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UNIFIED FACILITIES CRITERIA (UFC)

PILE DRIVING EQUIPMENT

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UNIFIED FACILITIES CRITERIA (UFC)

PILE DRIVING EQUIPMENT

U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by **1**\ ... /**1**/)

Change No.	Date	Location

This UFC supersedes TI 818-03, dated 3 August 1998. The format of this UFC does not conform to UFC 1-300-01; however, the format will be adjusted to conform at the next revision. The body of this UFC is the previous TI 818-03, dated 3 August 1998.

FOREWORD

\1\
The Unified Facilities Criteria (UFC) system is prescribed by MIL-STD 3007 and provides planning, design, construction, sustainment, restoration, and modernization criteria, and applies to the Military Departments, the Defense Agencies, and the DoD Field Activities in accordance with USD(AT&L) Memorandum dated 29 May 2002. UFC will be used for all DoD projects and work for other customers where appropriate. All construction outside of the United States is also governed by Status of forces Agreements (SOFA), Host Nation Funded Construction Agreements (HNFA), and in some instances, Bilateral Infrastructure Agreements (BIA.) Therefore, the acquisition team must ensure compliance with the more stringent of the UFC, the SOFA, the HNFA, and the BIA, as applicable.

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TI 818-03
3 August 1998

US Army Corps
of Engineers®

Technical Instructions

Pile Driving Equipment

Headquarters
U.S. Army Corps of Engineers
Engineering Division
Directorate of Military Programs
Washington, DC 20314-1000

CEMP-E

TI 818-03
3 August 1998

TECHNICAL INSTRUCTIONS

Pile Driving Equipment

Approved for public release; distribution is unlimited.

Record of Changes (changes indicated by \1\..../1/)

No. Date Location

This Technical Instruction supersedes EI 02G001, dated 1 July 1997.
(EI 02G001 text is included in this Technical Instruction and may carry EI 02G001 identification.)

FOREWORD

These technical instructions (TI) provide design and construction criteria and apply to all U.S. Army Corps of Engineers (USACE) commands having military construction responsibilities. TI will be used for all Army projects and for projects executed for other military services or work for other customers where appropriate.

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FOR THE DIRECTOR OF MILITARY PROGRAMS:

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EI 02G001

CEMP-E

Engineering Instructions
No. 02G001

01 July 1997

PILE DRIVING EQUIPMENT

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CHAPTER 1

INTRODUCTION

1-1. PURPOSE. This document presents guidelines to assist the preparation of specifications for pile installation and for assessment of construction operations.

1-2. SCOPE. Descriptions of types of piles, advantages, disadvantages, and usage of piles, equipment, and installation methods are discussed in these instructions.

a. Equipment. Proper equipment and installation methods are critical to prevent damage to the pile foundation during driving, to obtain adequate bearing capacity, and to minimize the cost of installation. Guidance is provided for monitoring the installation of piles including equipment operation, prevention of pile damage during installation, construction problems, and effects of driving on adjacent structures.

b. General Guidance. Guidance is provided on selection of equipment, verification of design, construction considerations, and the care and maintenance of piles.

c. Installation Methods. Special installation methods are sometimes required depending on the soil and the environment. Guidance is provided for pile installation assisted by jetting or where hammers or vibrators are not or cannot be used.

d. Case History. A case history study is included as an example of how to proceed with installation of a driven pile foundation. Guidance for design of driven piles is provided in TM 5-809-7.

e. Construction Guidance. Guidance for construction of drilled shafts is available in FHWA-HI-88-042 and Association of Drilled Shaft Contractors Publication.

1-3. REFERENCES. Appendix A contains a list of references used in this document.

1-4. HEARING CAUTION. Impact sound can exceed 140 decibels during pile driving operations. Unprotected personnel exposed to these high sound levels can incur permanent hearing loss that becomes worse over extended periods of time. A qualified industrial hygienist should be consulted to prescribe the appropriate hearing protection necessary to preserve hearing.

1-5. DESCRIPTION OF PILES. The type of pile influences the method selected for installation. For example, impact hammers may not be able to drive timber or closed-end pipe piles into firm ground without damage to the pile, and assisted installation may be required.

a. Types of Piles. Piles are classified according to the amount of soil displacement that will occur during installation. The soil is disturbed by driving causing cohesive clays to remold and cohesionless sands to change density. The displaced soil can cause the ground surface around the pile to heave. Pile driving can also cause the area around the pile to settle due to densification of a sand foundation material when driving nondisplacement piles. Specifications for the piles given in tables 1-1 through 1-6 assume that wave equation analysis was used for construction quality control.

(1) Displacement. These piles have a relatively large cross-sectional area. Driving these piles displaces the soil a relatively large amount and can cause significant ground heave.

(a) Timber (TIM). TIM piles are the oldest deep foundation known. They are typically used as round untrimmed logs cut to a suitable length, usually 20 to 66 feet with diameters from 6 to 16 inches as shown in table 1-1. They may be sawed into square sections, but durability may be reduced because the outer sapwood that best absorbs preservative is removed. TIM piles embedded below the ground-water level can last for many years without treatment. TIM piles are typically installed with the aid of jetting in a sand foundation and predrilling in a clay foundation. A creosote pressure treatment should be applied to piles that extend above the groundwater level or installed in a marine environment to protect against borers and decay.

Table 1-1. Timber Pile Specifications.

LENGTH	9 - 18 M (30 - 60 FT)
MATERIAL SPECIFICATION	ASTM D 25
DESIGN STRESSES	DOUGLAS FIR - 8.27 MPA (1.2 KSI) RED OAK - 7.58 MPA (1.1 KSI) SOUTHERN PINE - 8.27 MPA (1.2 KSI) EASTERN HEMLOCK - 5.52 MPA (0.8 KSI)
DRIVING STRESSES	3 TIMES THE ABOVE DESIGN STRESSES
DESIGN LOADS	100 - 500 kN (10 - 50 TONS)
DISADVANTAGES	DIFFICULT TO SPLICE VULNERABLE TO DAMAGE IN HARD DRIVING TIP MAY HAVE TO BE PROTECTED VULNERABLE TO DECAY UNLESS TREATED AND PILES ARE SUBMERGED OR DRY SUBJECT TO MECHANICAL WEAR
ADVANTAGES	LOW INITIAL COST PERMANENTLY SUBMERGED PILES ARE RESISTANT TO DECAY EASY TO HANDLE READILY CUT TO REQUIRED LENGTH
APPLICATION	FRICTION PILES IN SAND, SILT, OR CLAY NOT RECOMMENDED IN DENSE GRAVEL OR TILL
ILLUSTRATION	

(b) Precast Reinforced Concrete (PCRC). PCRC piles are made from concrete and a steel reinforcement cage consisting of several longitudinal bars and lateral tie steel as shown in table 1-2. The tie steel may be either hoops or a spiral. PCRC piles are usually replaced with precast prestressed piles.

(c) Precast Prestressed Concrete (PCPS). These piles are similar to reinforced precast concrete piles, except that the longitudinal reinforcement steel is replaced by prestressing steel enclosed in a conventional steel spiral (table 1-2). PCPS piles can usually be made longer than PCRC piles with the same rigidity because of the prestressing.

(d) Concrete Post-tensioned Cylinder (CPTC). These piles are centrifugally cast into 16-foot-long circular hollow sections (figure 1-1) using a low-porosity, high-density concrete with zero slump. Several sections are bonded with a plastic joint compound to the required length, then Post-tensioned with prestressing cables that run through holes cast in the pile wall.

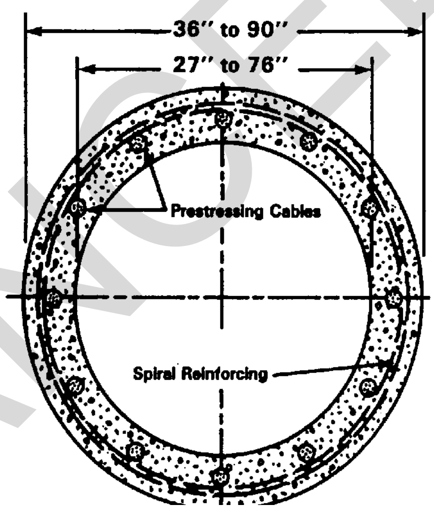

Figure 1-1. Cross Section of a Concrete Post-tensioned Cylinder Pile.

Table 1-2. Precast Concrete Pile Specifications.

LENGTH	12 - 15 M (40 - 50 FT) REINFORCED 18 - 30 M (60 - 100 FT) PRESTRESSED
MATERIAL SPECIFICATION	ACI 318 FOR CONCRETE ASTM A-615 FOR REINFORCING STEEL
DESIGN STRESSES	REINFORCED 0.33 f'c (COMPRESSION); USE GROSS CROSS SECTION TO DETERMINE ALLOWABLE LOADS PRESTRESSED 0.33 f'c (COMPRESSION), MINIMUM EFFECTIVE PRESTRESS 4.8 MPA (0.7 KS) MINIMUM f'c 34.5 MPA (5.0 KSI)
DRIVING STRESSES	REINFORCED COMPRESSION - 0.85 f'c TENSION - 3 $(f'c)^{1/2}$ PRESTRESSED COMPRESSION - 0.85 f'c - EFFECTIVE PRESTRESS TENSION - 3 $(f'c)^{1/2}$ + EFFECTIVE PRESTRESS
DESIGN LOADS	SPECIFICALLY DESIGNED FOR A WIDE RANGE OF LOADS
DISADVANTAGES	HIGH INITIAL COST VULNERABLE TO DAMAGE FROM HANDLING UNLESS PRESTRESSED PRESTRESSED DIFFICULT TO SPLICE
ADVANTAGES	HIGH LOAD CAPACITIES CORROSION RESISTANCE POSSIBLE HARD DRIVING READILY CUT TO REQUIRED LENGTH
APPLICATION	LOADS UP TO 4 MN (400 TONS) FRICTION AND/OR END BEARING PILES HIGH BENDING RESISTANCE
ILLUSTRATION	

(e) Closed-end Steel Pipe (CESP). These are relatively light cylinder piles, yet capable of hard driving and carrying heavy loads to a deep-bearing stratum (table 1-3). They can be readily cut or spliced to any length. CESP piles can be made closed-end by either a flat or conical tip. Conical tips can reduce uneven stresses at the pile tip as a result of driving and reduce the potential for damage to the pile. Pipe piles should be filled with concrete after driving to increase the section modulus and rigidity.

(f) Concrete Cast in Shell (CCS). Steel shells are placed on a mandrel, which can be a stepped mandrel for stepped shells with the shell of the mandrel driven to the required penetration depth (table 1-4). The mandrel is withdrawn and the shells filled with concrete. The shell, a monotube, may also be driven without a mandrel (table 1-5). The walls of monotube piles are fluted and the pile tip is made of a welded steel point. The walls can be reinforced and dowels welded to the top of the pile for connection with the pile cap. Shell nominal diameters vary from 9 to 18 inches with lengths from 4 to 16 feet. Shell gauges vary from 10 to 18 with the heavier gauges placed at the lower portions of the pile.

(2) Small Displacement. These piles displace the soil a little during driving and lead to minimal or no ground heave. Penetration resistance is usually less during driving, thus allowing these piles to be driven through obstructions and bedrock more easily than large displacement piles.

(a) Round Open-end Pipe (ROEP). These piles are open-end steel cylinders similar to those in table 1-3, but without the flat or conical tip. Soil within the circular pipe can be easily taken out and the open tube filled with concrete.

(b) Steel H-section (HP). These piles have similar to flanged or wedged steel beams and have the ability to be driven through hard and resisting materials including some rock formations as shown in table 1-6.

(c) Helical Steel. These piles consist of square extension sections of galvanized steel coupled to helical sections, table 1-7. Helical screws are spaced from 3 to 10 feet apart. They can be installed in locations where space is limited using portable rotary equipment. Several sections are bolted together to provide the correct length. Helical screws are useful for resisting both tension and compression loads. Construction of the structure can begin immediately after the screw pile is installed because concrete is often not necessary; however, concrete can be placed at the top of the pile to increase rigidity and bending resistance. Helical screw piles are useful for supporting lightly loaded structures.

(3) Composite. Composite piles consist of sections of different materials joined together to use the advantages of each of the materials. These piles typically consist of concrete and steel pipe or concrete and steel HP sections (table 1-8). Concrete and timber combinations are seldom used because good joints are difficult to construct.

(4) Sheet Piles. Sheet pile cellar structures have numerous applications, but are primarily used as cofferdams. They may also be used as retaining walls, fixed crest dams and weirs, and walls in locks. Straight sheet piles are used for cofferdam cells, and Z sheet piles are used for walls.

(a) A principal advantage of sheet pile structures is that they may be built in water, thus eliminating any requirement for dewatering.

(b) The most common sheet piling material is steel, but sheet piling may also be made from aluminum, plastic, and concrete. Steel sheet piling specifications are available from Pile Buck Inc., Attn: Publications, P. O. Box 1056, Jupiter, FL 33468-1056.

(5) Special Piles. These piles are useful for special applications and require special construction methods.

(a) Rotated Casing. These piles are installed by rotating heavy-gauge steel tubular casing that has a cutting edge. Soil cuttings are removed with circulating drilling fluid. The interior of the casing may then be filled by pumping a sand-cement grout through a tremie. The casing may be removed while the grout is being pumped into the hole. Reinforcing steel may be placed to increase resistance to lateral and uplift forces. These piles are useful where boulders or other obstructions are encountered.

(b) Expanded Base Compacted (Franki Piles). A steel tube is first driven to the required depth. An enlarged base is formed by driving small charges of zero slump concrete down the tube with hammer blows until the appropriate base is formed (table 1-9). Additional concrete charges are rammed into place against the soil as the tube is withdrawn. These types of piles are expected to provide high skin friction and end bearing resistance.

Table 1-3. Steel Pipe Pile Specifications.

LENGTH	12 - 36 M (40 - 120 FT)
MATERIAL SPECIFICATION	ASTM A36 FOR CORE ASTM A252 FOR PIPE ACI 318 FOR CONCRETE
DESIGN STRESSES OF PIPE	62 MPA (9 KSI); HIGHER STRESSES PERMITTED IF PILE LOAD TESTS ARE PERFORMED AND EVALUATION CONFIRMS SATISFACTORY RESULTS 83 MPA (12 KSI) IF THE PILE IS END BEARING ON ROCK AND IF THE PILE IS EXPECTED TO PENETRATE THROUGH SOIL DEPOSITS WITHOUT OBSTRUCTIONS
DRIVING STRESSES	0.9 YIELD STRENGTH OF THE STEEL Fy 217 MPA (31.5 KSI) FOR ASTM A252, Fy = 241 MPA (35 KSI) 279 MPA (40.5 ksi) FOR ASTM A252, Fy = 310 MPA (45 KSI)
DESIGN LOADS	0.8 - 1.2 MN (80 - 120 TONS) WITHOUT CORES 5.0 - 14.9 MN (500 - 1500 TONS) WITH CORES
DISADVANTAGES	HIGH INITIAL COST DISPLACEMENT FOR CLOSED END PIPE DIFFICULT TO HANDLE WITHOUT DAMAGE TIP SHOULD BE PROTECTED DURING DRIVING
ADVANTAGES	NO DISPLACEMENT FOR OPEN END PIPE CORROSION RESISTANCE CAN BE OBTAINED HIGH LOAD CAPACITIES READILY CUT TO REQUIRED LENGTH CAPABLE OF HARD DRIVING
APPLICATION	HIGH BENDING RESISTANCE FOR LATERALLY LOADED UNSUPPORTED LENGTH FRICTION AND/OR END BEARING PILES
ILLUSTRATION	

Table 1-4. Concrete Cast in Shell Driven with Mandrel.

LENGTH	3 - 16 M (10 - 120 FT), BUT USUALLY 15 - 24 M (50 - 80 FT)
MATERIAL SPECIFICATION	ACI 318 FOR CONCRETE
DESIGN STRESSES	33% OF 28-DAY CONCRETE STRENGTH, WITH INCREASE TO 40% OF 28-DAY STRENGTH PROVIDING (1) CASING IS MINIMUM 14-GAUGE THICKNESS (2) CASING IS SEAMLESS OR SEAMS ARE WELDED (3) RATIO OF STEEL YIELD STRENGTH Fy TO CONCRETE 28-DAY STRENGTH IS NOT LESS THAN 6 (4) PILE DIAMETER IS NOT GREATER THAN 0.43 M (17")
DESIGN LOADS	SPECIFICALLY DESIGNED FOR A WIDE RANGE OF LOADS
DISADVANTAGES	SOIL DISPLACEMENT THIN SHELL MAY BE DAMAGED DURING DRIVING DIFFICULT TO SPLICE AFTER CONCRETE PLACEMENT
ADVANTAGES	INITIAL LOW COST TAPERED OR STEP-TAPERED SHELL CAN PROVIDE HIGHER BEARING CAPACITY THAN STRAIGHT MONOTUBE SHELL IN GRANULAR SOIL CAN BE INTERNALLY INSPECTED AFTER DRIVING LOW STEEL USE
APPLICATION	LOADS USUALLY UP TO 1.0 MN (100 TONS) FRICTION AND/OR END BEARING PILES HIGH BENDING RESISTANCE
ILLUSTRATION	

Table 1-5. Concrete Cast in Shell Driven without Mandrel.

LENGTH	9 - 24 M (30 - 80 FT)
MATERIAL SPECIFICATION	ACI 318 FOR CONCRETE
DESIGN STRESSES	SEE TABLE 1-3
DESIGN LOADS	0.5 - 0.7 MPA (50 - 70 TONS)
DISADVANTAGES	DISPLACEMENT OF SOIL DIFFICULT TO SPLICE AFTER CONCRETE PLACEMENT TIP SHOULD BE PROTECTED DURING DRIVING
ADVANTAGES	CAN BE REDRIVEN SHELL IS NOT EASILY DAMAGED INTERNALLY INSPECTED AFTER DRIVING LOW SOIL DISPLACEMENT WHEN DRIVEN
APPLICATION	FRICTION PILES HIGH BENDING RESISTANCE
ILLUSTRATION	

Table 1-6. Steel HP Section Piles.

LENGTH	12 - 30 M (40 - 100 FT)
MATERIAL SPECIFICATION	ASTM A36
DESIGN STRESSES	62 MPA (9 KSI); HIGHER STRESS PERMITTED IF BASED ON LOAD TESTS
DRIVING STRESSES	0.9Fy, Fy = STEEL YIELD STRENGTH 223 MPA FOR ASTM A36, Fy = 248 MPA (36 KSI) 310 MPA FOR ASTM A572 or A690, Fy = 345 MPA (50 KSI)
DESIGN LOADS	0.4 - 19.2 MPA (40 - 200 TONS)
DISADVANTAGES	VULNERABLE TO CORROSION CAN BE DEFLECTED BY OBSTRUCTIONS CAN BE DAMAGED BY OBSTRUCTIONS; TIP SHOULD BE PROTECTED DURING HARD DRIVING
ADVANTAGES	AVAILABLE IN MANY SIZES AND LENGTHS HIGH CAPACITY EASY TO SPLICE CAN PENETRATE HARD, RESISTANT SOIL OR ROCK IF TIP PROTECTED DRIVES WITHOUT SOIL HEAVE
APPLICATION	FRICTION PILES, END BEARING PILES HIGH BENDING RESISTANCE
ILLUSTRATION	

Table 1-7. Helical Steel Piles.

LENGTH	3 - 6 M (10 - 20 FT)
MATERIAL SPECIFICATION	ASTM A36
DESIGN LOADS	11 - 22 kN/M (0.4 - 0.8 TON/FT) TYPICAL 27 - 54 kN (3 - 6 TONS) TYPICAL TOTAL COMPRESSION LOAD 0.9 MN (100 TONS) MAXIMUM TENSION LOAD
DISADVANTAGES	LIMITED LATERAL BENDING RESISTANCE LIMITED COMPRESSION CAPACITY CAN CORRODE IN SOME SOILS CANNOT PENETRATE OBSTRUCTIONS
ADVANTAGES	CAN BE INSTALLED IN LIMITED SPACE LOW COST EASY TO ADD SECTIONS NO SOIL HEAVE
APPLICATION	ANCHORS RESIST TENSION FORCES SUCH AS FROM EXPANSIVE SOIL OR UPLIFT REMEDIAL REPAIR
ILLUSTRATION	

Table 1-8. Composite Piles.

LENGTH	18 - 60 M (60 - 200 FT)
MATERIAL SPECIFICATION	ACI 318 FOR CONCRETE ASTM A36 FOR STRUCTURAL STEEL ASTM A252 FOR STEEL PIPE ASTM D25 FOR TIMBER
DESIGN STRESSES	33% OF 28-DAY CONCRETE STRENGTH 62 MN (9 KSI) FOR STRUCTURAL AND PIPE SECTIONS SEE TABLE 1-1 FOR TIMBER WEAKEST MATERIAL GOVERNS ALLOWABLE STRESSES
DESIGN LOADS	3 - 10 MN (30 - 100 TONS), WEAKEST MATERIAL GOVERNS ALLOWABLE LOAD
DISADVANTAGES	DIFFICULT TO OBTAIN GOOD JOINT BETWEEN DISSIMILAR MATERIALS EXCEPT FOR THE PIPE COMPOSITE PILE
ADVANTAGES	LOW COST HIGH CAPACITY FOR PIPE AND HP COMPOSITE PILES INTERNAL INSPECTION POSSIBLE FOR PIPE COMPOSITE PILES EASY TO ADD SECTIONS NO SOIL HEAVE
APPLICATION	ANCHORS RESIST TENSION FORCES SUCH AS FROM EXPANSIVE SOIL OR UPLIFT LOADS REMEDIAL REPAIR
ILLUSTRATION	

Table 1-9. Expanded Base Piles.

