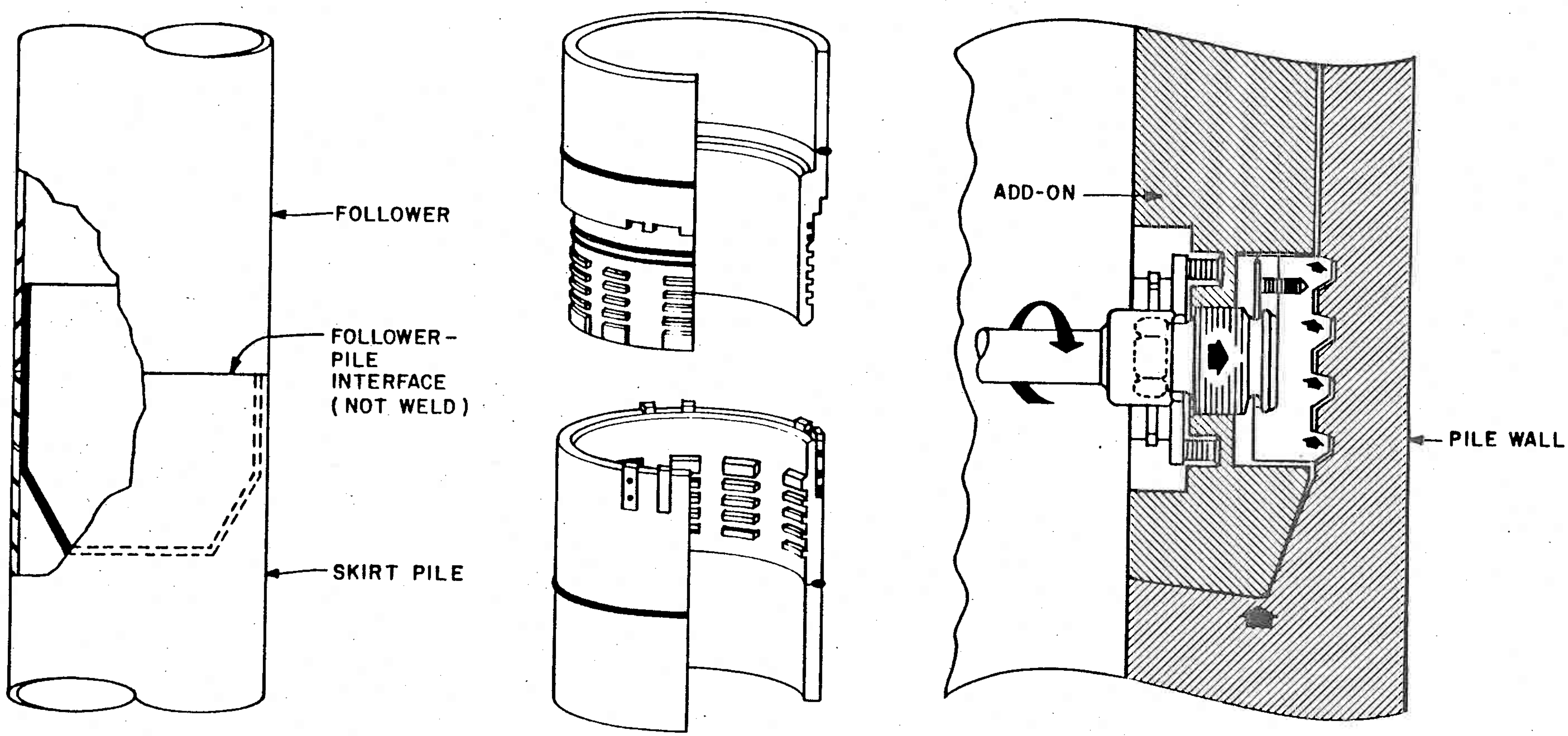


Fig. 25 - 4

LIFTING METHODS


FOLLOWER
FOLLOWER-
PILE
INTERFACE
(NOT WELD)
SKIRT PILE
ADD-ON
PILE WALL

(A) (B) (C)