LENGTH	3 - 18 M (10 - 60 FT)
MATERIAL SPECIFICATION	ACI 318 FOR CONCRETE
DESIGN STRESSES	33% OF 28-DAY CONCRETE STRENGTH 62 MN (9 KSI) FOR PIPE SHELL IF THICKNESS GREATER THAN 3.2 MM (0.125")
DESIGN LOADS	6 - 12 MN (60 - 120 TONS)
DISADVANTAGES	BASE OF FOOTING CANNOT BE MADE IN CLAY OR WHEN HARD SPOTS SUCH AS ROCK LEDGES ARE IN THE PENETRATED SOIL PRECAUTIONS REQUIRED FOR SHAFT GROUPS WHEN CLAY LAYERS MUST BE PENETRATED TO REACH THE BEARING STRATUM
ADVANTAGES	HIGH CAPACITY SHAFTS CAN BE PLACED WITHOUT EXCAVATION OR DEWATERING HIGH BLOW ENERGY AVAILABLE FOR OVERCOMING OBSTRUCTIONS HIGH UPLIFT RESISTANCE WHEN ADEQUATE STEEL REINFORCEMENT
APPLICATION	GRANULAR SOIL WHERE BEARING CAPACITY CAN BE OBTAINED BY COMPACTION AT THE BASE 1.2 - 1.8 M (4 - 6 FT) MINIMUM SPACING
ILLUSTRATION	

(c) Thermal Piles. These piles are designed to ensure long-term thermal stability of the foundation by preventing degradation of permafrost in low temperature regions. Details of these piles are provided in TM 5-582-4.

b. Material Specifications.

(1) Timber Piles. Most timber piles are softwoods such as varieties of Douglas fir and southern pine.

(a) Physical specifications of round timber piles shall conform to ASTM D 25. Other conformed sources are listed in appendix B.

(b) Specifications shall limit defects such as checks, splits, shakes, and knots because these defects can be made more severe by driving. The inner exposed surfaces of treated piles can be subject to decay and attack by insects. A check is a longitudinal separation across the growth rings of a pile starting from the pile surface extending partially into the pile. A split is a check that extends through the pile. A shake is a separation between growth rings of the pile.

(c) Timber piles shall be free of large or loose knots, splits, decay, and sharp bends.

(d) Bark should be removed from timber piles where they are designed as friction piles because a slip can occur between the bark and the trunk. Removal of the bark also improves penetration of preservatives into the wood.

(e) Timber piles shall have a uniform taper from butt to the tip. The piles should be sufficiently straight that a line from the center of the pile at the butt to the center of the tip shall lie within the body of the pile.

(f) Dimensions should be specified to satisfy design requirements. Some specified dimensions may not be fulfilled because the pile may not be driven to the full ordered length. The pile butt in this case may be less than that specified.

(g) Specifications may require the pile tips and butts to be protected with a steel shoe or special fittings, figure 1-2. Steel bands may also be attached at specified intervals along the pile length. This protection helps to prevent the pile from splitting or brooming while driving.

(2) Concrete Piles. Materials consist of concrete reinforcement steel, steel casing, structural steel cores, grout, anchors, and splices.

(a) Material specifications should conform to ACI Committee 543 R-74 reaffirmed in 1980. Piles received from the manufacturer should be accompanied with an inspection report so that design requirements for the piles can be confirmed.

(b) Dimensions should be as specified by the inspection report. Corners and edges of square piles shall be chamfered. The width of the face of the chamfer should not exceed 1.5 inches so that the dimension of the side is not reduced more than 2.0 inches.

(c) Tip protection such as steel H, pipe projections, or steel shoes shall be cast into the pile tips and conform to the specifications.

(3) Steel Piles. Pipe piles may be specified according to grade by ASTM A 252. H-piles may be specified according to ASTM A 36 or ASTM A 572. Laboratory reports must be provided with the piles to ensure that piles will meet design requirements.

(a) The diameter and wall thickness must be at least the minimum values specified. Thicker walls may be required to properly drive the piles without damage or to achieve adequate pile penetration. Dimensions of steel piles are given in TM 5-809-7.

(b) Steel piles may corrode by reacting with oxygen and water to form the metal oxide. Corrosion is minimized where oxygen is deficient such as in clays and clean, fresh water. The potential for corrosion is often significant in swamps, peat bogs, caliche and coarse-grained soils, industrial and mine waste areas, and sea water. Surface paint coatings and encasement in various materials such as concrete are frequently applied to inhibit corrosion.

(c) Pipe piles may be ordered with machine-cut or flame-cut ends, or the ends may be beveled if required for welding. Milled finishes are not required.

(d) Tip protection provided by steel shoes as shown in figure 1-2 is particularly important for H-piles subject to hard driving to refusal in or on a hard layer.

c. General Installation Method. Driven piles are forced into the ground using a pile hammer resting or clamped to the pile butt. A cap block assembly is fitted between the hammer and the pile. Alignment of the pile with the hammer is provided by leads suspended by a crane, except when vibratory hammers are used.

(1) Impact Hammers. Impact hammers drive the pile into the ground by applying a dynamic force to the pile butt. Several types of hammers are available depending on the energy requirement.

(2) Vibratory Drivers. Vibratory hammers drive the pile into the ground by a push and pull action of counter-rotating weights. Vibratory action of the driver causes soil adjacent to the pile to be like a viscous fluid with little or no skin friction.

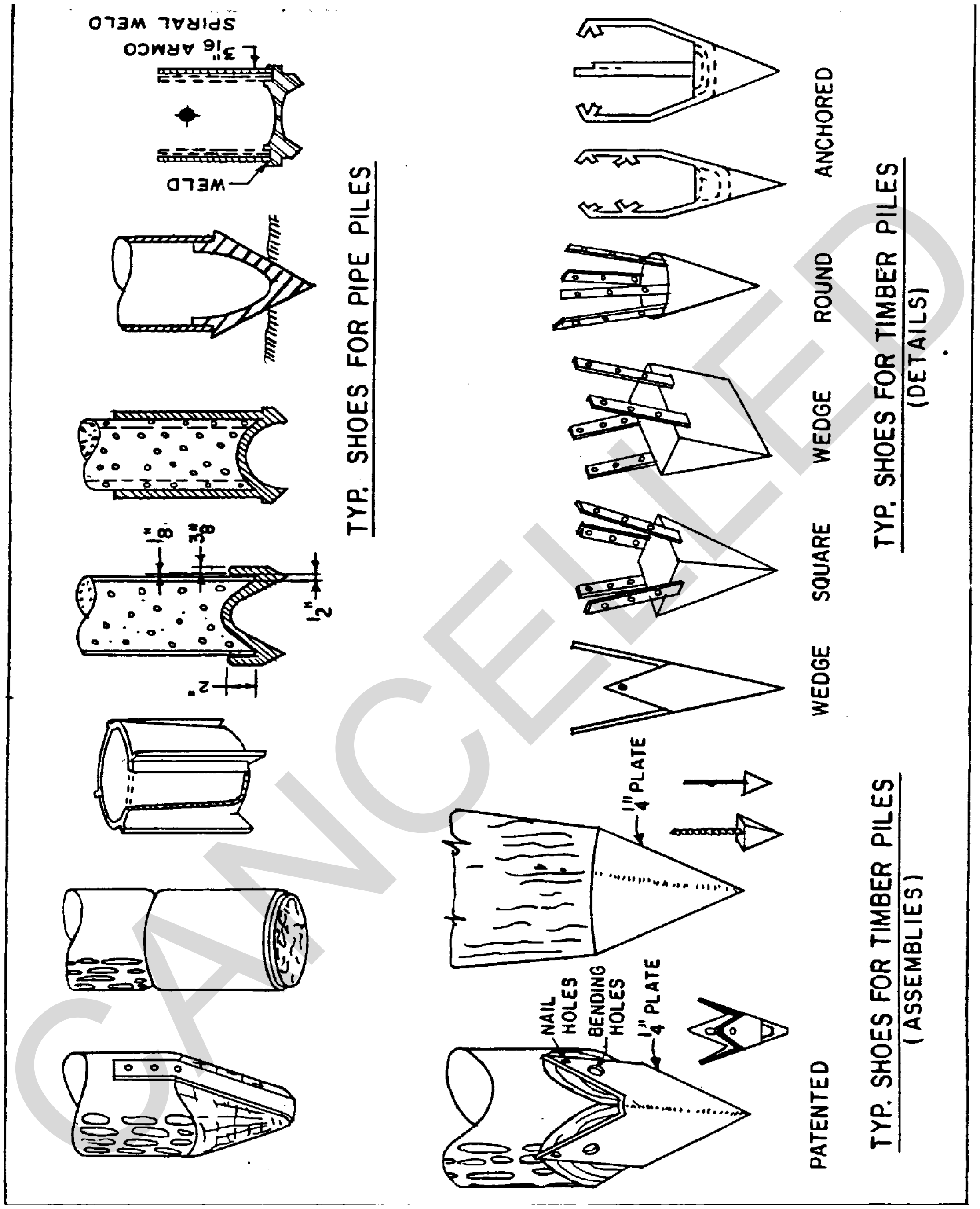

Figure 1-2. Typical pile shoes.

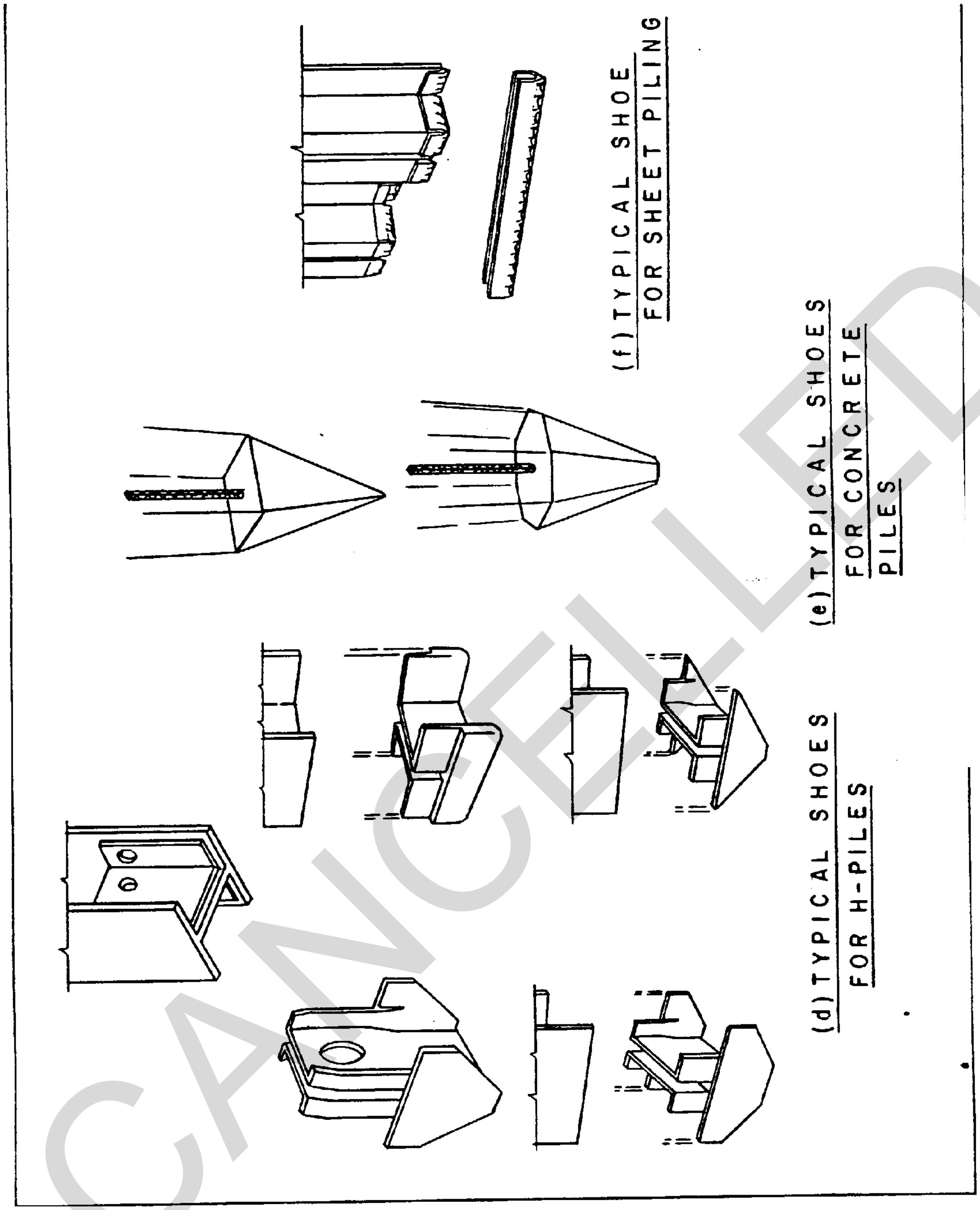

Figure 1-2. Typical pile shoes (concluded).

CHAPTER 2

FACTORS FOR DESIGN AND CONSTRUCTION

2-1. EQUIPMENT SELECTION. This chapter describes the criteria by which pile driving equipment is selected.

a. Commercial Factors. The Government may use either Government-owned equipment or rent pile driving equipment from a commercial source. First consideration should be given to the use of Government-owned equipment, and a thorough search should be carried out to ascertain if such equipment is available. If suitable equipment cannot be located as a result of this investigation, then select rental equipment on the basis of the other criteria in this chapter.

(1) Proximity and Availability. The importance of using readily available equipment cannot be overemphasized. Unless the requirements of the foundation dictate otherwise, equipment selected should be in reasonable proximity to the job site, available from more than one source, and in time for construction.

(2) Size of equipment. Larger equipment is generally less available than smaller equipment. Use of large equipment that is not readily available will result in delays and, with rental equipment and higher rental rates, will add to the cost of the job.

b. Noise. The entire matter of allowable noise disturbance is subjective and should be carefully evaluated before seeking special methods to reduce its effect. Pile driving can generate high noise levels. In many cases, proper explanation of needs, development of alternative methods and job site arrangements, and/or judicious selection of hours of operation can eliminate confrontation.

(1) Noise from Impact Hammers. Impact hammers produce the highest sound pressure levels. There are two primary types of noise which are produced by an impact hammer. The first is impact noise produced by the ram striking the pile. The second type of noise is produced by the operating steam, air, or diesel exhaust as it is exhausted from the cylinder. (This is not present with hydraulic impact hammers.)

(a) Impact Noise. Cushion material can be used to reduce the noise levels as well as modify the impulse duration as required by soil type and piling composition. Also, when driving steel piles, a canvas hood should be provided to reduce noise.

(b) Hammer Exhaust Noise. The exhaust noise can also be reduced through the use of an exhaust muffler. Any device attached to the hammer must be secured properly to prevent it from being jarred loose during driving.

(2) Other Equipment Types. Preboring and the use of vibratory hammers may produce lower sound pressure levels, but may not be less disturbing than the use of an impact hammer. This depends upon the interaction of the pile and the soil, and whether the hammer and pile resonate in a fashion that produces high noise levels. Where noise must be completely eliminated, jacking and screwing methods are likely to provide the least disturbance.

c. Effects of Vibration. The vibration due to pile driving shall be considered concerning possible damage to adjacent construction. Damage due to vibration is a function of both amplitude and frequency. If there are adjacent structures susceptible to vibration damage, then preliminary investigations should be conducted and vibration monitoring should be done during pile load tests and actual driving if necessary. Some vibration attenuation techniques are given below.

(1) Impact Hammers. With impact hammers, this might indicate use of a smaller hammer or a double acting hammer as opposed to a single acting model.

(2) Vibratory Hammers. With vibratory hammers, this indicates the use of a high-frequency, low-amplitude vibratory hammer. If these recourses fail, it may be necessary to adopt a method of foundation installation other than pile driving. Vibratory hammers typically cause more ground vibration than impact hammers.

d. Obstructions. Where piles must penetrate ground containing numerous obstructions, use of jacked or screwed piles generally is not attempted. However, open-end jacked piles can penetrate through obstructions by the use of a chopping bit.

(1) Hammering. The usual procedure is to use a heavy pile with a driving tip and a heavy hammer to try to force a way through the obstruction. This procedure invites damage to the pile and must be used with caution and only when the pile section has been conservatively proportioned.

(2) Spudding. Another procedure is to use a spud. This is a section of heavy pile which is driven through or past the obstruction and removed for reuse. Then the pile is inserted through the passage so cleared.

(3) Probing. If the group size and spacing permit, it is also feasible to probe for a way through or past the obstructions. Use of a vibratory hammer, in which the pile can be readily extracted and reinserted one or several times, may be a useful solution to this problem.

e. Limited Overhead Clearance. When operating in a situation where the overhead clearance is limited, the use of a very short hammer, the use of special crane and leader arrangements, or the jacking of piles in multiple sections may be required.

f. Selection of Pile. The type of pile depends upon a wide variety of factors, including soil type, corrosion, local availability and cost, contractor preference, and the load bearing requirements of the foundation. The various types of piles and their characteristics are given in chapter 1.

g. Selection of Pile Driving System. The selection of a pile driving system begins with the proper selection of both the pile and the hammer. Generally speaking, the pile should be chosen first. If hammers cannot be obtained following the criteria in section 2-1, then the pile should be altered to enable proper equipment selection. The remainder of the system (crane, leaders, etc.) should be configured around the hammer requirements. The hammer should be changed if the resultant rig is either unavailable or impractical.

(1) Preliminary Analysis. The selection of a pile driving hammer is a process that has several stages and requires careful analysis.

(a) Quick Guide to Hammer Type. A quick guide to the selection of various types of pile driving equipment is shown in table 2-1. As with any method of this sort, this is only a guide; specific jobsite circumstances may warrant use of other methods. Beyond this generalization, the sizing of a particular pile driver is different, depending upon whether the hammer is impact or vibratory.

(b) Pile Resistance Computation. Virtually any analysis of the hammer/pile system to determine drivability will require a computation of the soil resistance. Methods used for such a computation are given in TM 5-809-7. The information computed should include the soil type(s) encountered, resistance for the pile shaft, toe, and, in the case of hollow piles, the inner shaft resistance. For the wave equation, the distribution of the shaft resistance should also be noted.

Table 2-1. Quick Guide to Selection of Pile Driving Equipment
(modified from a chart prepared by G. Barber and W. D. Engle of McKiernan-Terry Corp., Dover, NJ)

SPT Value	Class	Wood Pile	Open-End Pipe Pile	Closed-End Pipe Pile	H Pile	Sheet Piling	Concrete Piling
				Cohesionless Soils			
0-3	Very Loose	I (DA)	I (DA) or V	I (DA) or V	I (DA) or V	V	I (DA)
4-10	Loose	I (DA)	I (DA) or V	I (DA) or V	I (DA) or V	V	I (DA)
10-30	Medium	I (SA)	I (DA) or V	I (DA) or V	I (DA) or V	V	I (SA)
30-50	Dense	I (SA)	I (DA) or V	I (SA) or V	I (DA) or V	V	I (SA)
50+	Very Dense	I (SA)	I (SA)	I (SA)	I (SA)	I (DA) or V	I (SA)
				Cohesive Soils			
0-2	Very Soft	I (DA)	I (DA) or V	I (DA)	I (DA) or V	V	I (DA)
4-8	Medium	I (DA)	I (DA) or V	I (SA)	I (DA) or V	I (DA) or V	I (SA)
8-15	Stiff	I (SA)	I (DA)	I (SA)	I (DA)	I (DA)	I (SA)
15-30	Very Stiff	I (SA)	I (SA)	I (SA)	I (SA)	I (SA)	I (SA)
30+	Hard	I (SA)	I (SA)	I (SA)	I (SA)	I (SA)	I (SA)

Key: I (SA) - Impact Hammer, Single Acting/Free Fall Suggested (Air/Steam, Diesel, or Hydraulic)
I (DA) - Impact Hammer, Double or Differential Acting/Assisted Fall Suggested, Single Acting also permissible (Air/Steam, Diesel, or Hydraulic)
V - Vibratory Hammers

(2) Impact Hammers.

(a) Wave Equation Analysis for Drivability. The best method to size an impact hammer for a particular jobsite is to conduct a drivability study using a wave equation analysis. In this way, all of the factors that influence drivability can be taken into consideration. The documentation provided with the wave equation program being used will describe this procedure in detail. Wave equation analyses should also be used to determine driving stresses. Those stresses should be compared with the pile's maximum dynamic stress criterion, and the pile and/or hammer should be altered to limit these stresses.

(b) Jobsite Control. Once the hammer has been selected and the installation of the piles is about to begin, a pile load test program should be conducted to verify any hammer selection analysis. During driving, the methods described in chapter 4 should be adhered to as well.

(c) Quick Method (Concrete and Steel Piles). For concrete and steel piles, if a rough estimate is desired, a method is presented in figure 2-1a in metric units and in figure 2-1b in English unit, Preliminary Method for Sizing Hammers for Concrete and Steel Piles, Metric Units (after Florida DOT Specification, Section 455).

(c) Wood Piles. A pile driving hammer for wood pile shall be rated by the manufacturer from 7,200 to 22,500 foot-pounds to deliver a minimum of 35 percent of its energy to the pile. The Engineering News Formula can be used for preliminary determination of drivability.

(3) Vibratory Hammers. At this point, there is no accepted method to determine either the drivability or the bearing capacity of a pile driven by vibration. In the absence of such a method, figure 2-2a (metric units) and figure 2-2b (English units) show a guide for determining whether or not a particular pile can be driven into a particular soil.

(1) Determine the design load R_d for a given pile in kN.

(2) Determine the estimated ultimate resistance R_u. There are two methods of doing this:

(a) This value can be given in the design plans or computed by the methods referred to in paragraph 2-11b.

(b) The design load R_d can be multiplied by a factor of safety, according to the formula

$$R_u = N\, R_d$$

where
- R_u = ultimate pile resistance, kN
- N = factor of safety
  - = 2.0 when static load tests are required
  - = 2.5 when the pile driving analyzer and wave equation analysis are required
  - = 3.0 when only the wave equation analysis is required. (If a factor of safety has been established for the piles elsewhere, then use that value.)

If values from both (a) and (b) are available, then use the greater of the two.

(3) Divide the ultimate resistance value by the pile area using the formula

$$S_u = R_u / A_p$$

where
- S_u = ultimate pile resistance per unit area, kPa. This value should not fall below 6200 kPa with concrete piles or 62,000 kPa with steel piles for subsequent computational purposes
- A_s = pile cross-sectional area, m²

(4) Compute the minimum rated striking energy for the pile hammer for the particular pile at 4 blows per centimeter (10 blows per inch.) The formula used depends upon whether the pile is steel or concrete.

$E_{r\,10BPI} = A_s S_u / (70.08\, e^{(-S_u/25449)}$ . . . . Concrete Piles

$E_{r\,10BPI} = A_s S_u^{1.62}/58500$ . . . . Steel Piles

where
- $E_{r\,10BPI}$ = minimum hammer rated striking energy at 4 blows per centimeter (10 blows per inch) of resistance, kJ

Figure 2-1a. Preliminary Method for Sizing Hammers for Concrete and Steel Piles, SI units (after Florida DOT Specification, Section 455) .

(5) Recompute the ultimate unit pile resistance using the formula

$$S_{u'} = 3\ R_d/A_s$$

where $S_{u'}$ = ultimate pile resistance per unit area for 8 blows per centimeter (20 BPI) case, kPa. This value should not fall below 6200 kPa with concrete piles or 62,000 kPa with steel piles for subsequent computational purposes

(6) Compute the minimum rated striking energy for the pile hammer for the particular pile at 8 blows per centimeter (20 blows per inch.) The formula used depends upon whether the pile is steel or concrete.

$E_{r\ 20BPI} = S_{u'}\ A_s/(100.78\ e^{(-Su'/22989)})$ . . . . Concrete Piles

$E_{r\ 20BPI} = A_s\ S_u'^{1.68}/165486$ . . . . Steel Piles

where $E_{r\ 20BPI}$ = minimum hammer rated striking energy at 8 blows per centimeter (20 blows per inch) of resistance, kJ.

(7) The minimum rated striking energy for the hammer is the greater of $E_{r\ 10BPI}$ and $E_{r\ 20BPI}$.

(8) Compute the minimum ram weight. Having computed the minimum rated striking energy for the hammer, one or more hammers can be selected as possible candidates for driving the pile. The equivalent stroke should be then computed by the equation

$$S_{eq} = 102\ E_r/m_s$$

where S_{eq} = equivalent stroke, m. This value should not exceed 2.44m for subsequent computational purposes
E_r = rated striking energy of the hammer, kJ
m_s = ram mass of the hammer, kg

(9) The minimum ram mass for each hammer can be computed for concrete and steel piles by the equation

$m_{smin} = 17659\ A_s/\text{Ö}(S_{eq})$ . . . . Concrete Piles

$m_{smin} = 161068\ A_s/\text{Ö}(S_{eq})$ . . . . Steel Piles

where m_{smin} = minimum ram weight, kg

A hammer should be rejected if its ram weight is less than the minimum ram weight, and other hammers considered until one has been selected with a sufficiently large ram.

Figure 2-1a. (Concluded)

(1) Determine the design load R_d for a given pile in pounds.

(2) Determine the estimated ultimate resistance R_u. There are two methods of doing this:

(a) This value can be given either in the design plans, or computed by the methods referred to in Paragraph 2-11b.

(b) The design load R_d can be multiplied by a factor of safety, according to the formula

$$R_u = N\,R_d$$

where R_u = ultimate pile resistance, pounds
N = factor of safety
= 2.0 when static load tests are required
= 2.5 when the pile driving analyzer and wave equation analysis are required
= 3.0 when only the wave equation analysis is required. (if a factor of safety has been established for the piles elsewhere, then use that value.)

If values from both (a) and (b) are available, then use the greater of the two.

(3) Divide the ultimate resistance value by the pile area using the formula

$$S_u = R_u/A_p$$

where S_u = ultimate pile resistance per unit area, psi. This value should not fall below 900 psi with concrete piles or 9,000 psi with steel piles for subsequent computational purposes
A_s = pile cross-sectional area, square inches

(4) Compute the minimum rated striking energy for the pile hammer for the particular pile at 10 blows per inch. The formula used depends upon whether the pile is steel or concrete.

$E_{r\,10BPI} = A_s\,S_u\,/\,(21.36\,e^{(-S_u/3690)})$ . . . . Concrete Piles

$E_{r\,10BPI} = A_s\,S_u^{1.62}/5400$ . . . . Steel Piles

where $E_{r\,10BPI}$ = minimum hammer rated striking energy at 10 blows per inch of resistance, foot-pounds

Figure 2-1b. Preliminary Method for Sizing Hammers for Concrete and Steel Piles, Metric Units (after Florida DOT Specification, Section 455).

(5) Recompute the ultimate unit pile resistance using the formula

$$S_{u'} = 3\ R_d/A_s$$

where $S_{u'}$ = ultimate pile resistance per unit area for 20 BPI case, psi. This value should not fall below 900 psi with concrete piles or 9,000 psi with steel piles for subsequent computational purposes

(6) Compute the minimum rated striking energy for the pile hammer for the particular pile at 20 blows per inch. The formula used depends upon whether the pile is steel or concrete.

$E_{r\ 20BPI} = S_{u'}\ A_s/(30.72\ e^{(-S_u'/3333)})$ . . . . Concrete Piles

$E_{r\ 20BPI} = A_s\ S_u'^{1.68}/13569$ . . . . Steel Piles

where $E_{r\ 20BPI}$ = minimum hammer rated striking energy at 20 blows per inch of resistance, foot-pound

(7) The minimum rated striking energy for the hammer is the greater of $E_{r\ 10BPI}$ and $E_{r\ 20BPI}$.

(8) Compute the minimum ram weight. Having computed the minimum rated striking energy for the hammer, one or more hammers can be selected as possible candidates for driving the pile. The equivalent stroke should be then computed by the equation

$$S_{eq} = E_r/W_s$$

where S_{eq} = equivalent stroke, feet. This should not exceed 8 feet for subsequent computations
E_r = rated striking energy of the hammer, foot-pounds
W_s = ram weight of the hammer, pound

(9) The minimum ram weight for each hammer can be computed for concrete and steel piles by the equation

$W_{smin} = 45.5\ A_s/\text{Ö}(S_{eq})$ . . . . Concrete Piles

$W_{smin} = 415\ A_s/\text{Ö}(S_{eq})$ . . . . Steel Piles

where W_{smin} = minimum ram weight, pounds

A hammer should be rejected if its ram weight is less than the minimum ram weight, and other hammers should be considered until one has been selected with a sufficiently large ram.

Figure 2-1b. (Concluded)

(1) For a vibratory hammer to be suitable for a particular application, the dynamic force should suit the equation

$$F_{dyn} \ \$ \ i \ (\$o R_{so} + \$i R_{si} + \$_t R_t)_{/R}$$

where F_{dyn} = dynamic force of vibrator, kN
\$ = beta factor for soil resistance (general)
$\$_i$ = beta factor for soil resistance (outside shaft)
$\$_o$ = beta factor for soil resistance (inside shaft)
$\$_t$ = beta factor for soil resistance (toe)
R_{si} = inside pile shaft soil resistance, kN
R_{so} = outside pile shaft soil resistance, kN
R_t = pile toe soil resistance, kN.
Q = pile factor (0.8 for concrete piling and 1 for all other piling.)
i = soil resilience coefficient (should be between 0.6 and 0.8 for vibration frequencies between 5 and 10 Hz and 1 for all other frequencies.)

Suggested values for \$ are given in table 2-2. The toe resistance and the outside and inside (where applicable with open ended pipe and cylinder pile) shaft resistance should be computed using methods similar to those employed for impact hammers. For extraction, this formula is altered to read

$$F_{dyn} \ \$ \ i \ (\$o \ R_{so} + \$_i \ R_{si} + \$_t \ R_t)_{/R} - F_{ext}$$

where F_{ext} = extraction force of crane, kN.

(2) Once this is known, a possible vibratory hammer for the job should be selected based on minimum permissible dynamic force in kN. The parameter of dynamic mass (the dynamic mass includes any mass of the vibrator not dampened from vibration, the clamp and any mass of the pile) should be noted, along with the frequency and eccentric moment of the machine.

(3) Next the basic parameters of the vibratory hammer/pile system must be checked. The first is the peak acceleration, whose value is computed using the equation

$$n = 102 \ F_{dyn} \ / \ M_{dyn}$$

where n = peak acceleration, g's.
M_{dyn} = dynamic mass of system, kg

Minimum values for this acceleration are given in table 2-3.

Figure 2-2a. Method for Sizing Vibratory Hammers, Metric Units.

If the peak acceleration figure is too low, a vibratory with a higher dynamic force must be chosen and the peak acceleration recalculated. This iteration must continue until the peak acceleration value is achieved.

(4) Next, the required amplitude of the vibratory system is computed using the equation

$$A = 2000\ K\ R\ /\ M_{dyn}$$

where A = Total Cycle Displacement Amplitude, mm
K = Eccentric moment, kg-m

The preferred amplitude values are shown in table 2-4a; combinations of pile, soil, and frequency in shaded boxes show cases where the pile should not be vibrated at the given frequency range and soil condition. Larger amplitudes than those shown are generally permissible.

If the amplitude is insufficient, then a new vibratory hammer with the same or greater dynamic force as the previous one and greater eccentric moment should be chosen and the amplitude checked again.

(5) Finally, the peak velocity should be checked; it is computed by the equation

$$v_{dyn} = 1.561\ n\ /\ 2$$

where v_{dyn} = peak dynamic velocity of vibrating system, m/sec; should fall between 0.5 and 0.8 m/sec, but it can be higher if necessary.
2 = frequency of vibrations, Hz

It should be kept in mind that method presented here is at best a very sophisticated "rule of thumb" and should be supplemented by local experience with both actual piles and soils and good engineering judgment. One item not considered here is the static weight of the system; this can be increased by mounting bias weights on top of the vibratory hammer. This can increase the speed of pile penetration.

Figure 2-2a. (Concluded)

(1) For a vibratory hammer to be suitable for a particular application, the dynamic force should suit the equation

$$F_{dyn}\ \$\ i\ (\$o \cdot R_{so} + \$i \cdot R_{si} + \$_t \cdot R_t)_{/R}$$

where	F_{dyn}	=	dynamic force of vibrator, tons
	\$	=	beta factor for soil resistance (general)
	$\$_i$	=	beta factor for soil resistance (outside shaft)
	$\$_o$	=	beta factor for soil resistance (inside shaft)
	$\$_t$	=	beta factor for soil resistance (toe)
	R_{si}	=	inside pile shaft soil resistance, tons
	R_{so}	=	outside pile shaft soil resistance, tons
	R_t	=	pile toe soil resistance, kN.
	Q	=	pile factor (0.8 for concrete piling and 1 for all other piling.)
	i	=	soil resilience coefficient (should be between 0.6 and 0.8 for vibration frequencies between 5 and 10 Hz and 1 for all other frequencies.)

Suggested values for \$ are given in table 2-2. The toe resistance and the outside and inside (where applicable with open-end pipe and cylinder pile) shaft resistance should be computed using methods similar to those employed for impact hammers. For extraction, this formula is altered to read

$$F_{dyn}\ \$\ i\ (\$o\ R_{so} + \$_i\ R_{si} + \$_t\ R_t)_{/R} - F_{ext}$$

where	F_{ext}	=	extraction force of crane, tons

(2) Once this is known, a possible vibratory hammer for the job should be selected based on minimum permissible dynamic force in tons. The parameter of dynamic mass (the dynamic mass includes any mass of the vibrator not dampened from vibration, the clamp and any mass of the pile) should be noted, along with the frequency and eccentric moment of the machine.

(3) Next the basic parameters of the vibratory hammer/pile system must be checked. The first is the peak acceleration, whose value is computed using the equation

$$n = 2000\ F_{dyn}\ /\ W_{dyn}$$

where	n	=	peak acceleration, g's
	M_{dyn}	=	dynamic mass of system, pounds

Minimum values for this acceleration are given in table 2-3.

Figure 2-2b. Method for Sizing Vibratory Hammers, English Units.

If the peak acceleration figure is too low, a vibratory with a higher dynamic force must be chosen and the peak acceleration recalculated. This iteration must continue until the peak acceleration value is achieved.

(4) Next, the required amplitude of the vibratory system is computed using the equation

$$A = 2\ K\ y\ /\ W_{dyn}$$

where A = total cycle displacement amplitude, inches
K = eccentric moment, inch-pounds

The preferred amplitude values are shown in table 2-4b; combinations of pile, soil, and frequency in shaded boxes show cases where the pile should not be vibrated at the given frequency range and soil condition. Larger amplitudes than those shown are generally permissible.

If the amplitude is insufficient, then a new vibratory hammer with the same or greater dynamic force as the previous one and greater eccentric moment should be chosen and the amplitude checked again.

(5) Finally, the peak velocity should be checked; it is computed by the equation

$$v_{dyn} = 307.24\ n/\text{rounds per minute}$$

where v_{dyn} = peak dynamic velocity of vibrating system, feet per second; should fall between 1.7 and 2.6 feet per second, but it can be higher if necessary.
RPM = frequency of vibrations, rev/min

It should be kept in mind that method presented here is at best a very sophisticated "rule of thumb" and should be supplemented by local experience with both actual piles and soils and good engineering judgment. One item not considered here is the static weight of the system; this can be increased by mounting bias weights on top of the vibratory hammer. This can increase the speed of pile penetration.

Figure 2-2b. (Concluded)

(3) Vibratory Hammers. At this point, there is no accepted method to determine either the drivability or the bearing capacity of a pile driven by vibration. In the absence of such a method, figure 2-2a (metric units) and figure 2-2b (English units) show a guide for determining whether or not a particular pile can be driven into a particular soil.

Table 2-2. Values of $\$_n$.*

Type of Soil	$\$_n$
Round Coarse Sand	0.10
Soft Loam/Marl, Soft Loess, Stiff Cliff	0.12
Round Medium Sand, Round Gravel	0.15
Fine Angular Gravel, Angular Loam, Angular Loess	0.18
Round Fine Sand	0.20
Angular Sand, Coarse Gravel	0.25
Angular/Dry Fine Sand	0.35
Marl, Stiff/Very Stiff Clay	0.40

*Where n = i, o, or t depending upon relative position of pile and soil in question.

Table 2-3. Minimum Peak Acceleration Values.

Type of Pile	Minimum Peak Acceleration n, g's
Steel Sheet Piling; H-Beams; Open Ended Pipe Piles	9
Caissons; Closed Ended Pipe Piles; Heavy Wall Pipe Piles	5
Concrete and Wood Piles	3

Table 2-4a. Amplitude Requirements, Metric Units.

Frequency, Hz	5-12 Hz	12-17 Hz	17-27 Hz
Type of Pile and Soil	Amplitude mm		
Steel Sheet Piling, Open-end Pipe Piles, H-Piles, and Other Piles with At < 150 cm²			
Sandy Soils	--	16-20	8-12
Clayey Soil	--	20-24	12-16
Closed-end Steel Pipe Piles, At < 800 cm²			
Sandy Soil	--	20-24	12-16
Clayey Soil	--	24-30	16-20
Reinforced Concrete Piles, Square or Rectangular Section, At < 2000 cm²			
Sandy Soil	24-30	--	--
Clayey Soil	30-40	--	--
Reinforced Concrete Cylinder Piles of Large Diameter, Driven with Soil Plug Removed; other pile profiles			
Sandy Soil	12-20	8-12	--
Clayey Soil	16-24	12-20	--

Table 2-4b. Amplitude Requirements, English Units.

Frequency, RPM	300-720 RPM	720-1020 RPM	1020-1620 RPM
Type of Pile and Soil	Amplitude inches		
Steel Sheet Piling, Open-end Pipe Piles, H-Piles, and Other Piles lighter than 80 lb/ft			
Sandy Soils	--	0.63-0.80	0.32-0.50
Clayey Soil	--	0.80-0.93	0.50-0.63
Closed-end Steel Pipe Piles lighter than 420 lb/ft			
Sandy Soil	--	0.80-0.95	0.50-0.63
Clayey Soil	--	0.95-1.18	0.63-0.80
Reinforced Concrete Piles, Square or Rectangular Section, Cross Sectional Area less than 310 in²			
Sandy Soil	0.95-1.18	--	--
Clayey Soil	1.18-1.57	--	--
Reinforced Concrete Cylinder Piles of Large Diameter, Driven with Soil Plug Removed; other pile profiles.			
Sandy Soil	0.50-0.80	0.32-0.50	--
Clayey Soil	0.63-0.95	0.50-0.80	--

h. Driving Resistance. It is inevitable that the resistance experienced by the hammer/pile system will vary as the soil varies on the jobsite and as the pile is being driven. Both low- and high-resistance situations call for attention to special conditions. Although low and high resistance will be encountered in driving, pile refusal should never be defined or required with impact hammers for blow counts in the low- or high-resistance ranges as described below.

(1) Low Resistance.

(a) Impact Hammers. Low resistance for impact hammers is generally defined when the number of blows of the hammer is less than 14 blows/centimeter (36 blows/foot). Under these conditions, several possible events must be watched for, including tension cracking in concrete piles, the result of excessive energy returning from the pile toe and not going into the soil; pile run, which can be damaging to the hammer by forcing high impact loads on the hammer frame, and also the possible loss of control of the pile; and inability to use a diesel hammer because it cannot start in low-resistance conditions. Especially on concrete piles, the hammer energy should be reduced during the first part of a pile's penetration to avoid both pile and hammer damage. If moderate to high resistance was anticipated, then low resistance suggests that the soils analysis requires reexamination. This is especially true with cal-careous soils. Consistent low resistance on a job can also indicate that the hammer selected is too large.

(b) Vibratory Hammers. Vibratory hammers experience little difficulty in operation due to low resistance. The largest problem with these hammers in low resistance is keeping the pile straight, especially since a vibratory hammer is generally run free hanging from the crane, and there is little or no lateral support for the system. If a vibratory hammer is operating at a high hydraulic pressure or amper-age during low resistance driving, then there may be difficulty with the vibratory hammer's operation.

(2) High Resistance.

(a) Impact Hammers. High resistance for an impact hammer is any resistance above 94 blows/centimeter (240 blows/foot). Under these conditions, excessive rebound may take place, damaging the hammer and pile top; the pile may be severely damaged by hitting an underground obstruction; and the job may be seriously elongated, which may create contract problems. Consistently high resistance on a job may indicate that the hammer is too small for the job.

(b) Vibratory Hammers. With vibratory hammers, the main problem with high resistance is overloading the hammer. If the hammer operates at consistently full hydraulic pressure or amperage during driving, the hammer is probably too small for the job, and may also overheat or fail in another fashion. Serious mechanical failure of the hammer is also likely if the toe of the pile has contacted rock. High resistance will also lead to excessive clamp heating and wear, and steel piling should not be driven at penetration speeds less than 1 foot/minute.

2-2. DESIGN VERIFICATION. The design will be checked in the field prior to or during initial installation of the piles that had been selected to be sure that the pile foundation will have adequate capacity to support the expected loads. This field check will consist of a feasibility analysis, installation of indicator piles with pile driver analyzer (PDA) equipment and wave equation analysis. Indicator piles will be driven prior to load testing. Pile load tests are recommended and should be completed for economically significant projects. Driving of indicator piles and load tests should be handled in a separate contract from the construction project or as a minimum accomplished prior to the ordering of the bulk of the production piles. Pile lengths should be determined based on final results from the indicator and pile load tests. Refer to chapter 6 of TM 5-809-7 for further information on verification of the design.

a. Feasibility of Foundation Selection. The type of pile, length, and dimensions selected during the design process will be checked to be sure that this is the most economical and suitable foundation to support the expected loads for the soils observed during the exploration program.

(1) Examination of Exploration Data. A thorough exploration program will have been completed to evaluate the design parameters used to determine the optimum foundation. The program should span the full range of site conditions.

(a) Decisions to limit predesign and preconstruction site investigation efforts to save money and time have risks. These risks could potentially result in catastrophic cost escalation/delays due to encountering changed conditions. Avoidance of such a situation requires professional judgment and customer education. A waiver requirement by higher authority to the pile testing procedures for verifying design is described in chapter 6 of TM 5-809-7.

(b) Data from the exploration program should be made available to designers and prospective bidders and contractors and should include complete boring logs showing all encountered strata, locations of changes in strata, and locations of groundwater including perched and artesian pressures.

(c) Cobbles and boulders should be indicated because they may interfere with the driving of piles.

(d) Landfills often contain poorly compacted materials and may not provide soils that will provide adequate pile capacity.

(e) The soil actually encountered during installation of the piles may not provide the penetration resistance that was expected from results of the soil exploration program. A variety of problems can be encountered leading to unexpected behavior of the driven pile as discussed in paragraph 4-3. Costly delays from unexpected pile behavior can be minimized by driving indicator piles with PDA equipment and by performing load tests as discussed in paragraphs 2-2b, 2-2c, and 2-2d below.

(f) Refer to TM 5-818-1 and TM 5-809-7 for guidance on the exploration program.

(2) Economy of Piles. The most economical piles will usually be those that can be obtained and manufactured locally.

(a) Cost estimates should be prepared and a comparison made of alternative pile materials and construction methods.

(b) The least cost alternative capable of providing acceptable pile capacity should be selected depending on the skills of the local construction force, availability of materials, and availability of equipment to install the piles.

(c) Alternative piles where costs are within 15 percent of each other should be included in the contract documents.

b. Indicator Piles. Indicator piles are the same as the actual production piles used to support the structure. These piles are driven at the start of construction to provide information on the behavior of the piles during their installation and to provide an assessment of the actual capacity of the piles. Indicator piles are usually designed to be part of the production piles that support the structure.

(1) Driving of Indicator Piles. Depending on the job size of the production piles, 2 to 5 percent is typically driven as indicator piles at locations specified by the design engineer or at locations that may have inadequate pile capacity.

(a) Locations where indicator piles should be driven include the corners, edges, and center of the site where piles are to be installed.

(b) Indicator piles should be driven at locations where the soil exploration program indicated relatively low standard penetration resistances, loose sands, or soft clays.

(c) The driving of indicator piles in loose sands or some clay soils may indicate penetration resistances that are too low to provide adequate pile capacity. Driving the piles may also cause pore water pressures to increase and further reduce the penetration resistance. Dissipation of pore water pressures over time will cause the penetration resistance to increase and lead to a pile with greater capacity. This is soil freeze.