Fig. 25-5

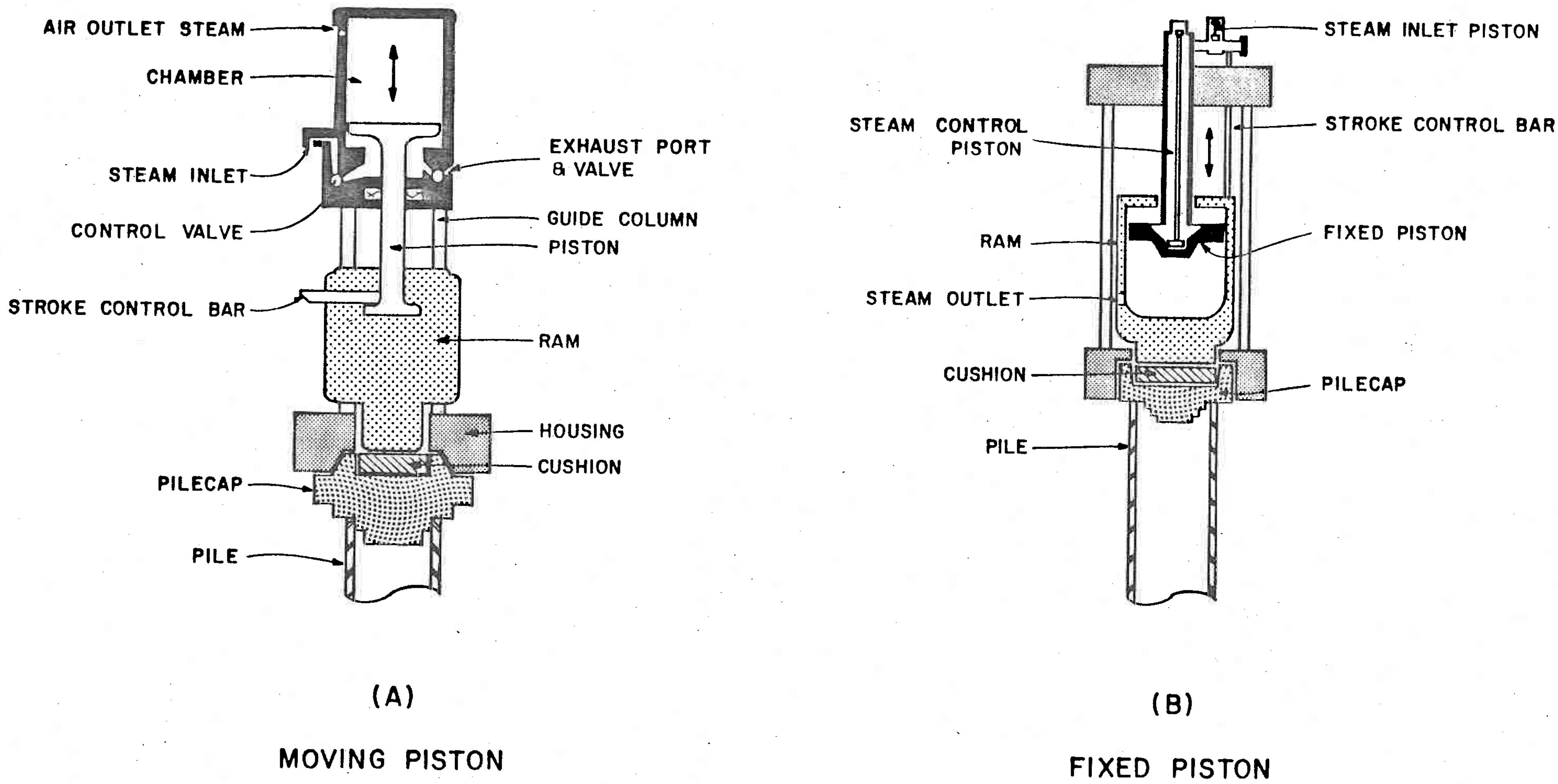


Fig. 25-7

STEAM HAMMER

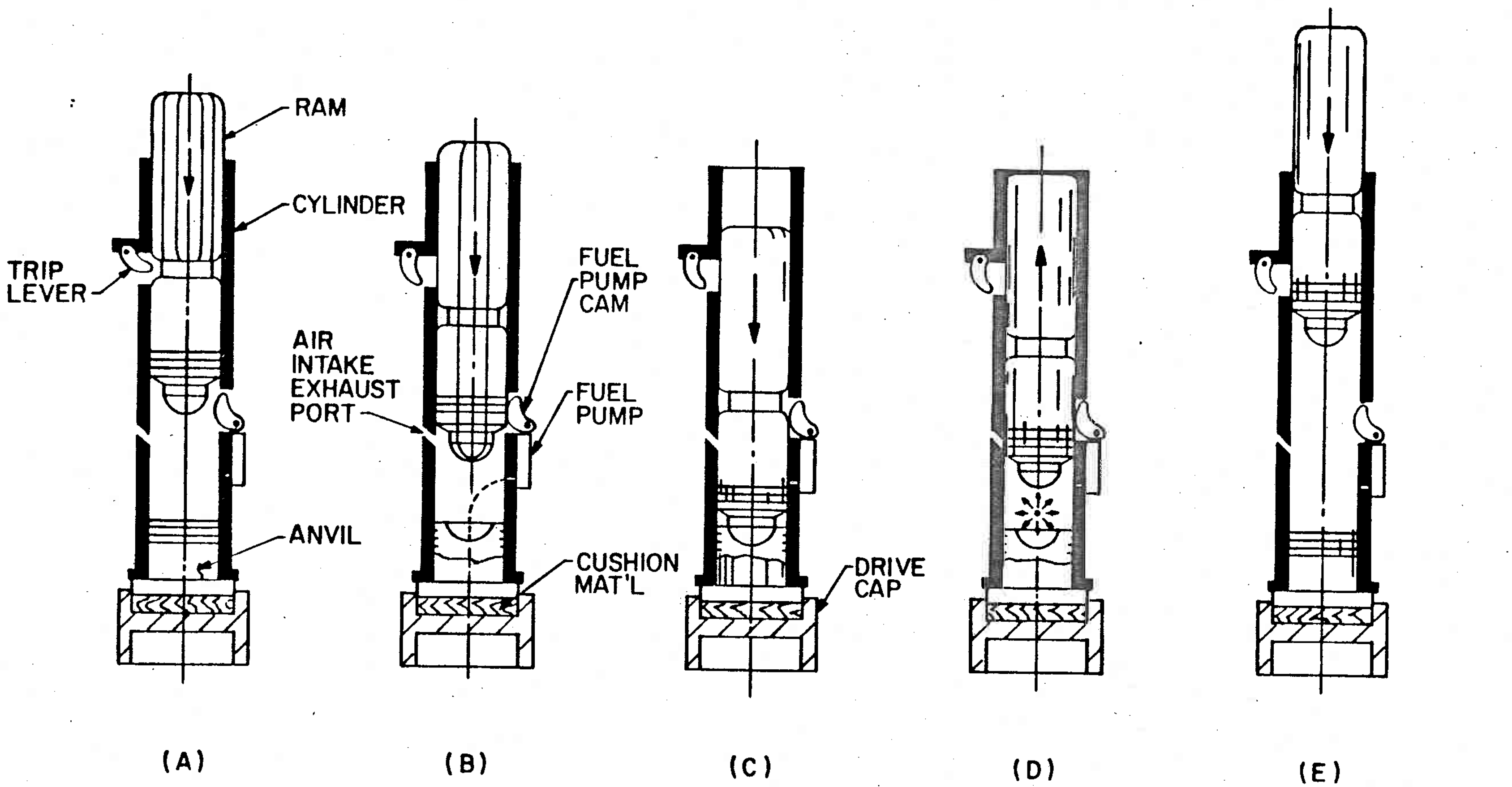


Fig. 25 - 8

SINGLE-ACTING DIESEL HAMMER

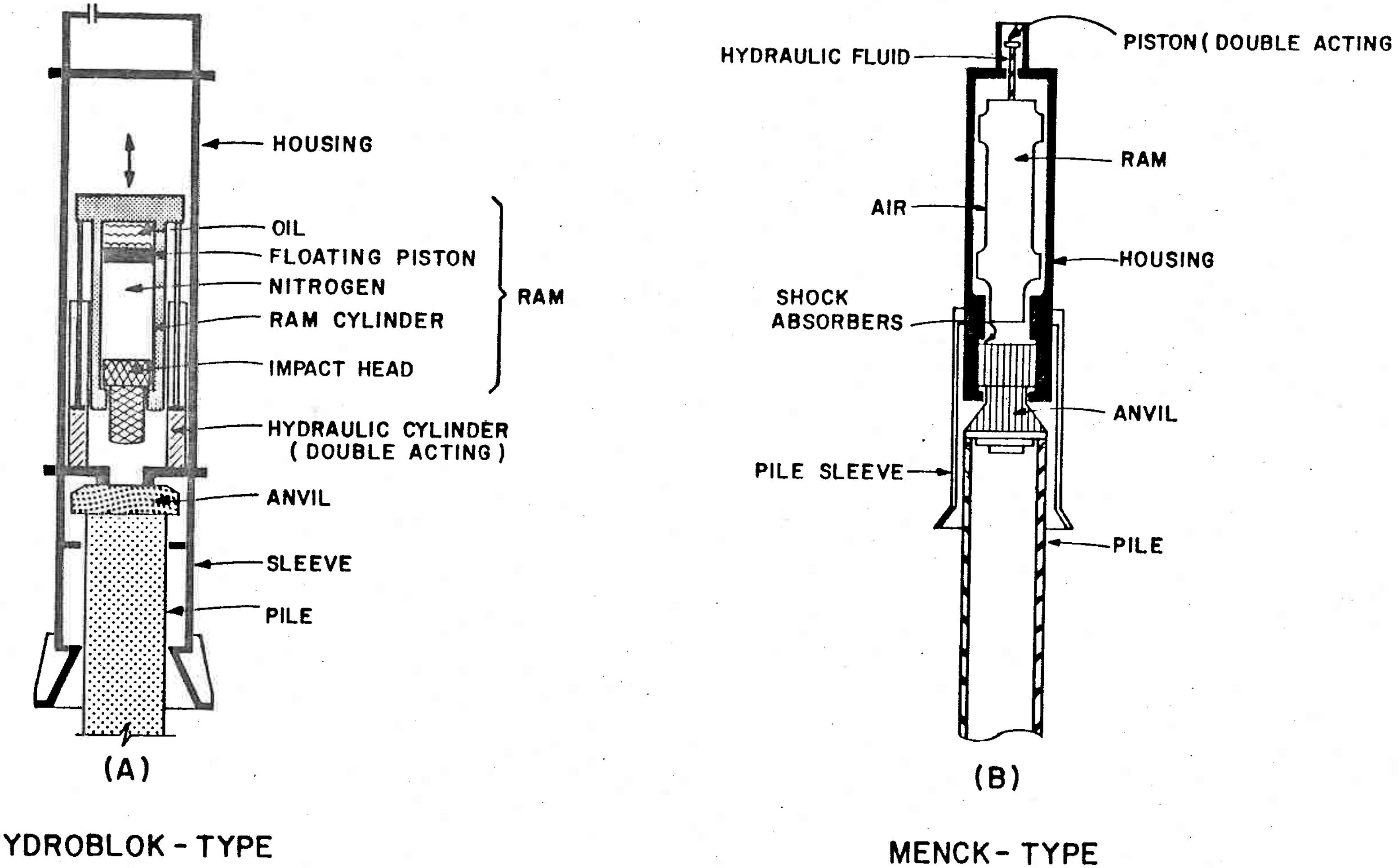


Fig. 25 - 9

HYDRAULIC HAMMER

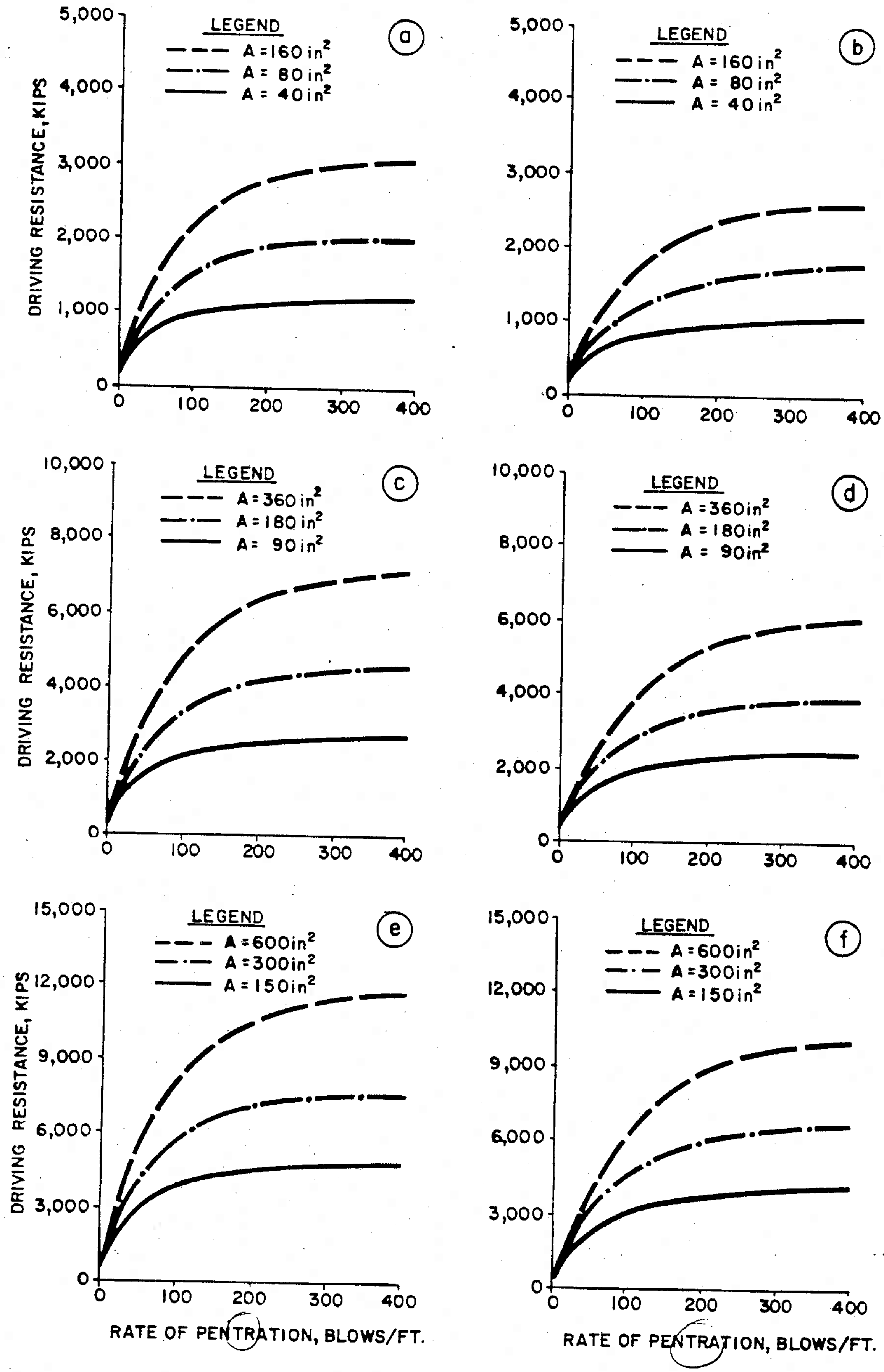