(d) An indicator pile should be driven where the soil is dense sand, silt, or where the soil is a stiff, fissured clay, friable shale, or a clay stone. Pore water pressures in these soils may decrease while driving and cause the penetration resistance to increase substantially. The penetration resistance may get so great as to exceed the capacity of the selected pile driving system and to exceed the capacity of the selected pile to take the driving stresses. This is soil relaxation.

(2) Pile Driving Analyzer. PDA is recommended while driving the indicator piles to increase the reliability of wave equation analysis. PDA equipment measures signals from two strain transducers and two accelerometers bolted to the pile near its top. These two different sets of measurements are interpreted to provide force versus time and velocity versus time plots of the pile driving. Soil input data such as skin and toe resistances and the distribution of skin resistance can be modified to cause the force and velocity versus time plots to match. This calibrates the wave equation analysis.

(a) Driving stresses calculated by wave equation from PDA results can be checked to be within allowable stresses.

(b) The PDA results when output on an oscilloscope can be interpreted by the PDA technician to indicate the quality of the pile that had been driven and signs of any damage as a result of driving the pile.

(c) The PDA results can indicate the effectiveness of different pile driving systems and assist selection of the best system for pile installation.

(d) Further information on the PDA is indicated in paragraph 4-2 and chapter 6 of TM 5-809-7.

(3) Restrikes. Some indicator piles should be restruck at a later time; e.g., 1 day, 2 days, and 5 days after original driving of the pile. Restriking the pile will indicate any significant change in the penetration resistance that could occur from soil freeze or relaxation. The soil freeze that is observed may provide adequate pile capacity and avoid the need for any redesign and/or remedial treatment of the foundation. If soil relaxation is significant, a pile that may cause negligible soil displacement such as an H-section may be preferred over a pile that could displace the soil significantly such as a precast concrete or steel pipe pile.

c. Wave Equation Analysis. The wave equation analysis is most useful for determining the ultimate pile capacity from the penetration resistance of the pile measured while driving to its embedment depth. This analysis will be performed by the Government as part of the design process.

(1) Computer Program GRLWEAP. The wave equation analysis is accomplished with program GRLWEAP (Goble, et al. 1988) licensed to the U.S. Army Engineer Waterways Experiment Station (USAEWES). Program GRLWEAP and User's Manual with applications are available from the Engineering Computer Programs Library, Information Technology Laboratory of the USAEWES, to offices of the Corps of Engineers.

(a) The data recorded on the pile driving record of the indicator piles plus an estimate of the quake and damping values, skin friction distribution, and a range of ultimate capacities are input into program GRLWEAP. The results of the analysis provide a calculation of the driving stresses and a bearing graph of the penetration resistance versus the ultimate pile capacity.

(b) The quake is the limiting deformation at which the maximum resistance is observed. Quakes are required for both skin and toe (end bearing) resistance and often set at 0.1 inch.

(c) Damping is velocity dependent component of soil resistance, and it is the slope of the velocity of the pile versus the soil resistance. The choice of skin and toe damping can have a substantial influence on the results of the wave equation analysis. Selection of these values depends on whether driving is hard or easy. The GRLWEAP User's Manual provides some guidance for these values. The standard

Smith damping is the recommended option.

(d) The driving stresses calculated by the wave equation analysis are checked with the allowable stresses for the pile to be sure that the structural integrity of the pile will be maintained.

(e) If recorded blow counts are low, but tension stresses calculated by wave equation analysis are too high, the cushion thickness should be increased, the hammer stroke should be decreased, or a heavier ram should be used. A reanalysis with GRLWEAP should then be completed to optimize driving of the production piles.

(f) If the recorded blow count is satisfactory, but compressive stresses in the pile calculated by GRLWEAP are too high, the cushion thickness should be increased or the stroke decreased. A reanalysis with GRLWEAP should then be completed to optimize pile installation.

(g) If the recorded blow count is too high and compressive stresses are also too high, the size of the pile or pile wall thickness should be increased and a reanalysis with GRLWEAP should be completed.

(2) The Bearing Graph. Estimates of the quakes, damping values, and skin resistances of soils are encountered during installation of the piles, and pile and driving equipment parameters should be input into program GRLWEAP to evaluate the penetration resistance versus pile capacity relationship which is the bearing graph.

(a) Hammer efficiencies provided by the manufacturer may be an overestimate of the actual energy absorbed by the pile in the field and could lead to an overestimate of the pile capacity. Significant overestimates of hammer efficiency are also possible for batter piles. A bracket analysis is recommended to provide a probable range of the pile capacity. Results of the PDA and load tests discussed below can be used to improve the results of the wave equation analysis.

(b) Results of wave equation analysis may not be applicable if soil freeze or relaxation effects occur. Restrike of the pile after 1, 2, or 5 days should be performed to provide a better estimate of the actual pile capacity.

(c) Driving stresses calculated by GRLWEAP should not exceed allowable limits. Refer to paragraph 1-6a and TM 5-809-7 for further information on allowable driving stresses.

(d) Results of the wave equation analysis can be contested by the contractor and resolved at the contractor's expense through resubmittals performed and sealed by a registered engineer, by field verification using driving, and load tests approved by the design engineer.

(3) Selection of Pile Driving Equipment. The bearing graph and driving stresses calculated by the wave equation analysis is useful to check the suitability of the selected hammer.

(a) If blow counts are low (5 blows/foot) and tensile or compressive stresses exceed allowable limits, then hammer energy may be excessive and a smaller hammer may be appropriate.

(b) If blow counts are high (240 blows/foot), then a larger capacity hammer may be appropriate.

d. Load Tests. Field load tests determine the axial and lateral load capacity, displacements for applied loads, and prove that the tested pile can support the design loads within tolerable settlement. Load tests verify capacity calculations and structural integrity of the pile. These tests should reflect the range of potential site conditions (e.g., on-shore and off-shore) after a minimum waiting period, usually 1 day. Refer to TM 5-809-7 for further information on load tests.

(1) Categories of Load Tests. Load tests that are frequently performed are proof tests without internal instrumentation which are conducted to only twice the design load. Load tests are also performed to failure with or without internal instrumentation.

(a) Proof tests do not indicate the ultimate pile capacity and, consequently, the pile can be designed to support a higher load than necessary. Proof tests are not adequate when the soil strength may deteriorate over time such as from frequent cyclic loads in some soils. Coral sands, for example, can cause cementation that can degrade with cyclic loads (see chapter 5).

(b) Piles tested to failure allow the reserve capacity and the allowable load to be evaluated. The allowable load when a load test is performed is one-half of the ultimate pile capacity.

(c) Internal instrumentation allows the skin resistance to be measured in the different soil strata through which the pile is driven and the end bearing capacity to be measured in the bearing stratum. The load test results can be used at other locations where the depth of the different soil strata and the bearing stratum varies, thereby reducing the need for additional load tests.

(2) Selection and Timing of Load Tests. Pile tests are not always practical or economically feasible under certain circumstances, particularly for deep foundations or remote locations; but, they are technically desirable for sound and complete engineering practice.

(a) Pile load tests are relatively expensive and may not be justified if the structure has a small number of piles. An important or complex structure could compensate for the added cost of a pile test program when the consequences of a failure will be disastrous. Pile tests are necessary and justifiable when the subsurface data indicate uncertain foundation soils that would compromise the integrity of the design.

(b) Consideration will be given to the timing of a pile test program in relation to the construction schedule. Cost estimates and schedules will include resources for site investigation, test data collection, and design verification. A pile test program by separate contract may be performed for desired subsurface data and to allow completion of the foundation design prior to award of the construction contract if time and funds permit.

(c) Test piles should be located at strategic positions such as near boundaries of the foundation and at interior locations to determine the distribution of the driving resistance. Adequate subsurface investigations and a pile test program should be performed to confirm the design assumptions. The static load test should be conducted on the pile with the lowest driving resistance. The use of PDA equipment permits calibration of wave equation analysis and can reduce the number of load tests required to determine the pile capacity.

(d) The influence of time-dependent strength gain from soil freeze or loss from soil relaxation is frequently important. Provisions should be considered in contract documents for delays during load testing and pile driving operations to allow for soil freeze or relaxation effects so that the true pile capacity can be determined. This requires careful planing and coordination between designers and construction personnel to prevent any undue impacts and delays.

(3) Axial Load Tests. Axial compressive load tests should be conducted and recorded according to ASTM D 1143 but can be modified to satisfy project requirements. The Osterberg load test, where the load cell is placed at the tip of the pile, is useful in situations where a loading frame cannot be constructed.

(a) The standard load test takes the most time and will measure the most consolidation settlement of any of the load tests; but, none of the load tests will measure all of the consolidation settlement.

(b) The quick load test described as an option in ASTM D 1143 is recommended for most applications but may not provide enough time for some soils or clays to consolidate, thereby underestimating settlement for these soils.

(c) The cyclic load test option of ASTM D 1143 will indicate the potential for deterioration in strength with time.

(d) The tension test may be conducted according to ASTM D 3689 to provide information on piles that must function in tension or tension and compression. Residual stresses may significantly influence results; therefore, a minimum waiting period of 7 days is required following installation before conducting this test in cohesionless soil and 14 days in cohesive soil.

(4) Lateral Load Tests. Monotonic and cyclic lateral load tests should be conducted according to ASTM D 3966. This test is used to verify the stiffness of the pile/soil system. Loads should be carried to failure.

(a) Lateral load tests may be conducted by jacking one pile against another.

(b) Groundwater will influence the lateral load response of the pile and should be the same that will exist during the life of the structure. Pile head boundary conditions will also influence the pile response.

(c) Sequence of applying loads is important when conducting cyclic tests. The load level for the cyclic test may be the design load. Alternatively, a deflection criterion may be selected where the pile is loaded to a predetermined deflection. That load level will then be used for the cyclic load tests. The test piles are loaded from zero to the load level and repeated for the required number of cycles according to ASTM D 3966.

2-3. CONSTRUCTION. Successful construction requires a well-written contract, unobstructed access to the site, efficient production methods, and adequate contractor experience.

a. Contract Specifications. Specifications for pile driving contracts shall be prepared with the appropriate guide specification for military construction in accordance with ER 1110-345-720. All approval of National Environmental Policy Act requirements will be documented and in place prior to contract award.

(1) Basic Requirements. Specifications in the contract documents of the pile foundation will be clearly presented and easily understood. These specifications should be flexible to maximize economy, yet rigid enough to result in the desired foundation. The Government should be permitted to focus on the contractor's compliance with specifications and ability of the equipment and methods used by the contractor to produce structurally sound piles driven within established tolerances and capable of developing the required capacity.

(a) Contracts should not be awarded when scope/design is not finalized. Careful consideration of all potential problems and factors affecting contract performance is essential prior to any contract modification. Unilateral contract actions, especially those that use all available overrun authority, are not recommended.

(b) Specifications should indicate whether the piles are to be driven to the design penetration resistance or to a specified bearing stratum.

(c) Specifications should indicate the minimum penetration into the bearing stratum, if the piles are to be driven to a specified bearing stratum. The specification should also give the resistance to where (blows/inch) the piles should be driven if they are end bearing piles.

(d) Specifications should indicate the intended function of the piles such as resistance to compression, tension or uplift, negative skin friction from compaction or consolidation of adjacent soil, and lateral loads.

(e) Specifications should include provision for contractor and Government responsibilities when using PDA equipment. The PDA may be useful to indicate hammer efficiency, driving energy delivered to the pile, driving stresses, pile capacity, and possible damage to the pile. However, measurements may be distorted because of residual stresses in the pile and soil freeze or relaxation effects.

(2) Installation Equipment. Specifications will detail requirements of the pile driving system.

(a) Minimum and maximum energy of the hammer will be specified. The minimum energy is required to be sure that the pile capacity will be properly indicated by the penetration resistance. Hammers that are too light can indicate low displacements/blow without any significant penetration and capacity. The maximum energy is required so that the piles will not be damaged by driving with a hammer that is too large.

(b) Requirements for driving helmets, caps, and hammer and pile cushions will be specified. Cushions that are too soft will absorb too much energy and driving may stall. Cushions that are too hard will cause hammer or pile damage. Commonly used materials for hammer cushions are hardwood, plywoods, woven steel wire, laminated micarta and aluminum discs, and plastic laminated discs. Laminated materials provide superior energy transmission characteristics, maintain their elastic properties uniformly during driving, and have a relatively long useful life. Commonly used pile cushions are plywood and oak board. Engineering experience combined with wave equation analysis is recommended for selecting the cushion materials and thickness.

(c) Specifications will describe the piles selected for the project.

(d) Specifications should clearly define the basis of hammer approval (e.g., by results of wave equation analysis) and state criteria that will be used to establish the penetration limits. Installation equipment or methods suspected of compromising the pile foundation should be clearly excluded from the specifications. The contractor may question these exclusions and may substantiate any claim at the contractor's expense by performing an independent wave equation analysis, field verification of driving and static load tests, dynamic monitoring, or other methods designated by the designer.

(3) Pile Installation. Specifications will describe the piles selected for the work and how the piles will be driven.

(a) Care and handling of piles will be specified to avoid overstressing. Sweep (camber) limitations will be specified because excessive sweep can cause piles to be driven out of tolerance. For example, sweep may be limited to 2 inches for steel H-piles. The required number and locations of permissible pick-up points on the pile will be clearly indicated in the specifications. Loading and unloading of long steel piles should be done by supporting at a minimum of two points about one-fourth length from the ends of the pile. Precast concrete piles should be supported at several points when they are lifted to the driving position. Any deviations from the specifications must be approved by the design engineer.

(b) Type of lead, whether fixed or swinging, shall be specified to assure adequate alignment. Driving tolerances will be specified to assure adequate positioning and vertical alignment. A lateral deviation from the specified location at the pile head of not more than 3 to 6 inches measured horizontally and a final variation in alignment of not more than 0.25 inch/foot measured along the longitudinal axis should normally be permitted. A deviation of ±1 inch from the specified cutoff elevation is reasonable. These recommendations are for large pile groups and should be verified for specific projects.

(c) Splices will be specified if these are required.

(d) Cutting of the piles will be specified as required by the design.

(e) Driving shoes and the manner of attachment will be specified if these are required by the design.

(4) Refer to the following examples of guide specifications.

(a) Steel H-piles. CEGS-02360 covers the requirements for procurement, installation, and testing of steel H-piles.

(b) Round Timber Piles. CEGS-02361 covers the requirements for round timber piles for fresh water and marine use.

(c) Prestressed Concrete Piles. CEGS-02362 covers the requirements for prestressed concrete piles for fresh water and marine use.

(d) Cast-In-Place Concrete Piles with Steel Casing. CEGS-02363 covers the requirements for procurement, installation, and testing of cast-in-place concrete piles utilizing steel casing.

(e) Wood and Cast-In-Place Concrete Composite Piles. CEGS-02365 covers the requirements for composite, wood on cast-in-place concrete piles.

b. Site Constructibility. The site should be prepared for optimum construction efficiency.

(1) Topography of Land Sites. The site should be prepared for efficient movement of the pile rig, for access to and from the site, for delivery of materials, and for storage of equipment and materials.

(a) The site should have adequate drainage to prevent water from ponding. Dewatering will be provided unless specifications allow for wet installation.

(b) The construction area should be level to facilitate driving of the piles, unless construction is required on a slope. Sloping surfaces may require field adjustment of the pile location if the actual surface differs from the reference plane used in the plans to show the pile location.

(2) Surface Soils. The top soil should be gravel, sands, or clays with low plasticity index $PI < 12$ and liquid limit $LL < 35$ to provide mobility. Lime can be sprinkled on the surface of high plasticity clays to improve mobility in wet weather.

(3) Overhead Clearance. The pile driving rig, boom, leaders, and clearance required when raising and placing the pile in the leader should be determined and checked to be sure that clearance at the site is adequate.

(4) Coordination. Proper scheduling of delivery of equipment and materials is required to provide continuous operation of the work.

(a) Site preparation should be completed on schedule to avoid delays in delivery of materials and equipment.

(b) Piles to be driven should be prepared by completing any preparatory splicing, cutting, and inspection prior to the time of actual installation.

c. Production of the Pile Foundation. Installation of the pile foundation requires systematic criteria to optimize productivity and minimize problems.

(1) Pile Preparation. Piles will be prepared for installation by splicing to required lengths, attachment of fittings such as driving shoes and bands, and application of coatings. Coatings to protect the pile from corrosion or to reduce negative skin friction may be required by the specifications.

(a) Timber Piles. Steel bands may be required at the top of the pile and at specified intervals to maintain the integrity of the pile. The top of the pile will be cut square with the pile axis and chamfered when necessary to fit the drive cap. A steel band may also be necessary at the top to prevent splitting. All cuts should be chemically treated after driving if the pile had previously been treated. Driving shoes may also be required at the pile tip.

(b) Steel Piles. Splices may be required to achieve the necessary lengths. Half of the splice may be attached or welded to the section to be installed to minimize time required to splice the section to the pile previously driven into the ground. Driving shoes will be attached or welded to the tip if required by the specifications.

(c) Concrete Piles. Sections of concrete piles will be attached with splice or joint fittings while being cast by the manufacturer.

(2) Pre-excavation. The soil may require Pre-excavation to facilitate installation. Preboring and prejetting or jetting while driving are the most common methods of facilitating the pile installation.

(a) Pre-excavation may be required to assist installation in hard or dense soils or to reduce or eliminate soil heave.

(b) The type of Pre-excavation depends on the soils. Jetting is effective in granular soil, while preboring is effective in cohesive soil. If jetting is permitted, restrictions will be required on the size of the jet, volume and pressure of water, and the depth of jetting to minimize the loss of pile capacity after installation. Jetting disturbs the soil around and below the pile. The pile should be seated with an impact hammer after jetting.

(3) Handling and Lofting Piles. Careful handling of the pile is required while the pile is being aligned into the proper position for driving.

(a) Hollow piles will not be dragged along the ground with the open end first to prevent the pile from picking up debris.

(b) Pointed tools will not be used with timber piles that could damage the wood.

(c) Concrete piles will be handled at designated contact points. The top slings for lifting coated concrete piles should be attached to the bottom sling to prevent slippage and damage to the coating and to provide safety while lifting. Coatings applied to reduce negative skin friction are usually not applied at the bottom of the pile so that the bottom sling will hold in place. Coatings applied full length require special provision for lifting such as boring lugs into the concrete.

(d) Holes can be cut in steel piles for attachment of lifting shackles if the portion of the pile with the holes will later be cut off.

(4) Length of Piles in the Leader. If part or total payment for pile installation is based on the length of pile raised in the leaders, then a record will be kept of the length for each pile in the leader. For piles where length is added to the leader, the pile spliced, and some of the length cut from the pile, the length of pile over which payment is made will be adjusted to avoid payment for pile length that is cut from the pile.

(5) Splicing. Splices are necessary if the length of the pile cannot be made long enough to reach

the embedment depth, overhead clearance is not sufficient for the full length of the pile, or the driving rig cannot accommodate the full length of the pile. Splices available for joining various types of piles are provided in figure 2-3. These splices should be made as strong as the pile.

(a) Splices for composite piles, especially concrete-steel or concrete-wood, are time consuming and difficult to accomplish. Composite piles should be avoided for this reason.

(b) Splices made during installation of the pile take time and should be avoided where soil freeze can occur. Soil freeze increases the penetration resistance and makes further driving of the pile more difficult and may even lead to driving stresses that could damage the pile.

(c) The type and quality of splice can be determined by the contract pile installation documents if splices can be made in the leader. The top of the previously driven portion of the pile may have been damaged and may require some cutting and leveling before the next section can be added. Fittings for part of the splice should be secured to the pile that is waiting to be driven to expedite the splicing operation.

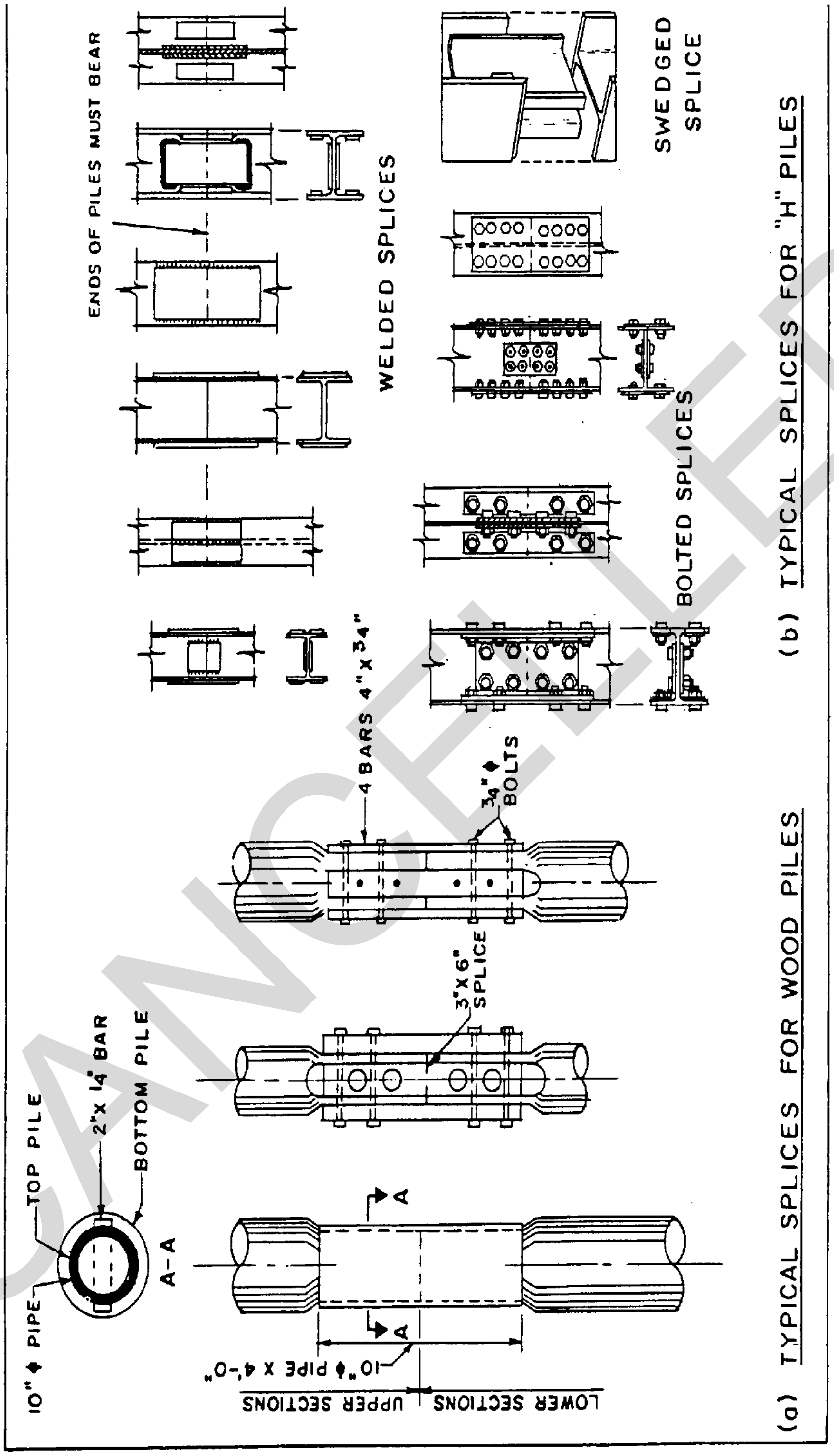

Figure 2-3. Typical splices for piles.

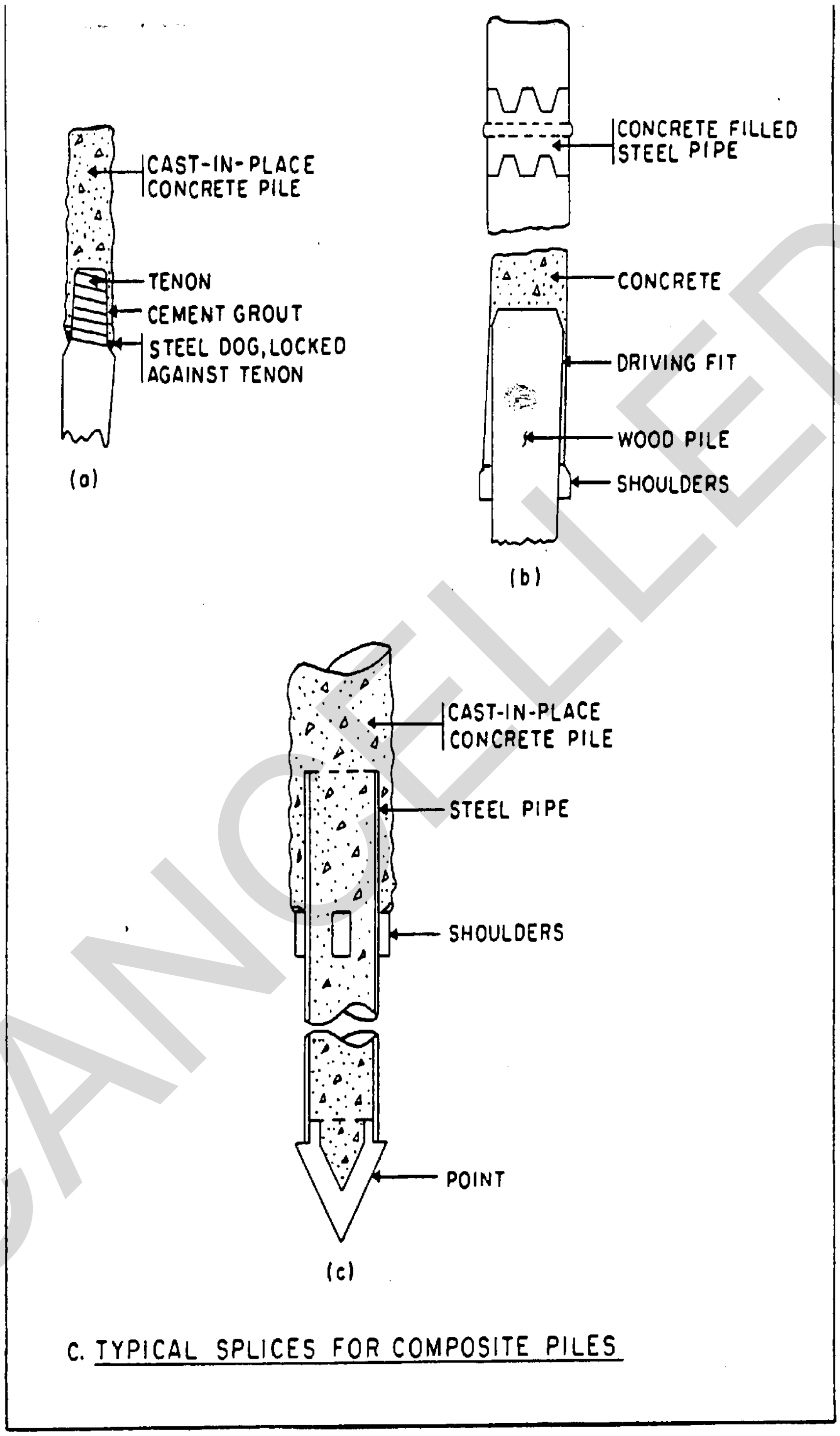

Figure 2-3. (Concluded)

(6) Cutting. The piles will be cut at the top to the proper elevation required by the design. Damaged sections will also be cut from the pile before driving into the soil and splices made to restore the pile to the required length.

(7) Spotting. The tip of the pile will be accurately positioned over the location stake for the pile. The position can be checked by measuring the pile tip relative to two stakes driven 90 degrees apart.

(a) An accurate position of the pile can be made with fixed leaders and an adjustable spotter.

(b) A template is sometimes used to facilitate accurate positions.

(c) H-piles should be driven with their flanges in the correct orientation as shown by the plans.

(8) Alignment. Piles are aligned either vertically or at a specified batter.

(a) Alignment of vertical piles can be checked after spotting and before driving with a carpenters level. The initial alignment is critical for piles driven through water or overburden and where the final cutoff elevation is below the water or ground surface. The pile alignment should be checked periodically while driving and on exposed lengths of not less than 5 feet.

(b) Alignment of batter piles can be assured with a wood template that is cut to the correct batter angle. The template is held against the leader, pile, or the drill stem.

(c) The drill rig should be positioned on level ground. Timber mats can be used to support the drill rig and to maintain a level orientation if the ground surface does not stable.

(9) Sequence of Installation. The displacement of the soil and vibration associated with the driving compacts the soil. The penetration resistance at the design embedment depth will often be greater than that of the piles driven earlier. Smaller spacings between piles increase compaction and the penetration resistance. Driving should begin at the center of the pile group and proceed toward the ends of the group.

(10) Heaved Piles. The compaction of the soil as a result of driving may cause the piles driven first in the group to heave when later piles are driven. Piles that have heaved as a result of pile driving should be restruck to determine that the penetration resistance is adequate and that the piles are driven to the required embedment depth.

(11) Pile Driving System. The piles will usually be installed by either impact hammer or vibratory driver. Results of the exploration program, wave equation analysis, driving of indicator piles with PDA, and pile load tests will assist design of the pile foundation, determine the pile and length to be installed and the driving energy. A minimum hammer size will be specified in the contract documents.

(a) Installation by impact hammers will require specification of the pile length and/or the penetration resistance at the embedment depth.

(b) Installation by vibratory drivers will require specification of the embedment depth or elevation of the pile tip. The specifications will usually require that an impact hammer be used to confirm the penetration resistance, especially if the pile capacity is primarily from end bearing. Vibratory drivers are most effective when installing friction piles and may not require confirmation of capacity by an impact hammer.

d. Contractor Experience. The work shall be performed by a contractor that specializes in the foundation system to be installed. The contractor shall have had previous experience on installing the piles specified in the contract documents. This experience will be compatible with the soils that will be encountered during installation of the piles.

2-4. CARE AND MAINTENANCE. Proper care and maintenance is required to avoid damage to the piles before installation.

a. Timber Piles. Pointed tools should be avoided when handling timber.

(1) Treated piles should be handled to avoid puncturing or breaking through the outer portions of the piles. The treatment may not extend deeper than about 1 inch into the pile. Treated piles should also not be dragged along the ground surface.

(2) Timber piles must be stored on blocks above the ground surface if not installed within 1 week of delivery to the construction site. An air space must be provided beneath the piles and the piles must not lie in water. The blocks must be made of nondecaying materials. The timber must be kept free of debris and decayed materials. The blocks must be spaced at intervals to provide adequate support and to prevent permanent bends.

b. Concrete Piles. These piles should be supported at sufficiently frequent points in slings to avoid overstressing from bending and cracking.

(1) Long unsupported lengths are prohibited. A three-point pickup may be required for piles longer than 100 feet.

(2) Impacts along the pile length and dumping while unloading are prohibited. A lifting beam or spreader slings are required if the piles are coated.

c. Steel Piles. A one-point pickup is usually adequate for steel pipe piles, but bending of the piles must be avoided. Long H-piles must be lifted horizontally with webs vertical and with a sufficient number of pickup points to avoid bending. Damage to coatings must be prevented while handling.

(1) Closed-end pipe piles should not be dragged along the ground surface with the open end first to avoid accumulation of debris within the pile.

(2) Piles should be stored above the ground surface and above standing water. Chocks should be used to prevent pipe pile stacks from spreading. H-piles should be nested with flanges vertical and stored on adequate numbers of timber blocks to avoid bending.

CHAPTER 3

EQUIPMENT

3-1. IMPACT HAMMER PILE DRIVING SYSTEM. Impact hammers are hammers which drive the pile by first inducing downward velocity in a metal ram, as shown in figure 3-1. Upon impact with the pile accessory, the ram creates a force far larger than its weight, which, if sufficiently large, then moves the pile an increment into the ground. An idealized version of this force is shown in figure 3-2.

a. Hammers. The characteristics, strengths, and weaknesses of various types of impact hammers are shown in table 3-1. A listing of impact hammers presently manufactured is shown in table 3-2. This table is divided into three parts. Table 3-2a is an instruction for the use of the tables. Table 3-2b is in the SI unit measurements, and table 3-2c is in the English units. Impact hammers can be divided into two categories, external combustion and internal combustion.

(1) External Combustion Hammers. External combustion hammers are hammers which burn the fuel that provides the energy for the operation of the hammer outside of the hammer itself. These hammers have external power sources such as the crane itself, steam boilers, air compressors, and/or hydraulic power packs to provide the energy to move the ram upward, and in some hammers, downward as well. The various types of external combustion hammers are detailed below.

(a) Drop Hammers. The drop hammer is the oldest type of pile driving hammer in existence. A typical drop hammer is shown in figure 3-3. The hammer is connected to a cable which is attached to a winch on the crane. The hammer is raised to the desired stroke. The winch has a clutch on it that then allows the operator to release the hammer, which falls by its own weight and strikes a pile cap and the pile. Drop hammers are mainly used on very small jobs and for small piling.

(b) Single-Acting Air/Steam Hammers. These hammers use steam or compressed air to raise the ram. At a point in the upstroke, the valve is moved and the ram floats to the top of the stroke; the ram then falls by its own weight and makes impact. These hammers are generally referred to as "air/steam" because they can be operated by air or steam; a few are operable by only one or the other. A typical single-acting air/steam hammer is shown in figure 3-4; the operating cycle is shown in figure 3-5. Many air/steam hammers contain a device to change the upstroke valve turnover point as shown in figure 3-6. This device enables the hammer to operate at two energies, a capability which is especially important during the installation of concrete piles.

(c) Double-Acting Air/Steam Hammers. These hammers are similar to the single-acting hammers except that, upon upstroke valve turnover, they apply steam or air pressure to the top of the piston. This enables the stroke to be shorter, as it accelerates the ram to the desired impact velocity more quickly than with single-acting hammers. This makes a higher blow rate possible, which is advantageous in some situations. A typical double-acting air/steam hammer with a description of the operating cycle is shown in figure 3-7. Double-acting hammers are especially popular in driving sheet piling where vibratory hammers cannot penetrate the soil or where they are favorable economically.

(d) Differential-Acting Air/Steam Hammers. These are similar to double-acting hammers except that the air or steam is constantly pressurized under the piston. This allows for a simpler valve configuration than with a double-acting hammer with similar operating characteristics. A typical differen-tial-acting hammer is shown in figure 3-8. Its operating cycle and characteristics are shown in figure 3-9.

(e) Hydraulic Impact Hammers. These hammers substitute hydraulic fluid for air or steam, and it is applied to the piston to move the ram. Hydraulic impact hammers can be single acting, double acting, differential acting, or other variations. Most but not all hydraulic hammers employ the use of an electric valve operated with a variable timer. The timer allows for very flexible control of the output energy. A typical hydraulic hammer is shown in figure 3-10.

(f) Jacking. Pile jacking machines are not true impact hammers but act by simply pushing the pile into the ground. Such a machine is depicted in figure 3-11. They are most effective when the soil resistance is lower than the maximum ram force and when there are neighboring piles to jack against (such as with sheet piling). They are most advantageous when vibrations and noise must be minimized.

(2) Internal Combustion Hammers. These hammers burn the fuel that powers them inside of the hammer, and for the most part, the diesel hammers are the only constituent of this class.

(a) Open-end Diesel Hammers. The open-end diesel hammer operates as is shown in figure 3-12. The piston, with the assistance of the starting device driven from the winch of the pile driving rig or hydraulically, is raised to an upper position. At this point, it is released by the starting device and falls down under its own weight. Before the bottom of the ram passes the exhaust ports, the piston pushes the fuel pump lever, and fuel from the pump is supplied to the spherical recess of the anvil (some models directly inject atomized fuel into the combustion chamber). At the bottom of the stroke, the piston impacts the anvil. The energy of impact is divided between fuel vaporization and its mixing with heated air and driving of the pile. After a short period of time, the air-fuel mixture is ignited, and because of the pressure of the expanding exhaust gases the piston is raised up and additional driving impulse is transmitted to the pile. A typical open-end diesel hammer is shown in figure 3-13.

(b) Closed-end Diesel Hammers. These are similar in operating principle to the open-end type except that a compression chamber or vacuum is employed on top of the piston to assist the ram in the downstroke. This speeds up the blow rate of the hammer, but some of these hammers have a heavier ram relative to the energy than the open-end type. A typical closed-end diesel hammer is shown in figure 3-14.

b. Driving Accessories. It is not possible for the striking end of the ram of an impact hammer to directly adapt itself to all shapes of piles; therefore, it is necessary to have driving accessories of various types to be inserted between the bottom of the hammer and the pile to both mate the two geometrically and transmit the force of impact from hammer to pile.

(1) Hammer Cushion. Most impact hammers have some kind of cushion under the end of the ram which receives first the striking energy of the hammer. This cushion is necessary to protect the striking parts from damage; it also modulates the force-time curve of the striking impulse and can be used to match the impedance of the hammer to the pile, thus increasing the efficiency of the blow. The actual material of the cushion and its configuration will vary, depending upon the hammer configuration and the cushion material being used. Any hammer cushion should be installed and used in accordance with the recommendations of the hammer manufacturer. Figure 3-15 shows a variety of cushion configurations. Table 3-3 shows a summary of the various types of cushion materials and their characteristics.

(2) Anvil. The rams of most external combustion hammers strike the cushion material or top plate directly. With internal combustion hammers, an anvil is necessary to trap the combustible mixture and thus allow it to build pressure. Figure 3-12 shows an anvil setup. The term “anvil” is also sometimes used to describe the drive cap or helmet (see next paragraph).

(3) Helmet. The helmet actually mates the hammer system to the pile. In doing so, it distributes the blow from the hammer more uniformly to the head of the pile to minimize pile damage. Figure 3-16 shows some typical driving helmets.

(4) Pile Cushion. When driving concrete piles, it is necessary to use a cushion between the hammer and a pile. This cushion is generally made of plywood. One cushion is made for each concrete pile to be driven, and it is installed on top of the concrete pile before it is driven or in the cap. The depth of this cushion can vary from 6 to 18 inches. Figure 3-1 shows a typical pile cushion configuration.

(5) Mandrel. A mandrel is used to install thin-wall shell piles which are subsequently filled with concrete. The mandrel is necessary because the wall is too thin to withstand the stresses of driving. Figure 3-17 shows examples of mandrels.

3-2. VIBRATORY DRIVERS. A vibratory pile driver is a machine that installs piling into the ground by applying a rapidly alternating force to the pile. This is generally accomplished by rotating eccentric weights about shafts. Each rotating eccentric produces forces acting in a single plane and directed toward the centerline of the shaft. Figure 3-18 shows the basic setup for the rotating eccentric weights used in most current vibratory pile driving/extracting equipment. The weights are set off center of the axis of rotation by the eccentric arm. If only one eccentric is used, in one revolution a force will be exerted in all directions, giving the system a good deal of lateral whip. To avoid this problem, the eccentrics are paired so the lateral forces cancel each other, leaving us with only axial force for the pile. Machines can also have several pairs of smaller, identical eccentrics synchronized and obtain the same effect as with one larger pair. The classification of these machines is shown below.

a. Low-frequency Hammers. These are vibratory drivers with a vibrator frequency of 5 to 10 Hz, used primarily with piles with high mass and toe resistance such as concrete and large steel pipe piles. They tend to have large eccentric moments to achieve their dynamic force with high resultant amplitudes. An example of this type of machine is shown in figure 3-19.

b. Medium-frequency Machines. These are drivers with a vibrator frequency of 10 to 30 Hz, used for piling such as sheet piles and small pipe piles. An example of this type is shown in figure 3-20. These machines make up the majority of vibratory pile drivers in use today, since they combine the dynamic force necessary to excite the soil, the correct frequency to properly interact with most soils, and the sufficient amplitude to get through the hard spots in the soil.

c. High-frequency Machines. These consist of all machines which vibrate at frequencies of more than 30 Hz. They are of two basic types. The first are machines in the 30 to 40 Hz range which are designed primarily to minimize vibration of neighboring structures. These have been developed simultaneously both in Europe and in the United States, and they are similar in construction to the medium-frequency machines. The primary advantage of these machines is their lowered transmission of ground excitation to neighboring structures. The frequencies of these machines are not high enough to improve driving. In some cases, these machines have problems in overcoming toe resistance.

d. Sonic or Resonant Hammers. In a class by itself is the resonant pile driver, first introduced in the early 1960's. The central principle of the resonant driver is to induce resonant response in the pile, thus facilitating driving and extracting. The resonant driver operates at frequencies in the range of 90 to 120 Hz. In most cases the driving took place at the half-wave frequency of the pile. The ability to achieve this response was dependent upon properly matching the frequency range of the machine to the length of the pile. In cases where this was not possible in a normal hammer/pile setup, a heavy wall follower connected the pile with the hammer. When the pile is exceptionally long, second and third overtones can be achieved. Although, in principle, this concept has held great potential. The mechanical complexity of this machine has withheld it from extensive use.

e. Impact-vibration Hammer. The term “impact-vibration hammer” refers to a type of vibratory pile driver that imparts both vibrations and impacts to the pile during operation. Such a machine is shown in figure 3-21. In common with more conventional vibratory hammers, it contains counter rotating eccentrics which impart vertical vibrations; however, these are contained in a head which is free of the pile to some degree. Generally speaking, its motion is regulated by a set of springs which link it to the frame. The frame can be connected to the pile in numerous ways. These springs transmit their compression force, which is of a vibratory nature, to the pile. In addition to this, depending upon the position of the head relative to the impact point and the effect of the springs on the vertical motion of the head, (at or near the bottom of the vibratory cycle), the head strikes the anvil and produces an impact similar to traditional impact hammers, but at a higher blow rate. Although this can produce variations in the eccentric rotational speed of up to 40 percent (as opposed to the

5 percent or so normal for vibratory hammers), this variation generally does not impede the continuous, stable operation of the equipment. These hammers have very limited usage, at the present time, and are not manufactured in the United States.

f. Types of Pile Driven with Vibratory Hammers. Vibratory hammers are used to drive a large variety of piles. A quantitative method of evaluating these was presented in chapter 2.

g. Qualitative descriptions are listed below.

(1) Sheet Piling. To drive steel sheet piling, the sheeting is set up according to normal American practice, namely to set the wall in place using a template and then to drive the pile to the desired depth. This practice requires that the vibratory hammer be no wider at the throat than about 14 inches as the hammer must clear the adjacent piles. In driving sheeting in this way, it is also normal to drive the sheets two at a time, using a jaw with two sets of teeth and a recess between them large enough to accommodate the interlock. An alternate method of driving sheeting with a vibratory hammer is to set the sheet piling as they are driven. As a rule, in this case the sheets are driven one at a time. Impact-vibration hammers have been designed for and used with steel sheet piling in soils that produce high toe resistances and thus are not congenial to vibratory driving. Hammers that are used for both driving and extraction are able to impact both upward and downward. Impact-vibration hammers for sheet piles generally are equipped with clamps instead of an inertial frame.

(2) H-Beams. The conditions to drive H-beams are similar to driving sheeting; however, when the pile's batter angle is critical, the vibratory hammer can be mounted in a set of leaders much as is done with an impact hammer. In addition to a bearing application (where the beam might be impacted to refusal), vibrated H-Beams are used for soldier beams and in slurry wall construction.

(3) Caissons and Pipe Pile. Caissons are a versatile item, extensively used with drilled shafts. To drive these, a special device called a caisson beam is employed. This is a horizontal slide with a set of two clamps attached to it. The clamps affix the pile to the hammer on opposite sides of the caisson. The clamps are locked to the slide during use but can be moved along the slide to enable a caisson beam setup to drive a variety of pile. The equipment setup for caissons is duplicated with pipe pile. Generally, it is best to vibrate open-end pipes, although some closed-end installation is done. An example application of driving pipe pile is the installation of pipe piles for offshore structures such as petroleum production platforms. For some of these two caisson beams where two sets of clamps are used, the beams are being configured in an "x" arrangement.

(4) Concrete Piles. Concrete pile installation with vibratory hammers is rare in the United States but more common abroad. It is done with both prismatic (square and octagonal) and cylinder pile. As concrete pile is always displacement pile, the vibratory hammer must develop some toe impact by raising and lowering the pile during the vibration cycle, thus allowing penetration. This is generally accomplished using low-frequency vibrators with high amplitudes. An alternative to this is to use an impact-vibration hammer, which can more effectively deal with high toe resistance than can a vibratory hammer. Indeed, the need to drive concrete piles has been one of the most important factors in the development of these hammers.