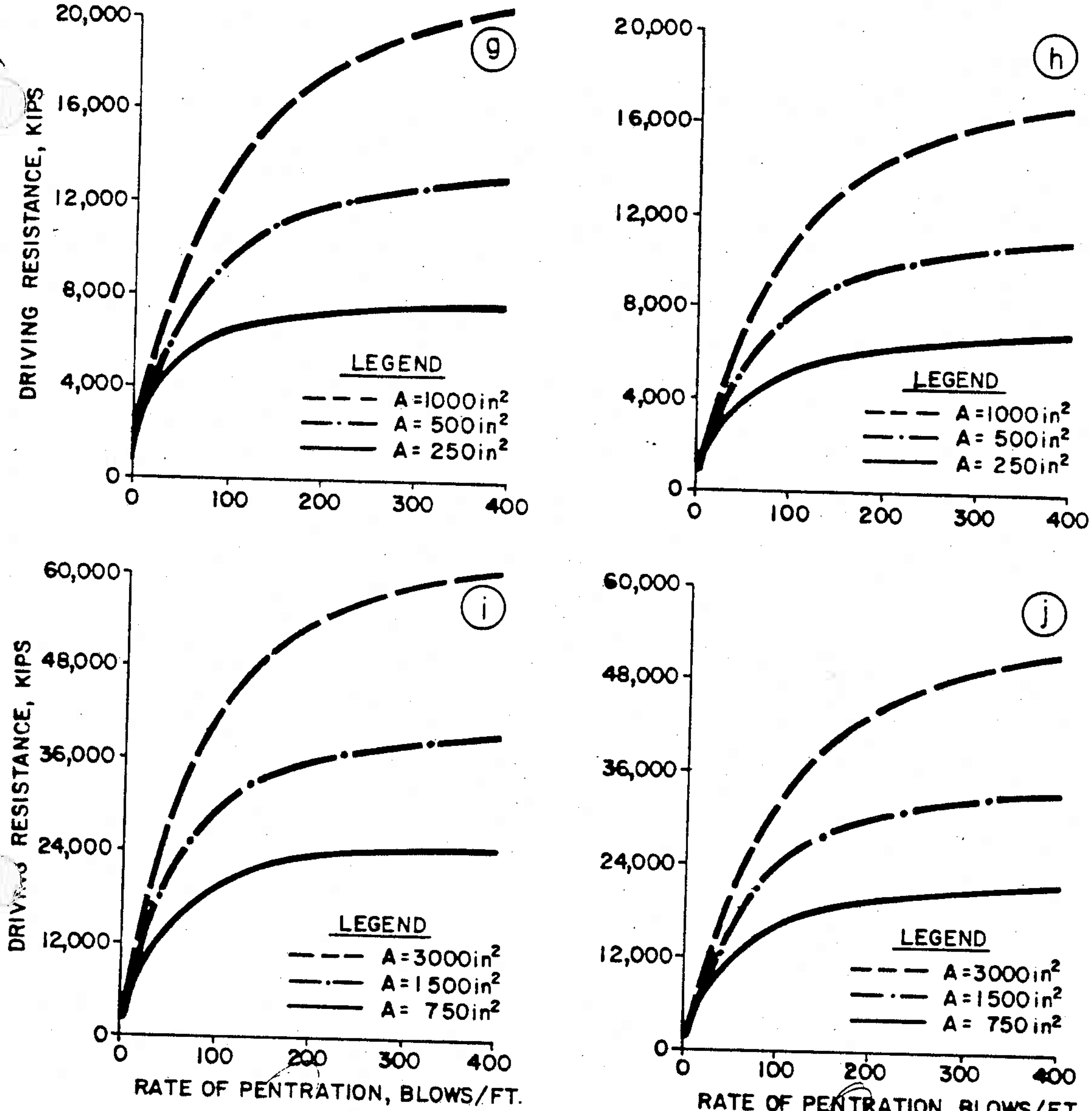

LEGEND
A = 160 in²
A = 80 in²
A = 40 in²
(a)
(b)
LEGEND
A = 360 in²
A = 180 in²
A = 90 in²
(c)
(d)
LEGEND
A = 600 in²
A = 300 in²
A = 150 in²
(e)
(f)
DRIVING RESISTANCE, KIPS
RATE OF PENTRATION, BLOWS/FT.

Fig. 25-10



DRIVING SYSTEM

(k)

RATED ENERGY (FT./LB.)	*STROKE (FT.)*	*CUSHION STIFFNESS (K/IN)*	*C.R.*	*PILECAP WEIGHT (K)*	*EFFICIENCY*
80,000	4	12,500	0.7	10	60%
180,000	4	30,000	0.7	22	60%
300,000	5	50,000	0.7	36	60%
500,000	5	90,000	0.7	60	60%
1500,000	6	250,000	0.7	120	60%

PILE

3 pile area per hammer, As noted.
400' pile length w/300' penetration.