(5) Wood Piles. As it is almost exclusively bearing pile, wood is rarely vibrated in the United States. Extraction of wood pile, however, is common, and the vibratory hammer is an effective tool for this purpose. The wood pile can be extracted intact in this manner. Special wood clamps are used for this purpose.

3-3. RIGS. All pile drivers require some kind of rig to lift hammer and pile and to guide the system into the ground. Although there are many different configurations used for this purpose, for impact hammers the most common is shown in figure 3-22. This is a commercial crane adapted for pile driving. There are many variations of this, depending upon the application. However, the ultimate choice of a rig configuration is very dependent upon its availability. Most of these rigs can be adapted to a floating, barge-type configuration as well.

a. Crane. After the hammer, the crane is the single most important part of a pile driving system. Its proper selection is essential for correct, safe, and economical installation of the piling.

(1) Cranes for Impact Hammers. The crane must have adequate lifting capacity to lift the hammer, leaders, driving accessories, and pile. It must have three lifting lines, one for the hammer, one for the pile, and one for the leaders (if swinging leaders are used; if not, two will suffice). This lifting capacity must be adequate to lift all of the parts of the system no matter how far the crane is boomed out to drive the piling in question. This is important because frequently piling must be driven from a spot which is relatively far from the crane cab, and thus the maximum capacity of the crane in the highest boom position is not available for the particular job. Cranes for pile driving must have superior side loading capacity, and this must be watched for, especially with hydraulic cranes.

(2) Boom Point Connection. With both underhung and fixed leaders, the boom point connection is important. Inadequate or poorly designed and manufactured boom point connections can lead to accidents and job delays. Types of boom point connections are shown in figure 3-23.

(3) Cranes for Vibratory Hammers. Vibratory hammers are generally not operated in leaders but are free hanging from the crane. However, it is important that cranes for vibratory hammers have adequate capacity to both lift the hammer and pile for driving and any possible extraction load that might be needed to pull piles out of the ground. The boom must also be able to withstand any residual vibration transmitted to it by the suspension.

b. Power Systems. All external combustion hammers and the vast majority of vibratory hammers require some kind of power source to operate the equipment. This power source depends upon the nature of the hammer.

(1) Air/Steam Hammers. Except for the largest marine rigs, most air/steam hammers today are powered by air compressors. These can be mounted either on the ground or, more commonly, on the back of the crane as a substitute for the counterweight. Boilers still used with pile hammers are either of the fire tube Scotch marine type or the vertical water tube Raymond type. Both compressors and boilers should be configured in accordance with the manufacturer's recommendations. Also, essential for air/steam hammers is a three-way shut-off valve that can blow pressure out of the lines after the hammer is stopped or in an emergency, and a line oiler of the sight feed or pump type to provide a continuous stream of atomized oil to the cylinder.

(2) Hydraulic Vibratory Hammers. For a variety of reasons, hydraulic systems have become dominant for vibratory hammers. These systems use a diesel engine to drive a hydraulic pump, which in turn drives the motor on the exciter. A reservoir is used to store hydraulic fluid, to make up fluid in case of leakage, and to assist in the cooling of the fluid. A system of valves is used to control the fluid flow, both in starting and stopping the machine as well as during operation. Beyond these basics, there are specific differences between the various hydraulic power packs available.

(a) Pump Drive and/or Gearbox. The hydraulic pump is connected to the engine through a pump drive. Sometimes this pump drive is a gearbox acting as a speed changer to optimize the pump, but in others, a direct drive is employed, eliminating gear losses.

(b) Clamp Pumps. Most units have separate pumps for the hydraulic clamps, but some integrate these into the main power source.

(c) Variation of Frequency and Force. Both of these can be varied either by using variable displacement pumps in the power pack or by simply varying the engine speed. Variable displacement pumps can have very sophisticated flow control mechanisms.

(d) Control Type. These units can employ air, electric, or manual controls for the hydraulic circuitry. Manual controls are the simplest; however, they confine the operator of the unit to the power pack's location, which, depending upon visibility and other factors, may not be the most convenient place from whence to operate the machine. Remote controls allow more flexibility for the operator but are an added expense and source of trouble for the machine.

(e) Enclosure. Some power packs have a sheet metal enclosure and some do not. The principal advantage of an enclosed power pack is protection from weather and criminal activity. Enclosures are also helpful if they provide sound deadening, although many do not. Open power packs are more economical and there is better access to the parts for service.

(f) Open- and Closed-loop Hydraulic Systems. Both appear on power packs in this application. Closed-loop systems allow for better controlled starting, running, and stopping of the machines, but have traditionally been more complicated. Open-loop systems are more adaptable for powering other equipment.

(g) Crane Hydraulic Systems. In some cases the crane hydraulic system can be employed to power a vibratory hammer, generally the smaller models. This eliminates the need for both an external power pack and diesel engine. However, all other comments on control systems and operation apply regardless of whether the crane system or an external power pack is used.

(3) Hydraulic Impact Hammers. The hydraulic systems for impact hammers are very similar to those used with a hydraulic vibratory hammer, except that there is no clamp circuit and adaptations are made to accommodate the intermittent flow characteristics of these hammers.

(4) Electric Vibratory Hammers. The exciters for these units usually employ three-phase induction motors driven at a single frequency, which has encouraged the development of many systems to vary the eccentric moment and thus the driving force. In some cases electric vibratory hammers can be driven from nearby three-phase outlets, obviating the need for a generator set. The hammer thus only requires a switch box to control it. A separate, small power pack, driven with an electric motor, is required to operate the hydraulic clamp, if there is one. This can either be on the ground or mounted on the static overweight. Electric systems are less and less popular because of maintenance and reliability considerations.

c. Leaders. Leaders are essential for properly guiding the impact hammer and pile downward during driving. Their configuration is dependent upon the application.

(1) Fixed Leaders. Fixed leaders are defined as leaders which are attached by a mechanical joint at the boom point and at the bottom of the leaders. The boom point connection should always enable the leaders to rotate about the boom point, and should be below the top of the leaders. A typical fixed leader setup is shown in figure 3-24. Advantages and disadvantages of fixed leaders are described in table 3-4. Fixed leaders are generally differentiated by their spotters; these different types are described below.

(a) Conventional. These fixed leaders use either a fixed length spotter (useful only when all of the pile on a particular job has the same batter) or a spotter with a system of holes through which pins can be run. The batter of the leaders can thus be changed by telescoping the spotter to the desired length, lining up the holes in both inner and outer tubes, and inserting the pins.

(b) Moonbeam Spotters. These are fixed in the usual way to the boom; however, a curved beam at the bottom of leaders guides the leaders in rotating about the boom point. A wheeled carriage on the leaders connects the leaders to the moonbeam. Such a leader is shown in figure 3-25. Moonbeam spotters are generally obsolete, having been replaced by the hydraulic spotters.

(c) Hydraulic Spotters. These spotters use hydraulic cylinders to control the movement of the spotters. Hydraulic cylinders are mounted to telescope the overall length in and out, and to pivot the spotter ends about their connection either with the crane cab or with the leaders, or both. These spotters are very versatile. Within their travel range they can be adjusted in an infinitely variable manner from the crane. They can be powered either by their own hydraulic power system or from the crane's own hydraulic system. An example of such a spotter is shown in figure 3-26.

(d) Vertical Travel Leaders. Vertical travel leaders are fixed leaders with hydraulic spotters which can also move the leaders up and down relative to the spotter and boom point. They are most advantageous when positioning of the leaders is exceptionally difficult, such as with railroad construction. Figure 3-27 shows an example of vertical travel leaders.

(2) Swinging Leaders. Swinging leaders are leaders that are suspended from the crane using a wire rope. These are by far the most common leaders in use. Advantages and disadvantages of swinging leaders are shown in table 3-5. A typical swinging leader is shown in figure 3-28. Swinging leaders are generally used with plumb pile; however, under certain conditions, and with the proper equipment and crane operator, they can be used on batter piles, as shown in figure 3-29. It is important when swinging leaders are used to avoid supporting weight of hanging leaders on the pile lest the pile buckle.

(3) Underhung Leaders. Underhung leaders are similar to fixed leaders, except that the boom point connection is made at the top of the leaders, and generally the leader can move only fore and aft from the crane. These leaders can be used with or without a spotter. Table 3-6 lists the strengths and weaknesses of these leaders, and typical underhung leader setups are shown in figure 3-30.

(4) Hammers without Leaders. Most impact hammers require leaders to operate; however, some hammers can be fitted with pants. Such an arrangement is shown in figure 3-31. Not all hammers can be fitted with pants; these should only be installed in accordance with the manufacturer's recommendations.

(5) Leader Accessories. Leaders are used with a wide variety of accessories. Some of these are described below.

(a) Cradle or Extension. A cradle or extension is used when the hammer is either too small for the leaders used or must be driven outside of the leaders. A typical extension is shown in figure 3-32.

(b) Pile Gate. This is used to help guide the pile into the leaders and to keep it in alignment during driving. Pile gates can be either manually or hydraulically operated. They open to admit the pile and close before driving.

(c) Stabbing Point. A stabbing point is used with swinging leaders to fix the lower end of the leaders. These are very important to assist in the stabilization of the leaders. Figure 3-33 shows stabbing points.

3-4. SPECIALIZED OPERATIONS AND EQUIPMENT. Because of the diverse situations under which pile foundations are driven, sometimes specialized operations are necessary to supplement the regular equipment used.

a. Jetting. Jetting is the use of pressurized fluid to temporarily loosen the bond between pile and soil, thus reducing the resistance of the pile to sinking into the ground. Piles may be sunk into place by jetting which may or may not be accompanied by impacts on the pile or by alternatively raising and dropping the pile. Jetting is also used to relieve driving stresses, to save time, to obtain increased penetration of piles, and to decrease vibration incident to driving piles. Piles should always be driven to their final penetration depth after jetting has ceased.

(1) Types of Jets. Various configurations of jets are shown in figure 3-34. There are basically two types of jets, fixed and movable jets.

(a) Fixed Jets. Fixed jets are jets which are a permanent part of the pile. Precast jets in concrete piles and concrete sheet piling may be used to avoid off-center and/or unsymmetrical jetting and the problem of keeping proper alignment. This type of pile is costly but may be desirable where conditions do not permit use of a movable jet.

(b) Movable Jets. These are attached to the pile to allow their removal after pile installation. Two jets symmetrically located give the most rapid penetration and best control of the pile path.

(2) Pipes. The diameter of the pipe is essential to allow the required water flow. The diameter of the pipe should be no less than 2 inches and can vary up to 4 inches. Nozzle diameters should be from 3/4 inch to 1-1/2 inches.

(3) Hose. The hose should be approximately 1 inch larger in diameter than the jet pipe but no less than 3 inches in diameter. It should have a protective jacket of canvas, cotton, or steel wire mesh. Hose length should be as short as possible to minimize friction losses.

(4) Pumps. To jet pile properly a large flow of water is required. This is accomplished by using a jet pump, whose flow should be no less than 250 gallons/minute and can range up to 1,057 gallons/minute. Water pressures should generally vary from 100 to 200 psi for most soils. However, in gravels the pressures should be set at 100 to 150 psi, and 40 to 60 psi in sands. They should be equipped with only bronze fittings. The prime mover for the pump should have adequate torque and horsepower to pressurize the water, and all fittings, hoses, and orifices should be properly sized to accommodate the flow and output the water jet at the desired pressure.

(5) Methods and Limitations of Jetting. Jetting can be performed using various methods and, as with any other techniques, has its limitations.

(a) Water Jetting. This is a method designed to discharge a water jet at the pile tip, with both volume of water and pressure sufficient to allow the discharge to come up around the pile.

(b) Spade or Multiple Jetting. This is a method for assisting the driving or sheet piling.

(c) Air Jetting. This method is practical for shallow depths and for probing or friction reduction but not for deep penetration.

(d) Combined Air and Water Jetting. This method is useful when a double water jet and heavy driving cannot secure the desired penetration.

(e) Limitations. Jetting applications are limited in clay soils where the jets may become plugged, in cohesive soils generally where jetting is not useful or practical, in fine grained and poorly grained soils where jetting may loosen the soil around the pile already driven, and in locations where there is considerable groundwater and the material disturbed by the jets cannot escape.

b. Underwater Driving. Most pile driving for coastal and river structures can be driven from the surface; however, in some cases it is advantageous to drive piling underwater. This eliminates the use of pile followers which add weight to the system. Although some air/steam hammers have been and are used for underwater driving, the best type of hammer to use underwater is the hydraulic hammer. Such a hydraulic hammer is shown in figure 3-35. The hydraulic power pack is generally on the barge deck and the hoses extend into the water. In some cases, large hose reels are used to store the hoses. With the largest hammers, the power pack is mounted on the hammer and driven with an electric motor, the cables extending to the generator on the deck. Hydraulic vibratory hammers can also be used underwater. The technique for using such a hammer is shown in figure 3-36.

c. Preboring. Preboring consists of drilling, auguring, or coring a hole in the ground and filling the hole with concrete or driving a pile into the hole. This is generally done with a continuous flight auger. Filling the hole with concrete is properly a drilled shaft, and is beyond the scope of this manual. For driven piles, preboring is advantageous when the ground resistance is extremely high. For square concrete piles, the diameter of the bored shaft should be approximately 125 percent of the nominal pile size. Although preboring will generally reduce the driving resistance, it does so at the expense of shaft resistance, which decreases during the preboring. This diminution of the pile capacity must be taken into account when determining whether a pile can be prebored.

d. Screwing. Screw piles consist of a pile casing fitted with one or more turns of helical screw having a larger diameter than the pile. Installation is made by screwing the casing into the ground to a predetermined level. Torque is provided by a capstan or similar device.

e. Pull Down. This is a type of pile jacking where the pile casing is jacked into place and filled with concrete. Where a closed-end casing is used, special equipment is limited to conventional screw or hydraulic jacks. Where an open-end casing is used, a jet or miniature orange peel bucket is used for removing the core. Figure 3-37 shows such a bucket; it can also be used for casings installed by vibration.

f. Concrete Pile Cutting. When the driving of a concrete pile is complete, the next step is to connect the top of the pile with the structure it is holding up. To do this, it is frequently necessary to have the reinforcing bar protrude above the top of the pile. Two basic ways to accomplish this are (1) to fabricate the pile with protruding reinforcing bar or cable, or (2) to cut off the top of the pile in such a way as to leave the reinforcing bar or cable exposed for connection. Assuming that the reinforcing bars are not damaged, piles made by method (1) are ready for connection upon driving. For method (2), it is necessary to cut the concrete pile, and if the reinforcing bars are needed, it is necessary to leave them protruding and undamaged.

(1) Manual Methods. Manual methods of concrete pile cutting involve the use of tools adapted for the task. In this case, concrete crushing is performed by jackhammers. Reinforcement bars are then cut by electric or resistance welding, gas, or flame cutting. Concrete pile saws are also used manually; these are rotary saws which simply cut the pile.

(2) Automatic Methods. These involve the use of hydraulic devices called pile beavers, which crush the pile by effort applied in a transverse direction perpendicular to the pile axis. As a result of this effort, the reinforcement bars were exposed and then cut by different ways at a predetermined level. Others combine the bond separation and the pile cracking in the same area of the pile, The cracking taking place both in the same plane as the bond failure and also above and below it. A pile beaver is shown in figure 3-38.

Table 3-1. Strengths and Weaknesses of Impact Hammer Types.

Hammer Type	Description	Advantages	Disadvantages
		External Combustion Hammers	
Drop Hammer	Hammer is raised by a rope running over the top of a framework and extending back to a drum or geared shaft; blow is delivered by the fall of the hammer under influence of gravity.	Allows greater variation in both weight and speed of blows.	Very low frequency of blows.
		Low initial cost and relatively long service life.	Efficiency reduced due to drag of rope and drum.
		Simple to operate in remote locations where other equipment is not obtainable.	Cannot be inverted and used as a pile extractor
			Cannot be used in locations where headroom is limited.
			Not readily adaptable for driving batter piles.
Single-Acting Air/Steam Hammers	Steam or air raises the movable mass of the hammer, which drops by gravity.	Good performance. Simple in design and dependable in service.	Relatively low (50-60 blows per minute) blow rate.
		Usable in all soil conditions, but particularly effective in penetrating heavy clays.	Cannot be used as an extractor.
Double-Acting Air/Steam Hammers	Steam or air raises the striking part and also impacts additional energy during downstroke.	High frequency (90-150 blows per minute) of blows keeps pile moving and speeds penetration.	Relatively high impact velocity results in pile head deformation of low compressive strength piles.
		Can be used in horizontal position.	Large compressor or boiler required for operation.
		Works best in sandy soil, but can be used in any soil.	Rebound effects make hammer energy output variable
		Can be inverted and used as a pile extractor.	
		Enclosed ram permits underwater driving.	

Table 3-1 (continued)

Hammer Type	Description	Advantages	Disadvantages
		External Combustion Hammers (Continued)	
Differential Acting Air/ Steam Hammers	Variation of double-acting hammer with different valve arrangement.	Frequency of blows approaches that of double acting hammer while the effective stroke is the same as for single-acting hammers. Can be used in horizontal position. Works best in sandy soil, but can be used in any soil.	Rebound effects make hammer energy output variable
Hydraulic Impact Hammers	Use hydraulic fluid to raise the ram; some units have assisted fall as well.	Efficient both in power pack energy conversion and generally with impact force transfer. More difficult to maintain than other impact hammers. May be used underwater. Some units have sound attenuation features.	Expensive to rent or purchase.
		Internal Combustion Hammers	
Single-Acting Diesel Hammers	Self-contained unit which uses ignition of fuel to impart additional energy during downstroke to drive pile upward.	Independent of outside power sources (boiler, compressor, hydraulic power pack, Etc.) Light weight and easily portable. Low operating cost. Ease of operation in cold weather.	Cannot be inverted and used as pile extractor. In soft driving, may stall due to inadequate rebound. Long stroke of ram may cause tension cracking in concrete piles. On hammers with atomized fuel system, fuel system is complicated. Low-frequency (40-60 blows per minute) blow rate.

Table 3-1 (concluded)

Hammer Type	Description	Advantages	Disadvantages
External Combustion Hammers (Continued)			
Double-Acting Diesel Hammers	Similar to single-acting diesels, except that trapped air stores and releases energy during operation.	High-frequency (80 blows per minute) blow rate. or vacuum in top of hammer No external power source required.	Relatively heavy hammer weight relative to single-acting diesels. Atomized fuel system can be difficult to service.

Table 3-2a. Instruction for Use of Hammer Tables.

The tables that follow are a summary of the impact and vibratory hammer that are presently manufactured and distributed in the United States. The table is intended to be a first reference to give an idea of what hammers are available.

There are two tables; table 3-2b is in metric units, and table 3-2c is in English units. Both tables cover the same set of hammers. Each table in turn is divided into two parts; impact hammers and vibratory hammers. All of the impact hammers are sorted according to maximum rated striking energy and the vibratory hammers according to maximum dynamic force. In both cases, with both types of hammers, it is possible to reduce the output of the hammer from its maximum by reducing the stroke and/or fluid pressure (with impact hammers) or by reducing the rotational speed of the eccentrics (with vibratory hammers.) This is important when a smaller hammer for the job is required but not available.

Impact hammers have a wide variety of types and operating principles, most of which are discussed in this chapter. In the table under "Description" there are codes listed for each hammer which categorize the hammer\; these are as follows:

A/S	Air/Steam
Air	Air Only Hammer
Asst.	Assisted Fall, Hydraulic Impact Hammer
Closed	Closed-end, Double-acting Diesel Hammer
Compound	Compound Hammer (this is a hammer where the air or steam is used expansively to help move the ram on the downstroke)
Diff.	Differential Acting Hammer
D/A	Double-acting Hammer
ECH	External Combustion Hammer
Free	Free Fall, Single-acting, Hydraulic Impact Hammer
Hyd.	Hydraulic Impact Hammer
ICH	Internal Combustion Hammer
Open	Open-end, Single-acting Diesel Hammer
S/A	Single Acting
Steam	Steam Only Hammer

The data given in these charts were compiled by Pile Buck, Inc., Jupiter, FL, from their publication "Pile Hammer Specifications", and is reproduced by permission. It is essential for the user of this chart to know that the data shown are compiled from manufacturers' own data. Definitions of several of the parameters vary from one manufacturer to the next. So direct comparison of the parameters (and thus the hammers) may be difficult. Also, new models are constantly being introduced and existing models are being updated. These specifications may be subject to change without notice. When considering the use of a particular hammer for an application, it is important to obtain the latest information concerning it from both the manufacturer and any other available source. This chart thus should only be used as a preliminary guide to the most important specifications of the hammers shown.

Table 3-2b. Hammer Chart, Metric Units.

Maximum Striking Energy, kJ	Model	Manufacturer	Description	BPM	Ram kg	Hammer Mass, kg	Length Mass, cm
			Impact Hammers				
3,000.0	S-3000	IHC	ECH, Hyd., Asst.	35	150,000	469,388	1,676
2,440.4	6300	Vulcan	ECH, A/S, S/A	42	136,054	260,771	914
2,145.2	MRBS 12500	Menck	ECH, A/S, S/A	36	125,000	244,957	1,090
2,000.0	S-2000	IHC	ECH, Hyd., Asst.	40	100,000	316,100	1,397
1,627.0	2000E6	Conmaco	ECH, A/S, S/A	40	90,703	222,222	1,082
1,627.0	6200	Vulcan	ECH, A/S, S/A	36	90,703	198,738	916
1,600.0	S-1600	IHC	ECH, Hyd., Asst.	40	80,000	263,039	1,270
1,423.6	1750E6	Conmaco	ECH, A/S, S/A	40	79,365	210,884	1,090
1,220.2	6150	Vulcan	ECH, A/S, S/A	41	68,027	124,717	1,082
1,176.8	MRBS 8000	Menck	ECH, A/S, S/A	38	80,000	149,973	940
1,016.9	1500E5	Conmaco	ECH, A/S, S/A	42	68,027	128,345	930
1,016.9	5150	Vulcan	ECH, A/S, S/A	46	68,027	124,717	801
1,016.9	D300	Delmag	ICH, Open	36-50	35,000	74,986	1,062
1,000.0	S-1000	IHC	ECH, Hyd., Asst.	45	50,000	146,939	940
800.0	S-800	IHC	ECH, Hyd., Asst.	45	40,000	120,816	932
691.5	850E6	Conmaco	ECH, A/S, S/A	40	38,549	78,730	767
677.9	5100	Vulcan	ECH, A/S, S/A	48	45,351	89,342	833
676.6	MRBS 4600	Menck	ECH, A/S, S/A	42	45,991	79,986	836
500.0	S-500	IHC	ECH, Hyd., Asst.	45	25,000	73,469	1,016
478.1	HH40	BSP	ECH, Hyd., Free	20	40,000	64,071	1,001
474.5	700E5	Conmaco	ECH, A/S, S/A	43	31,746	68,934	706
441.3	MRBS 3000	Menck	ECH, A/S, S/A	42	30,000	48,991	762
423.7	560	Vulcan	ECH, A/S, S/A	47	28,345	60,798	701
406.7	3100	Vulcan	ECH, A/S, S/A	60	45,351	88,662	709
406.7	D100-13	Delmag	ICH, Open	34-45	10,708	20,570	620
400.0	S-400	IHC	ECH, Hyd., Asst.	45	20,000	67,120	940
379.6	K150	Kobe	ICH, Open	45-60	15,011	36,508	904
358.6	HH30	BSP	ECH, Hyd., Free	28	30,000	48,978	935
305.1	450E5	Conmaco	ECH, A/S, S/A	45	20,408	46,712	709
305.1	D80-23	Delmag	ICH, Open	36-45	8,844	17,115	620
277.3	540	Vulcan	ECH, A/S, S/A	48	18,549	46,703	688
257.4	MRBS 1800	Menck	ECH, A/S, S/A	44	17,497	29,293	683
250.0	S-250	IHC	ECH, Hyd., Asst.	45	12,500	40,272	965
244.0	360	Vulcan	ECH, A/S, S/A	62	27,211	56,612	579
237.3	535	Vulcan	ECH, A/S, S/A	37	15,873	27,664	681
235.2	HH20	BSP	ECH, Hyd., Free	36	20,000	31,986	681
223.7	D62-22	Delmag	ICH, Open	36-50	6,621	12,270	594
203.4	60X	Raymond	ECH, A/S, S/A	60	27,211	38,549	688
203.4	300E5	Conmaco	ECH, A/S, S/A	40	13,605	26,485	635
203.4	530	Vulcan	ECH, A/S, S/A	42	13,605	26,159	622
202.8	MH80B	Mitsubishi	ICH, Open	42-60	8,000	19,773	594
200.0	S-200	IHC	ECH, Hyd., Asst.	45	10,000	34,921	889
191.2	MB70	Mitsubishi	ICH, Open	38-60	7,184	20,862	594
188.2	HH16	BSP	ECH, Hyd., Free	30	15,993	22,989	770
172.9	DE150/110	MKT	ICH, Open	40-50	6,803	13,379	605
164.7	HH14	BSP	ECH, Hyd., Free	35	13,998	20,490	716
162.7	340	Vulcan	ECH, A/S, S/A	60	18,141	44,526	566
158.6	D55	Delmag	ICH, Open	36-47	5,488	11,927	541
145.3	D46-32	Delmag	ICH, Open	37-53	4,600	8,890	528
143.4	B-5505	Berminghammer	ICH, Open	36-60	4,172	10,884	671
143.2	K60	Kobe	ICH, Open	42-60	5,986	17,007	739
135.6	40X	Raymond	ECH, A/S, S/A	64	18,141	28,118	582
135.6	200E5	Conmaco	ECH, A/S, S/A	46	9,070	21,769	582
135.6	520	Vulcan	ECH, A/S, S/A	42	9,070	21,624	622
135.6	200S	ICE	ICH, Open	53-70	9,070	15,238	518
129.4	HH11	BSP	ECH, Hyd., Free	38	10,998	17,293	650
126.8	DE-150/110	MKT	ICH, Open	40-50	4,989	11,134	544
126.6	MRBS 850	Menck	ECH, A/S, S/A	45	8,599	12,649	599
125.8	KC45	Kobe	ICH, Open	39-60	4,499	11,202	546
123.5	K45	Kobe	ICH, Open	39-60	4,490	11,610	564

(Continued)

(Sheet 1 of 6)

Table 3-2b. (continued)

Maximum Striking Energy, kJ	Model	Manufacturer	Description	BPM	Ram kg	Hammer Mass, kg	Length Mass, cm
			Impact Hammers (Continued)				
122.0	300	Conmaco	ECH, A/S, S/A	52	13,605	25,120	513
122.0	030	Vulcan	ECH, A/S, S/A	54	13,605	24,249	498
122.0	90S	ICE	ICH, Open	38-55	4,082	7,619	526
118.5	B-5005	Berminghammer	ICH, Open	36-60	3,447	10,159	671
118.0	D44	Delmag	ICH, Open	37-56	4,308	10,113	483
115.8	MH45	Mitsubishi	ICH, Open	42-60	4,762	11,156	546
113.9	M43	Mitsubishi	ICH, Open	40-60	4,300	10,277	495
113.7	D36-32	Delmag	ICH, Open	36-53	3,600	7,890	528
110.2	8/0	Raymond	ECH, A/S, S/A	40	11,338	15,420	589
107.8	J44	IHI	ICH, Open	42-70	4,400	9,751	452
107.1	K42	Kobe	ICH, Open	40-60	4,200	10,884	538
105.9	HH9	BSP	ECH, Hyd., Free	30	9,000	11,247	640
102.9	B-4505	Berminghammer	ICH, Open	36-60	2,993	7,256	566
101.7	30X	Raymond	ECH, A/S, S/A	70	13,605	23,583	582
99.0	3400	FEC	ICH, Open	40-60	3,400	6,621	488
97.9	KC35	Kobe	ICH, Open	39-60	3,500	7,891	513
96.0	K35	Kobe	ICH, Open	39-60	3,500	8,481	538
94.9	1070	ICE	ICH, Closed	64-68	4,535	9,751	544
94.9	DE70/50C	MKT	ICH, Open	40-50	3,175	6,667	546
94.9	70S	ICE	ICH, Open	38-55	3,175	6,395	508
94.8	D30-32	Delmag	ICH, Open	36-52	3,000	6,010	526
90.0	S-90	IHC	ECH, Hyd., Asst.	50	4,500	14,240	787
88.9	MH35	Mitsubishi	ICH, Open	42-60	3,501	8,390	526
87.6	M33	Mitsubishi	ICH, Open	40-60	3,293	7,683	401
86.1	J35	IHI	ICH, Open	42-70	3,506	7,664	442
85.4	3000	FEC	ICH, Open	40-60	32,000	5,986	472
84.7	125E5	Conmaco	ECH, A/S, S/A	41	5,669	9,977	549
82.3	HH7	BSP	ECH, Hyd., Free	38	7,000	9,247	640
81.5	K32	Kobe	ICH, Open	40-60	3,197	8,050	538
81.3	200	Conmaco	ECH, A/S, S/A	55	9,070	20,209	457
81.3	020	Vulcan	ECH, A/S, S/A	59	9,070	18,898	447
81.3	S-20	MKT	ECH, A/S, S/A	60	9,070	17,528	470
81.3	512	Vulcan	ECH, A/S, S/A	41	5,442	10,649	561
79.0	D25-32	Delmag	ICH, Open	37-52	2,500	5,510	526
78.5	V25	Vulcan Series I	ICH, Open	-	2,500	5,499	498
78.0	115E5	Conmaco	ECH, A/S, S/A	42	5,215	9,524	541
78.0	B-4005	Berminghammer	ICH, Open	36-60	2,268	6,531	566
77.1	5/0	Raymond	ECH, A/S, S/A	44	7,937	11,995	511
73.6	D30	Delmag	ICH, Open	39-60	2,993	5,578	434
72.9	B-400	Berminghammer	ICH, Open	37-60	2,268	6,803	452
69.8	KC25	Kobe	ICH, Open	39-60	2,499	5,501	432
70.0	S-70	IHC	ECH, Hyd., Asst.	50	3,500	12,472	686
68.7	K25	Kobe	ICH, Open	39-60	2,499	5,941	533
67.8	200C	Vulcan	ECH, A/S, Diff.	95	9,070	17,687	424
67.8	510	Vulcan	ECH, A/S, S/A	41	4,535	9,741	561
67.8	100E5	Conmaco	ECH, A/S, S/A	47	4,535	8,844	541
67.8	660	ICE	ICH, Closed	84-88	3,430	11,102	528
67.8	2500	FEC	ICH, Open	40-60	2,500	5,488	472
67.8	DE70/50C	MKT	ICH, Open	40-50	2,268	5,760	546
66.1	160	Conmaco	ECH, A/S, S/A	50	7,370	15,057	422
66.1	016	Vulcan	ECH, A/S, S/A	58	7,370	13,719	424
66.1	150C	Raymond	ECH, A/S, Diff.	95-105	6,803	14,739	480
66.1	4/0	Raymond	ECH, A/S, S/A	46	6,803	10,794	490
65.8	D22-33	Delmag	ICH, Open	38-52	2,200	5,170	523
63.6	MH25	Mitsubishi	ICH, Open	42-60	2,500	5,986	508
61.3	MRBS 500	Menck	ECH, A/S, S/A	48	5,000	6,898	508
61.0	M23	Mitsubishi	ICH, Open	42-60	2,300	5,088	429
60.7	DE50C	BSP	ICH, Open	42-54	2,259	4,671	437
59.7	MS-500	MKT	ECH, A/S, S/A	40-50	4,989	7,029	460

(Continued)

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Maximum Striking Energy, kJ	Model	Manufacturer	Description	BPM	Ram kg	Hammer Mass, kg	Length Mass, cm
			Impact Hammers (Continued)				
59.7	B-3505	Berminghammer	ICH, Open	36-60	1,814	5,442	551
58.8	HH5	BSP	ECH, Hyd., Free	40	4,998	7,247	582
58.0	D19-32	Delmag	ICH, Open	37-53	1,900	3,537	472
57.6	DA-55C	MKT	ICH, Open	40-50	2,268	7,710	528
56.9	140	Conmaco	ECH, A/S, S/A	55	6,349	13,946	422
56.9	014	Vulcan	ECH, A/S, S/A	59	6,349	12,472	417
56.9	42S	ICE	ICH, Open	37-55	1,854	3,451	490
56.0	K22	Kobe	ICH, Open	40-60	2,200	5,601	533
55.0	3/0	Raymond	ECH, A/S, S/A	50	5,669	9,524	475
54.6	B-300	Berminghammer	ICH, Open	37-60	1,701	4,317	427
54.5	D16-32	Delmag	ICH, Open	36-52	1,600	3,350	472
54.2	508	Vulcan	ECH, A/S, S/A	41	3,628	8,834	561
54.2	80E5	Conmaco	ECH, A/S, S/A	47	3,628	7,937	541
54.2	640	ICE	ICH, Closed	74-77	2,721	6,558	475
54.2	40S	ICE	ICH, Open	38-55	1,814	3,401	478
54.2	DE33/30/20C	MKT	ICH, Open	40-50	1,497	3,946	485
53.8	D22	Delmag	ICH, Open	42-60	2,200	5,079	432
53.0	J22	IHI	ICH, Open	42-70	2,200	4,898	427
51.8	DA-55C	MKT	ICH, Closed	78-82	2,268	7,710	528
50.8	S-14	MKT	ECH, A/S, S/A	60	6,349	14,376	414
50.7	115	Conmaco	ECH, A/S, S/A	52	5,215	9,447	432
48.8	140C	Vulcan	ECH, A/S, Diff.	101	6,349	12,691	373
46.8	B-2505	Berminghammer	ICH, Open	36-60	1,361	4,989	549
46.1	DA-45C	MKT	ICH, Open	40-50	1,814	6,440	460
44.7	DE33/30/20C	MKT	ICH, Open	40-50	1,497	3,946	485
44.6	100C	Vulcan	ECH, A/S, Diff.	103	4,535	10,068	427
44.1	S-10	MKT	ECH, A/S, S/A	55	4,535	10,150	429
44.1	100	Conmaco	ECH, A/S, S/A	55	4,535	8,744	432
44.1	010	Vulcan	ECH, A/S, S/A	50	4,535	8,517	457
44.1	2/0	Raymond	ECH, A/S, S/A	50	4,535	8,413	457
44.1	506	Vulcan	ECH, A/S, S/A	46	2,948	5,907	531
44.1	65E5	Conmaco	ECH, A/S, S/A	50	2,948	5,669	513
42.5	D12-32	Delmag	ICH, Open	36-52	1,279	2,839	472
41.8	MS-350	MKT	ECH, A/S, S/A	40-50	3,499	4,762	460
41.6	DA-45C	MKT	ICH, Closed	78-82	1,814	6,440	460
40.7	520-30	ICE	ICH, Closed	80-84	2,299	6,077	411
39.7	B-225	Berminghammer	ICH, Open	39-60	1,361	3,959	427
38.1	MH15	Mitsubishi	ICH, Open	42-60	1,501	3,810	490
38.0	DE33/30/20C	MKT	ICH, Open	40-50	1,270	3,719	485
36.7	1500	FEC	ICH, Open	40-60	1,497	3,277	432
36.7	D15	Delmag	ICH, Open	42-60	1,497	3,000	424
36.6	DE30C	BSP	ICH, Open	42-54	1,361	3,447	432
35.7	520-26	ICE	ICH, Closed	80-84	2,299	5,689	411
35.3	HH3	BSP	ECH, Hyd., Free	46	2,998	5,247	521
35.3	85C	Vulcan	ECH, A/S, Diff.	111	3,866	8,626	384
35.3	S-8	MKT	ECH, A/S, S/A	55	3,628	8,299	437
35.3	80	Conmaco	ECH, A/S, S/A	56	3,628	7,837	432
35.3	08	Vulcan	ECH, A/S, S/A	50	3,628	7,596	452
35.3	M14S	Mitsubishi	ICH, Open	42-60	1,347	3,293	411
35.0	S-35	IHC	ECH, Hyd., Asst.	60	3,300	10,522	559
33.9	505	Vulcan	ECH, A/S, S/A	46	2,268	5,351	531
33.9	50E5	Conmaco	ECH, A/S, S/A	48	2,268	4,989	513
33.1	80C	Vulcan	ECH, A/S, Diff.	109	3,628	8,111	384
33.1	80C	Raymond	ECH, A/S, Diff.	95-105	3,628	8,111	371
33.1	80CH	Raymond	ECH, Hyd., Asst.	110-120	3,628	8,063	363
33.1	K13	Kobe	ICH, Open	40-60	1,300	3,311	508
33.0	0	Vulcan	ECH, A/S, S/A	50	3,401	7,370	457
33.0	0	Raymond	ECH, A/S, S/A	50	3,401	7,256	457
32.5	C-826	MKT	ECH, A/S, Compound	85-95	3,628	8,050	371

(Continued)

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Table 3-2b. (continued)

Maximum Striking Energy, kJ	Model	Manufacturer	Description	BPM	Ram kg	Hammer Mass, kg	Length Mass, cm
			Impact Hammers (Continued)				
32.3	DA-35C	MKT	ICH, Open	40-50	1,270	4,898	518
31.2	B-23	Berminghammer	ICH, Closed	80	1,270	4,508	488
30.5	422	ICE	ICH, Closed	76-82	1,814	4,422	424
30.5	1200	FEC	ICH, Open	40-60	1,250	2,966	427
30.5	D12	Delmag	ICH, Open	42-60	1,250	2,744	424
30.5	30S	ICE	ICH, Open	44-67	1,361	2,834	376
28.5	DA-35C	MKT	ICH, Closed	40-50	1,270	4,898	518
27.1	DE33/30/20C	MKT	ICH, Open	40-50	907	3,356	488
26.4	65C	Raymond	ECH, A/S, Diff.	110	2,948	6,655	356
26.4	65CH	Raymond	ECH, Hyd., Asst.	128-136	2,948	6,628	366
26.4	1-S	Raymond	ECH, A/S, S/A	58	2,948	5,669	389
26.4	65	Conmaco	ECH, A/S, S/A	61	2,948	5,488	391
26.4	106	Vulcan	ECH, A/S, S/A	60	2,948	5,079	396
26.4	06	Vulcan	ECH, A/S, S/A	60	2,948	5,079	396
26.0	65C	Vulcan	ECH, A/S, Diff.	117	2,948	6,751	368
26.0	11B3	MKT	ECH, A/S, D/A	95	2,268	6,349	312
24.5	440	ICE	ICH, Closed	88-92	1,814	4,463	411
24.4	312	ICE	ICH, Closed	100-105	1,749	4,705	328
24.4	B-200	Berminghammer	ICH, Open	39-58	907	3,147	419
24.4	D8-22	Delmag	ICH, Open	38-52	799	1,914	470
23.5	HPH 2400	Dawson	ECH, Hyd.	140	1,900	5,999	528
22.0	S-5	MKT	ECH, A/S, S/A	60	2,268	5,651	404
21.7	C-5	MKT	ECH, Steam, Compound	100-110	2,268	5,351	267
20.6	HH2	BSP	ECH, Hyd., Asst.	55	1,999	4,098	310
20.5	50C	Vulcan	ECH, A/S, Diff.	117	2,268	5,343	335
20.3	1	Vulcan	ECH, A/S, S/A	60	2,268	4,399	389
20.3	50	Conmaco	ECH, A/S, S/A	64	2,268	4,807	391
19.3	C-5	MKT	ECH, Air, Compound	100-110	2,268	5,388	267
17.8	10B3	MKT	ECH, A/S, D/A	105	1,361	4,921	279
14.2	D6-32	Delmag	ICH, Open	39-52	600	1,728	381
12.7	DA-15C	MKT	ICH, Open	40-50	499	2,188	424
12.3	D5	Delmag	ICH, Open	42-60	499	1,238	381
11.9	DE10	MKT	ICH, Open	40-50	499	1,406	371
11.9	9B3	MKT	ECH, A/S, D/A	145	726	3,175	254
11.9	900	BSP	ECH, A/S, D/A	145	726	3,220	239
11.8	HPH 1200	Dawson	ECH, Hyd.	120	1,043	3,000	467
11.1	DA-15C	MKT	ICH, Closed	86-92	499	2,188	427
11.0	180	ICE	ICH, Closed	90-95	782	2,107	343
9.8	30C	Vulcan	ECH, A/S, Diff.	133	1,361	3,191	272
9.8	2	Vulcan	ECH, A/S, S/A	70	1,361	3,039	353
6.4	700N	BSP	ECH, A/S, D/A	225	385	3,007	226
5.6	7	MKT	ECH, A/S, D/A	225	363	2,268	185
5.4	DGH-900	Vulcan	ECH, A/S, Diff.	328	408	2,268	206
4.9	D4	Delmag	ICH, Open	50-60	379	617	236
4.1	600N	BSP	ECH, A/S, D/A	250	227	2,177	218
3.4	6	MKT	ECH, A/S, D/A	275	181	1,315	160
2.5	D2	Delmag	ICH, Open	60-70	220	359	206
1.6	500N	BSP	ECH, A/S, D/A	330	91	1,143	180
1.4	5	MKT	ECH, A/S, D/A	300	91	680	140
0.5	DGH-100D	Vulcan	ECH, A/S, Diff.	303	45	356	127
0.5	300	BSP	ECH, A/S, D/A	400	31	306	147
-	3	MKT	ECH, A/S, D/A	400	31	306	135
0.2	200	BSP	ECH, A/S, D/A	500	22	156	84
-	2	MKT	ECH, A/S, D/A	500	22	156	74
-	1	MKT	ECH, A/S, D/A	500	10	66	99

(Continued)

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Table 3-2b. (continued)

Maximum Dynamic Force, kN	Model	Manufacturer	Frequency, Hz	Eccentric Moment, kg-m	Max. Extraction Full, kN	Suspended Mass, kg	Length cm
			Vibratory Hammers				
3,469	V-140	MKT	23.3	161.3	1,334	19,048	442
3,469	400	APE	23.3	161.3	1,779	12,381	325
2,651	300	APE	25.0	115.2	1,334	8,617	244
2,553	HBV260.01	Delmag/Tunkers	26.7	91.4	979	9,977	450
2,171	110H1	PTC	22.5	110.0	1,174	17,710	307
2,171	110H2	PTC	22.5	110.0	783	14,921	389
1,975	1412B	ICE	20.8	115.2	1,334	12,925	442
1,940	600	HPSI	26.7	69.1	667	7,483	295
1,779	520	HPSI	26.7	59.9	534	7,483	297
1,770	60H1	PTC	27.5	60.0	783	13,243	396
1,619	V-36	MKT	26.7	57.6	712	8,526	399
1,601	180	APE	28.3	50.7	890	4,535	229
1,487	4600A	Vulcan	26.7	53.0	712	6,349	333
1,487	4600	Vulcan	26.7	53.0	587	7,256	244
1,477	50H3	PTC	27.5	50.0	391	10,567	427
1,459	1412A	ICE	20.0	92.2	890	12,200	366
1,454	450	HPSI	26.7	51.8	667	6,168	282
1,441	23HF1	PTC	40.0	23.0	391	5,828	300
1,423	V-30	MKT	26.7	50.7	712	6,803	338
1,423	815	ICE	26.7	50.7	445	7,075	310
1,343	423	ICE	36.7	25.3	356	4,490	244
1,317	150	APE	30.0	36.9	890	4,308	229
1,317	23HF1	PTC	38.3	23.0	391	5,828	300
1,290	400	HPSI	26.7	46.1	667	6,168	282
1,272	HBV130.01	Delmag/Tunkers	26.7	46.0	979	5,714	404
1,192	4150	Foster	25.0	48.0	489	7,481	198
1,032	V-20	MKT	27.5	34.6	534	5,669	371
979	HVB100.01	Delmag/Tunkers	26.7	34.0	489	4,798	427
890	260	HPSI	26.7	30.0	534	4,875	254
890	2800	Vulcan	40.0	13.8	222	3,764	198
801	612	ICE	20.0	50.7	356	5,760	284
747	13HF1	PTC	38.3	13.0	294	3,243	208
744	2300A	Vulcan	26.7	26.5	427	3,878	293
744	2300	Vulcan	26.7	26.5	294	3,719	236
738	25H2	PTC	27.5	25.0	391	4,240	272
738	223	ICE	38.3	12.7	356	2,426	170
712	V-17	MKT	26.7	25.3	534	5,442	305
712	416L	ICE	26.7	25.3	356	4,490	244
694	HVB70.01	Delmag/Tunkers	27.2	23.8	489	3,197	310
694	V-16	MKT	29.2	20.7	356	4,195	340
614	25H1	PTC	25.0	25.0	391	4,240	272
587	HVB60.05	Delmag/Tunkers	25.0	23.8	245	3,197	160
578	1800	Foster	26.7	20.7	267	4,989	203
525	50	APE	30.0	15.0	356	2,177	155
512	416	ICE	25.0	20.7	356	5,941	267
-	H-1700	H&M	20.8	0.0	267	3,175	244
489	HVB60.05	Delmag/Tunkers	16.7	44.7	245	3,197	160
445	130	HPSI	26.7	15.0	267	3,197	198
445	V-5B	MKT	30.0	12.7	267	3,265	234
437	1400	Vulcan	40.0	6.9	222	1,973	264
409	V-14	MKT	25.0	17.3	356	4,535	338
391	HVB40.01	Delmag/Tunkers	25.0	15.5	249	2,585	295
391	13H1	PTC	28.3	12.5	196	2,449	196
374	7HF1	PTC	38.3	6.5	151	1,723	175
372	1150A	Vulcan	26.7	13.2	285	2,857	241
372	1150	Vulcan	26.7	13.2	294	2,948	198
356	216	ICE	26.7	12.7	356	2,426	198
329	1000	Foster	26.7	11.5	178	2,764	180
294	HVB30	Delmag/Tunkers	30.0	8.2	116	952	102
267	V-5	MKT	24.2	11.5	267	2,358	224

(Continued)

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Table 3-2b. (concluded)

Maximum Dynamic Force, kN	Model	Manufacturer	Frequency, Hz	Eccentric Moment, kg-m	Max. Extraction Full, kN	Suspended Mass, kg	Length cm
			Vibratory Hammers (Continued)				
-	H-75B	H&M	31.7	0.0	133	1,814	170
-	30	APE	41.7	0.6	44	79	46
240	HVB24	Delmag/Tunkers	35.0	5.0	116	952	99
211	65	HPSI	26.7	7.5	222	1,406	150
205	6H1	PTC	30.0	0.0	196	1,442	124
160	V-2A	MKT	30.0	4.6	71	1,088	112
151	HVB16	Delmag/Tunkers	35.0	3.2	116	907	99
151	400	Vulcan	40.0	2.3	89	499	122
151	400A	Vulcan	40.0	2.3	89	499	122
107	HVB10	Delmag/Tunkers	35.0	2.2	116	907	99

(Sheet 6 of 6)

Table 3-2c. Hammer Chart, English Units.