SOIL

2 Set of soil parameters for each pile-hammer combination:*
Clay - $Q_s = Q_p = 0.1''$, $J_s = 0.05$, $J_p = 0.15$, % @ Tip = 10%
Sand - $Q_s = Q_p = 0.1''$, $J_s = 0.08$, $J_p = 0.15$, % @ Tip = 50%
* *Damping and quake parameters from Roussel (1979)*[51]

Fig. 25-10 (CONT'D)

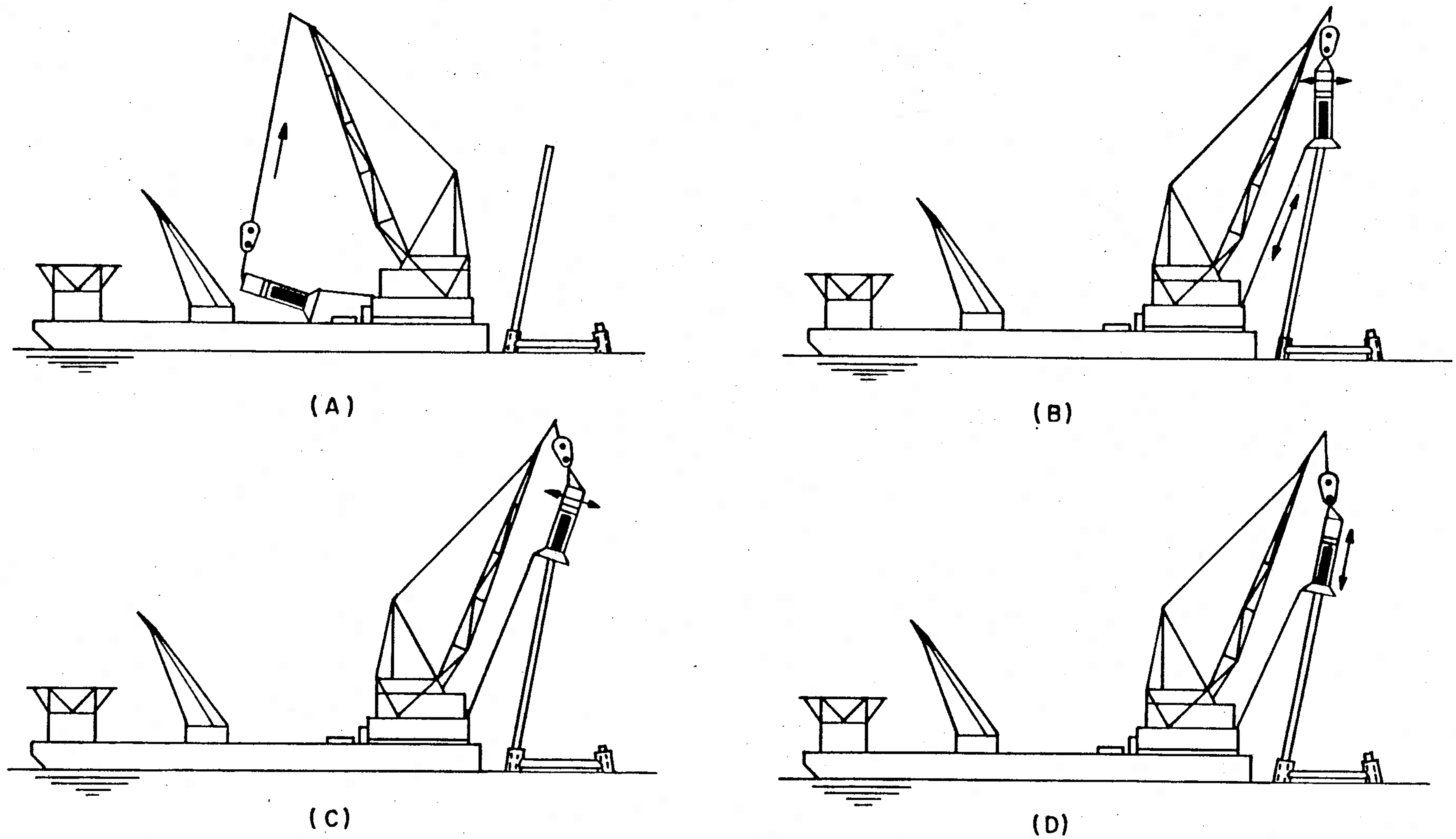


Fig. 25-11
HAMMER PLACEMENT

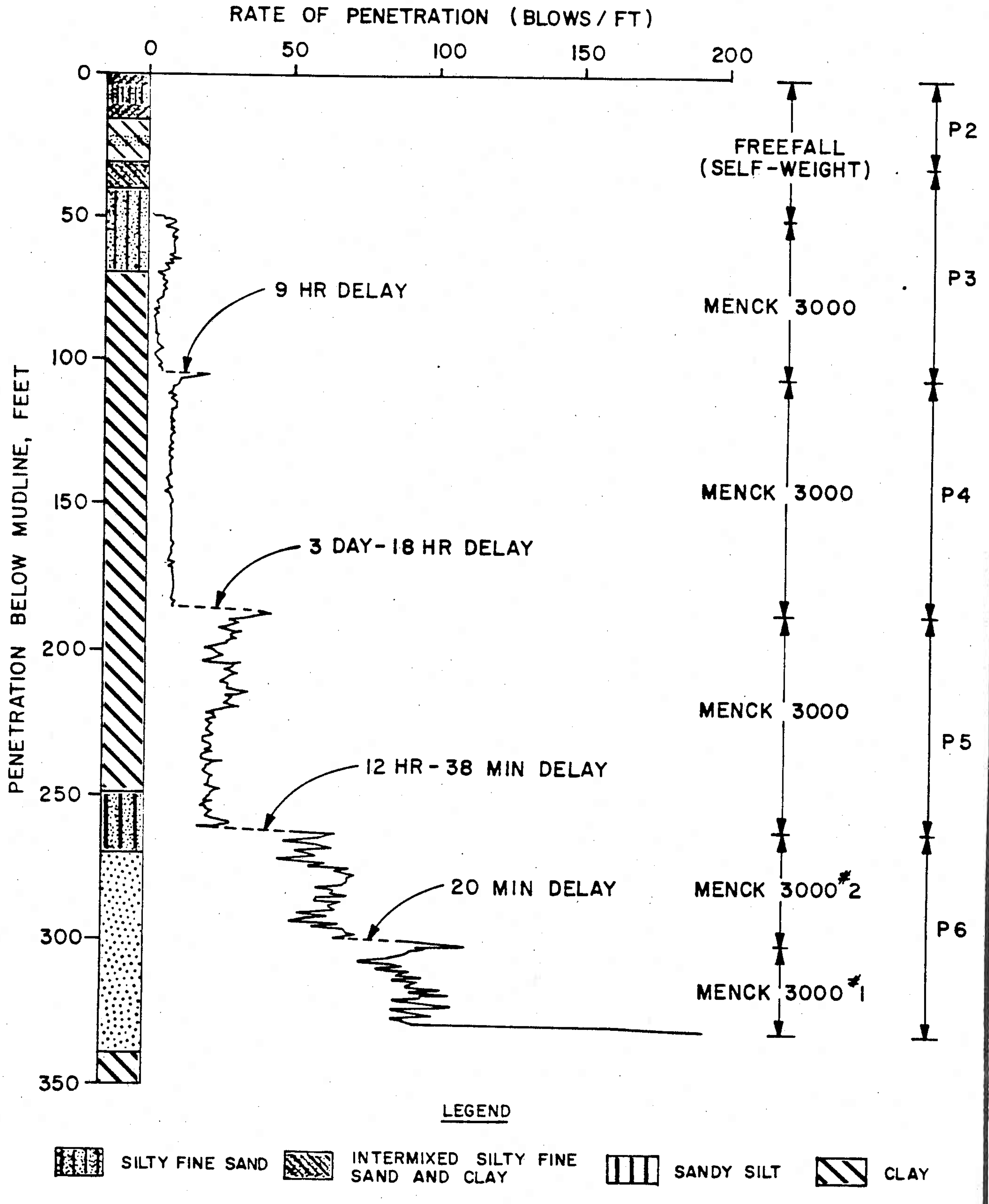

RATE OF PENETRATION (BLOWS / FT)
0
50
100
150
200
PENETRATION BELOW MUDLINE, FEET
0
50
100
150
200
250
300
350
9 HR DELAY
3 DAY-18 HR DELAY
12 HR-38 MIN DELAY
20 MIN DELAY
FREEFALL (SELF-WEIGHT)
MENCK 3000
MENCK 3000
MENCK 3000
MENCK 3000#2
MENCK 3000#1
P2
P3
P4
P5
P6
LEGEND
SILTY FINE SAND
INTERMIXED SILTY FINE SAND AND CLAY
SANDY SILT
CLAY

Fig. 25-12

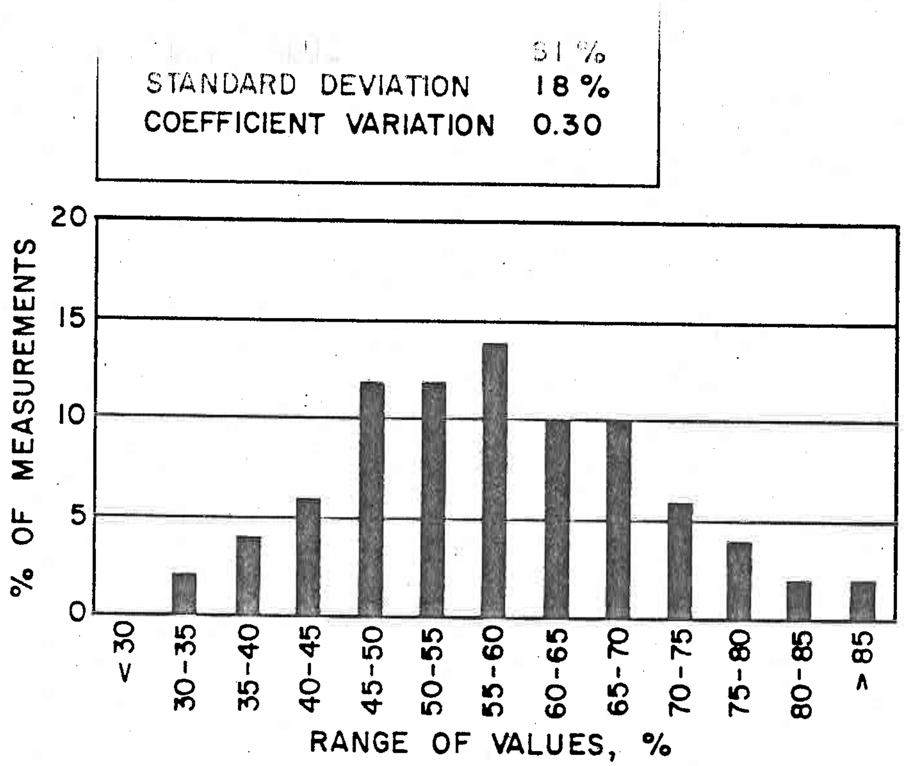


Fig. 25-13

HISTOGRAM OF HAMMER EFFICIENCY

ALL MEASUREMENTS

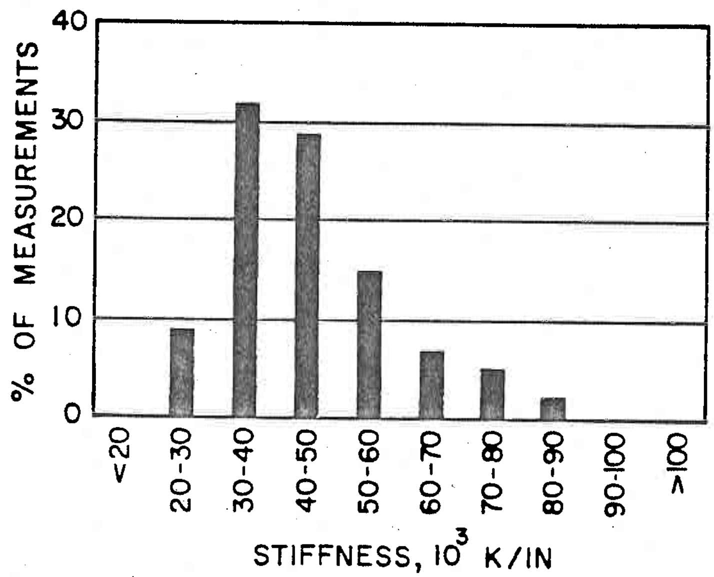


Fig. 25 - 14

HISTOGRAM OF CUSHION STIFFNESS

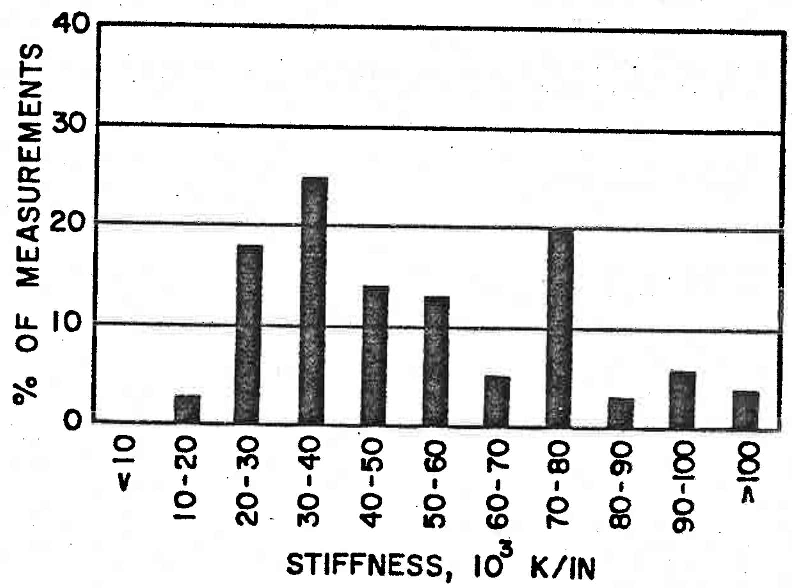


Fig. 25 - 15

HISTOGRAM OF CUSHION STIFFNESS

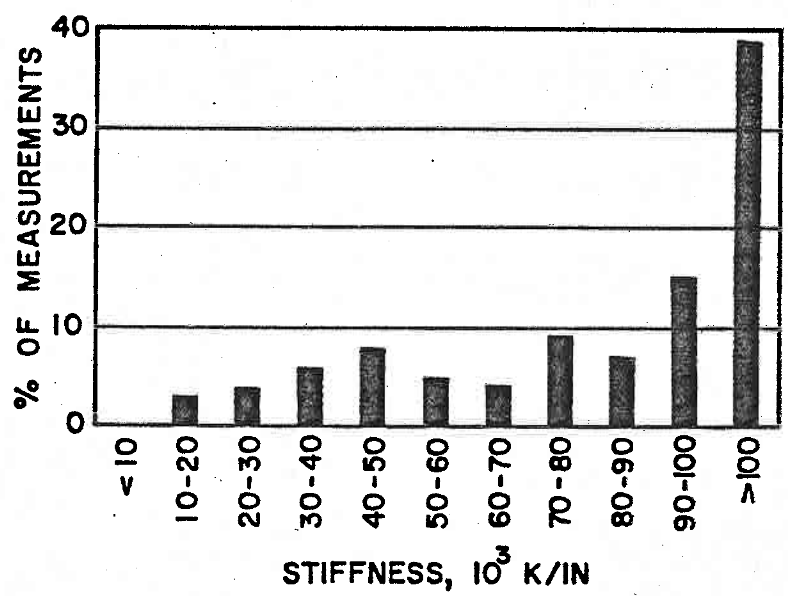
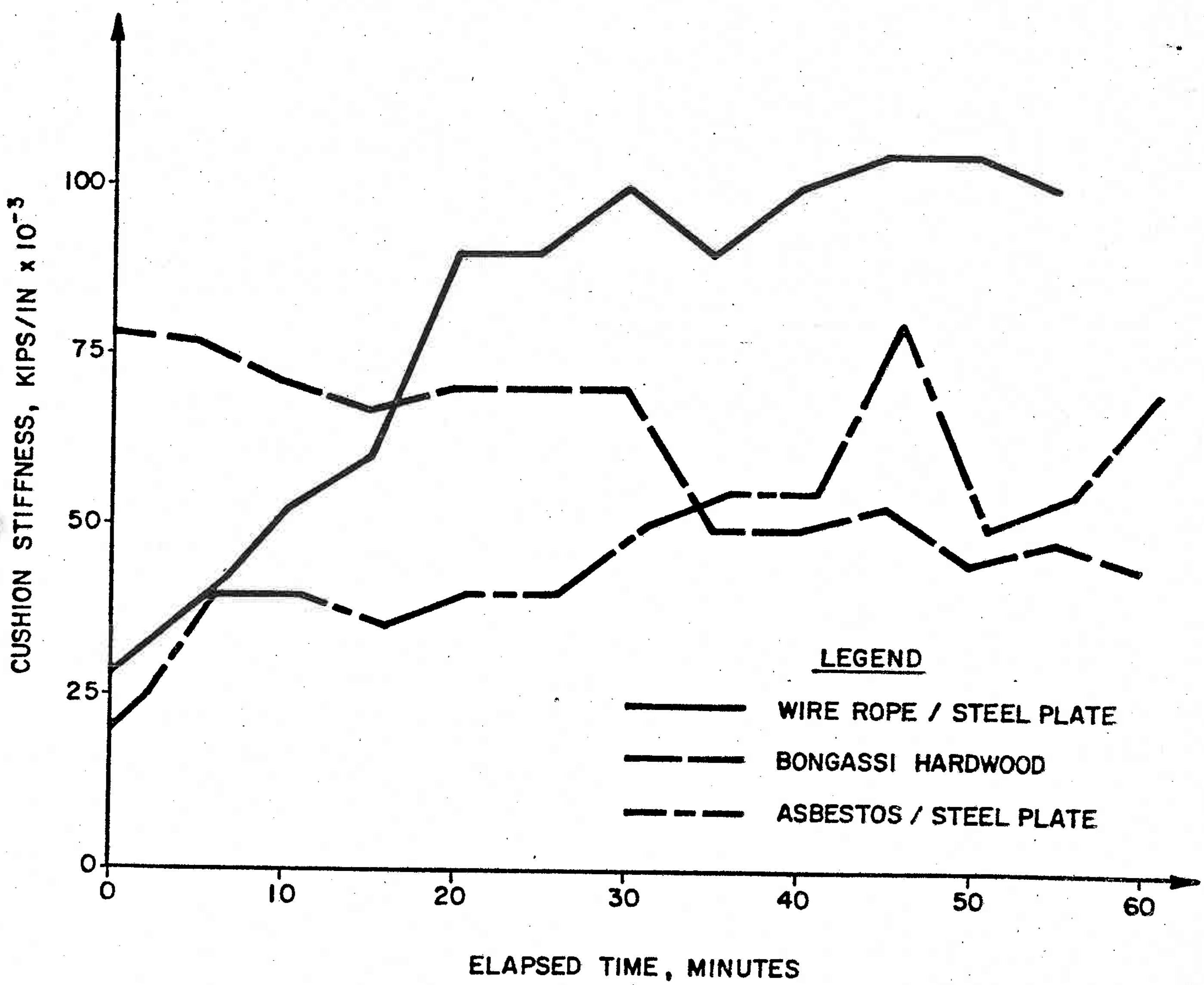


Fig. 25 - 16

HISTOGRAM OF CUSHION STIFFNESS



CUSHION STIFFNESS, KIPS/IN x 10^-3
100
75
50
25
0
0
10
20
30
40
50
60
ELAPSED TIME, MINUTES
LEGEND
WIRE ROPE / STEEL PLATE
BONGASSI HARDWOOD
ASBESTOS / STEEL PLATE

Fig. 25 - 17

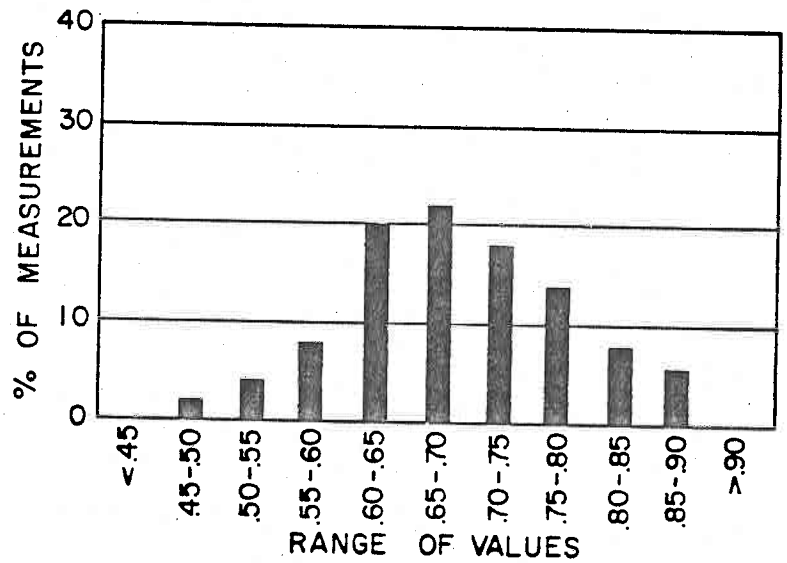


Fig. 25 - 18

HISTOGRAM OF COEFFICIENT OF RESTITUTION

ALL MEASUREMENTS

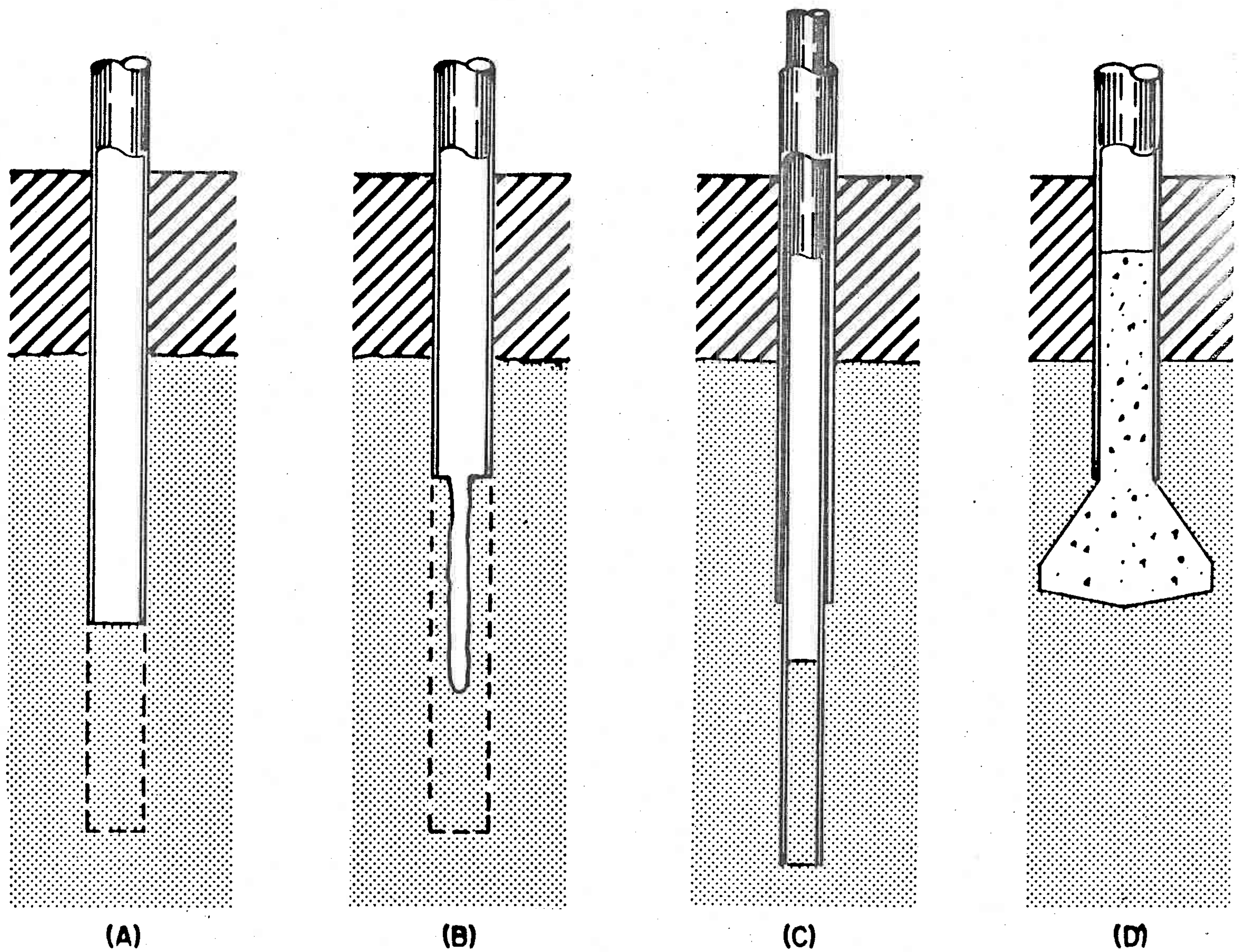


Fig. 25-19 SUPPLEMENTAL INSTALLATION PROCEDURES