Maximum Striking Energy, ft-lb	Model	Manufacturer	Description	BPM	Ram Weight, lb	Hammer Weight, lb	Length in.
			Impact Hammers				
2,210,000	S-3000	IHC	ECH, Hyd., Asst.	35	332,000	1,035,000	660
1,800,000	6300	Vulcan	ECH, A/S, S/A	42	300,000	575,000	360
1,582,220	MRBS 12500	Menck	ECH, A/S, S/A	36	275,580	540,130	429
1,460,000	S-2000	IHC	ECH, Hyd., Asst.	40	221,000	697,000	550
1,200,000	2000E6	Conmaco	ECH, A/S, S/A	40	200,000	490,000	426
1,200,000	6200	Vulcan	ECH, A/S, S/A	36	200,000	438,218	361
1,160,000	S-1600	IHC	ECH, Hyd., Asst.	40	117,600	580,000	500
1,050,000	1750E6	Conmaco	ECH, A/S, S/A	40	175,000	465,000	426
900,000	6150	Vulcan	ECH, A/S, S/A	41	150,000	275,000	423
867,960	MRBS 8000	Menck	ECH, A/S, S/A	38	176,370	330,690	370
750,000	1500E5	Conmaco	ECH, A/S, S/A	42	150,000	283,000	366
750,000	5150	Vulcan	ECH, A/S, S/A	46	150,000	275,000	316
750,000	D300	Delmag	ICH, Open	36-50	77,161	165,345	418
730,000	S-1000	IHC	ECH, Hyd., Asst.	45	110,500	324,000	370
580,000	S-800	IHC	ECH, Hyd., Asst.	45	88,000	266,400	367
510,000	850E6	Conmaco	ECH, A/S, S/A	40	85,000	173,600	302
500,000	5100	Vulcan	ECH, A/S, S/A	48	100,000	197,000	328
499,070	MRBS 4600	Menck	ECH, A/S, S/A	42	101,410	176,370	329
365,000	S-500	IHC	ECH, Hyd., Asst.	45	55,300	162,000	400
352,640	HH40	BSP	ECH, Hyd., Free	20	88,160	141,276	394
350,000	700E5	Conmaco	ECH, A/S, S/A	43	70,000	152,000	278
325,480	MRBS 3000	Menck	ECH, A/S, S/A	42	66,135	108,025	300
312,500	560	Vulcan	ECH, A/S, S/A	47	62,500	134,060	276
300,000	3100	Vulcan	ECH, A/S, S/A	60	100,000	195,500	279
300,000	D100-13	Delmag	ICH, Open	34-45	23,612	45,357	244
290,000	S-400	IHC	ECH, Hyd., Asst.	45	44,000	148,000	370
280,000	K150	Kobe	ICH, Open	45-60	33,100	80,500	356
264,480	HH30	BSP	ECH, Hyd., Free	28	66,120	107,996	368
225,000	450E5	Conmaco	ECH, A/S, S/A	45	45,000	103,000	279
225,000	D80-23	Delmag	ICH, Open	36-45	19,500	37,739	244
204,500	540	Vulcan	ECH, A/S, S/A	48	40,900	102,980	271
189,850	MRBS 1800	Menck	ECH, A/S, S/A	44	38,580	64,590	269
182,500	S-250	IHC	ECH, Hyd., Asst.	45	27,600	88,800	380
180,000	360	Vulcan	ECH, A/S, S/A	62	60,000	124,830	228
175,000	535	Vulcan	ECH, A/S, S/A	37	35,000	61,000	268
173,500	HH20	BSP	ECH, Hyd., Free	36	44,080	70,530	268
165,000	D62-22	Delmag	ICH, Open	36-50	14,600	27,055	234
150,000	60X	Raymond	ECH, A/S, S/A	60	60,000	85,000	271
150,000	300E5	Conmaco	ECH, A/S, S/A	40	30,000	58,400	250
150,000	530	Vulcan	ECH, A/S, S/A	42	30,000	57,680	245
149,600	MH80B	Mitsubishi	ICH, Open	42-60	17,600	43,600	234
145,000	S-200	IHC	ECH, Hyd., Asst.	45	22,200	77,000	350
141,000	MB70	Mitsubishi	ICH, Open	38-60	15,840	46,000	234
138,800	HH16	BSP	ECH, Hyd., Free	30	35,265	50,690	303
127,500	DE150/110	MKT	ICH, Open	40-50	15,000	29,500	238
121,450	HH14	BSP	ECH, Hyd., Free	35	30,865	45,180	282
120,000	340	Vulcan	ECH, A/S, S/A	60	40,000	98,180	223
117,000	D55	Delmag	ICH, Open	36-47	12,100	26,300	213
107,170	D46-32	Delmag	ICH, Open	37-53	10,143	19,602	208
105,800	B-5505	Berminghammer	ICH, Open	36-60	9,200	24,000	264
105,600	K60	Kobe	ICH, Open	42-60	13,200	37,500	291
100,000	40X	Raymond	ECH, A/S, S/A	64	40,000	62,000	229
100,000	200E5	Conmaco	ECH, A/S, S/A	46	20,000	48,000	229
100,000	520	Vulcan	ECH, A/S, S/A	42	20,000	47,680	245
100,000	200S	ICE	ICH, Open	53-70	20,000	33,600	204
95,470	HH11	BSP	ECH, Hyd., Free	38	24,250	38,130	256
93,500	DE-150/110	MKT	ICH, Open	40-50	11,000	24,550	214
93,340	MRBS 850	Menck	ECH, A/S, S/A	45	18,960	27,890	236
92,752	KC45	Kobe	ICH, Open	39-60	9,920	24,700	215
91,100	K45	Kobe	ICH, Open	39-60	9,900	25,600	222

(Continued)

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Table 3-2c. (continued)

Maximum Striking Energy, ft-lb	Model	Manufacturer	Description	BPM	Ram Weight, lb	Hammer Weight, lb	Length in.
			Impact Hammers (Continued)				
90,000	300	Conmaco	ECH, A/S, S/A	52	30,000	55,390	202
90,000	030	Vulcan	ECH, A/S, S/A	54	30,000	53,470	196
90,000	90S	ICE	ICH, Open	38-55	9,000	16,800	207
87,400	B-5005	Berminghammer	ICH, Open	36-60	7,600	22,400	264
87,000	D44	Delmag	ICH, Open	37-56	9,500	22,300	190
85,400	MH45	Mitsubishi	ICH, Open	42-60	10,500	24,600	215
84,000	M43	Mitsubishi	ICH, Open	40-60	9,460	22,660	195
83,880	D36-32	Delmag	ICH, Open	36-53	7,938	17,397	208
81,250	8/0	Raymond	ECH, A/S, S/A	40	25,000	34,000	232
79,500	J44	IHI	ICH, Open	42-70	9,720	21,500	178
79,000	K42	Kobe	ICH, Open	40-60	9,260	24,000	212
78,075	HH9	BSP	ECH, Hyd., Free	30	19,835	24,800	252
75,900	B-4505	Berminghammer	ICH, Open	36-60	6,600	16,000	223
75,000	30X	Raymond	ECH, A/S, S/A	70	30,000	52,000	229
73,000	3400	FEC	ICH, Open	40-60	7,500	14,600	192
72,182	KC35	Kobe	ICH, Open	39-60	7,720	17,400	202
70,800	K35	Kobe	ICH, Open	39-60	7,700	18,700	212
70,000	1070	ICE	ICH, Closed	64-68	10,000	21,500	214
70,000	DE70/50C	MKT	ICH, Open	40-50	7,000	14,700	215
70,000	70S	ICE	ICH, Open	38-55	7,000	14,100	200
69,900	D30-32	Delmag	ICH, Open	36-52	6,615	13,252	207
66,000	S-90	IHC	ECH, Hyd., Asst.	50	10,000	31,400	310
65,600	MH35	Mitsubishi	ICH, Open	42-60	7,720	18,500	207
64,600	M33	Mitsubishi	ICH, Open	40-60	7,260	16,940	158
63,500	J35	IHI	ICH, Open	42-70	7,730	16,900	174
63,000	3000	FEC	ICH, Open	40-60	6,600	13,200	186
62,500	125E5	Conmaco	ECH, A/S, S/A	41	12,500	22,000	216
60,725	HH7	BSP	ECH, Hyd., Free	38	15,430	20,390	252
60,100	K32	Kobe	ICH, Open	40-60	7,050	17,750	212
60,000	200	Conmaco	ECH, A/S, S/A	55	20,000	44,560	180
60,000	020	Vulcan	ECH, A/S, S/A	59	20,000	41,670	176
60,000	S-20	MKT	ECH, A/S, S/A	60	20,000	38,650	185
60,000	512	Vulcan	ECH, A/S, S/A	41	12,000	23,480	221
58,250	D25-32	Delmag	ICH, Open	37-52	5,513	12,149	207
57,876	V25 Series I	Vulcan	ICH, Open	-	5,512	12,125	196
57,500	115E5	Conmaco	ECH, A/S, S/A	42	11,500	21,000	213
57,500	B-4005	Berminghammer	ICH, Open	36-60	5,000	14,400	223
56,875	5/0	Raymond	ECH, A/S, S/A	44	17,500	26,450	201
54,250	D30	Delmag	ICH, Open	39-60	6,600	12,300	171
53,750	B-400	Berminghammer	ICH, Open	37-60	5,000	15,000	178
51,518	KC25	Kobe	ICH, Open	39-60	5,510	12,130	170
51,000	S-70	IHC	ECH, Hyd., Asst.	50	7,700	27,500	270
50,700	K25	Kobe	ICH, Open	39-60	5,510	13,100	210
50,000	200C	Vulcan	ECH, A/S, Diff.	95	20,000	39,000	167
50,000	510	Vulcan	ECH, A/S, S/A	41	10,000	21,480	221
50,000	100E5	Conmaco	ECH, A/S, S/A	47	10,000	19,500	213
50,000	660	ICE	ICH, Closed	84-88	7,564	24,480	208
50,000	2500	FEC	ICH, Open	40-60	5,500	12,100	186
50,000	DE70/50C	MKT	ICH, Open	40-50	5,000	12,700	215
48,750	160	Conmaco	ECH, A/S, S/A	50	16,250	33,200	166
48,750	016	Vulcan	ECH, A/S, S/A	58	16,250	30,250	167
48,750	150C	Raymond	ECH, A/S, Diff.	95-105	15,000	32,500	189
48,750	4/0	Raymond	ECH, A/S, S/A	46	15,000	23,800	193
48,500	D22-33	Delmag	ICH, Open	38-52	4,850	11,400	206
46,900	MH25	Mitsubishi	ICH, Open	42-60	5,510	13,200	200
45,200	MRBS 500	Menck	ECH, A/S, S/A	48	11,020	15,210	200
45,000	M23	Mitsubishi	ICH, Open	42-60	5,060	11,220	169
44,800	DE50C	BSP	ICH, Open	42-54	4,980	10,300	172
44,000	MS-500	MKT	ECH, A/S, S/A	40-50	11,000	15,500	181
44,000	B-3505	Berminghammer	ICH, Open	36-60	4,000	12,000	217

(Continued)

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Table 3-2c. (continued)

Maximum Striking Energy, ft-lb	Model	Manufacturer	Description	BPM	Ram Weight, lb	Hammer Weight, lb	Length in.
			Impact Hammers (Continued)				
43,375	HH5	BSP	ECH, Hyd., Free	40	11,020	15,980	229
42,800	D19-32	Delmag	ICH, Open	37-53	4,190	7,800	186
42,500	DA-55C	MKT	ICH, Open	40-50	5,000	17,000	208
42,000	140	Conmaco	ECH, A/S, S/A	55	14,000	30,750	166
42,000	014	Vulcan	ECH, A/S, S/A	59	14,000	27,500	164
42,000	42S	ICE	ICH, Open	37-55	4,088	7,610	193
41,300	K22	Kobe	ICH, Open	40-60	4,850	12,350	210
40,600	3/0	Raymond	ECH, A/S, S/A	50	12,500	21,000	187
40,300	B-300	Berminghammer	ICH, Open	37-60	3,750	9,520	168
40,200	D16-32	Delmag	ICH, Open	36-52	3,528	7,386	186
40,000	508	Vulcan	ECH, A/S, S/A	41	8,000	19,480	221
40,000	80E5	Conmaco	ECH, A/S, S/A	47	8,000	17,500	213
40,000	640	ICE	ICH, Closed	74-77	6,000	14,460	187
40,000	40S	ICE	ICH, Open	38-55	4,000	7,500	188
40,000	DE33/30/20C	MKT	ICH, Open	40-50	3,300	8,700	191
39,700	D22	Delmag	ICH, Open	42-60	4,850	11,200	170
39,100	J22	IHI	ICH, Open	42-70	4,850	10,800	168
38,200	DA-55C	MKT	ICH, Closed	78-82	5,000	17,000	208
37,500	S-14	MKT	ECH, A/S, S/A	60	14,000	31,700	163
37,375	115	Conmaco	ECH, A/S, S/A	52	11,500	20,830	170
36,000	140C	Vulcan	ECH, A/S, Diff.	101	14,000	27,984	147
34,500	B-2505	Berminghammer	ICH, Open	36-60	3,000	11,000	216
34,000	DA-45C	MKT	ICH, Open	40-50	4,000	14,200	181
33,000	DE33/30/20C	MKT	ICH, Open	40-50	3,300	8,700	191
32,885	100C	Vulcan	ECH, A/S, Diff.	103	10,000	22,200	168
32,500	S-10	MKT	ECH, A/S, S/A	55	10,000	22,380	169
32,500	100	Conmaco	ECH, A/S, S/A	55	10,000	19,280	170
32,500	010	Vulcan	ECH, A/S, S/A	50	10,000	18,780	180
32,500	2/0	Raymond	ECH, A/S, S/A	50	10,000	18,550	180
32,500	506	Vulcan	ECH, A/S, S/A	46	6,500	13,025	209
32,500	65E5	Conmaco	ECH, A/S, S/A	50	6,500	12,500	202
31,320	D12-32	Delmag	ICH, Open	36-52	2,820	6,260	186
30,800	MS-350	MKT	ECH, A/S, S/A	40-50	7,716	10,500	181
30,700	DA-45C	MKT	ICH, Closed	78-82	4,000	14,200	181
30,000	520-30	ICE	ICH, Closed	80-84	5,070	13,400	162
29,250	B-225	Berminghammer	ICH, Open	39-60	3,000	8,730	168
28,100	MH15	Mitsubishi	ICH, Open	42-60	3,310	8,400	193
28,000	DE33/30/20C	MKT	ICH, Open	40-50	2,800	8,200	191
27,100	1500	FEC	ICH, Open	40-60	3,300	7,225	170
27,100	D15	Delmag	ICH, Open	42-60	3,300	6,615	167
27,000	DE30C	BSP	ICH, Open	42-54	3,000	7,600	170
26,300	520-26	ICE	ICH, Closed	80-84	5,070	12,545	162
26,025	HH3	BSP	ECH, Hyd., Free	46	6,610	11,570	205
26,000	85C	Vulcan	ECH, A/S, Diff.	111	8,525	19,020	151
26,000	S-8	MKT	ECH, A/S, S/A	55	8,000	18,300	172
26,000	80	Conmaco	ECH, A/S, S/A	56	8,000	17,280	170
26,000	08	Vulcan	ECH, A/S, S/A	50	8,000	16,750	178
26,000	M14S	Mitsubishi	ICH, Open	42-60	2,970	7,260	162
25,500	S-35	IHC	ECH, Hyd., Asst.	60	7,300	23,200	220
25,000	505	Vulcan	ECH, A/S, S/A	46	5,000	11,800	209
25,000	50E5	Conmaco	ECH, A/S, S/A	48	5,000	11,000	202
24,450	80C	Vulcan	ECH, A/S, Diff.	109	8,000	17,885	151
24,450	80C	Raymond	ECH, A/S, Diff.	95-105	8,000	17,885	146
24,450	80CH	Raymond	ECH, Hyd., Asst.	110-120	8,000	17,780	143
24,400	K13	Kobe	ICH, Open	40-60	2,860	7,300	200
24,375	0	Vulcan	ECH, A/S, S/A	50	7,500	16,250	180
24,375	0	Raymond	ECH, A/S, S/A	50	7,500	16,000	180
24,000	C-826	MKT	ECH, A/S, Compound	85-95	8,000	17,750	146
23,800	DA-35C	MKT	ICH, Open	40-50	2,800	10,800	204
23,000	B-23	Berminghammer	ICH, Closed	80	2,800	9,940	192

(Continued)

(Sheet 3 of 6)

Table 3-2c. (continued)

Table 3-2c. (continued)

Maximum Striking Energy, ft-lb	Model	Manufacturer	Description	BPM	Ram Weight, lb	Hammer Weight, lb	Length in.
			Impact Hammers (Continued)				
22,500	422	ICE	ICH, Closed	76-82	4,000	9,750	167
22,500	1200	FEC	ICH, Open	40-60	2,750	6,540	168
22,500	D12	Delmag	ICH, Open	42-60	2,750	6,050	167
22,500	30S	ICE	ICH, Open	44-67	3,000	6,250	148
21,000	DA-35C	MKT	ICH, Closed	40-50	2,800	10,800	204
20,000	DE33/30/20C	MKT	ICH, Open	40-50	2,000	7,400	192
19,500	65C	Raymond	ECH, A/S, Diff.	110	6,500	14,675	140
19,500	65CH	Raymond	ECH, Hyd., Asst.	128-136	6,500	14,615	144
19,500	1-S	Raymond	ECH, A/S, S/A	58	6,500	12,500	153
19,500	65	Conmaco	ECH, A/S, S/A	61	6,500	12,100	154
19,500	106	Vulcan	ECH, A/S, S/A	60	6,500	11,200	156
19,500	06	Vulcan	ECH, A/S, S/A	60	6,500	11,200	156
19,200	65C	Vulcan	ECH, A/S, Diff.	117	6,500	14,886	145
19,150	11B3	MKT	ECH, A/S, D/A	95	5,000	14,000	123
18,100	440	ICE	ICH, Closed	88-92	4,000	9,840	162
18,000	312	ICE	ICH, Closed	100-105	3,857	10,375	129
18,000	B-200	Berminghammer	ICH, Open	39-58	2,000	6,940	165
18,000	D8-22	Delmag	ICH, Open	38-52	1,762	4,220	185
17,360	HPH 2400	Dawson	ECH, Hyd.	140	4,189	13,227	208
16,250	S-5	MKT	ECH, A/S, S/A	60	5,000	12,460	159
16,000	C-5	MKT	ECH, Steam, Compound	100-110	5,000	11,800	105
15,180	HH2	BSP	ECH, Hyd., Asst.	55	4,408	9,036	122
15,100	50C	Vulcan	ECH, A/S, Diff.	117	5,000	9,700	132
15,000	1	Vulcan	ECH, A/S, S/A	60	5,000	11,000	153
15,000	50	Conmaco	ECH, A/S, S/A	64	5,000	10,600	154
14,200	C-5	MKT	ECH, Air, Compound	100-110	5,000	11,880	105
13,100	10B3	MKT	ECH, A/S, D/A	105	3,000	10,850	110
10,500	D6-32	Delmag	ICH, Open	39-52	1,322	3,810	150
9,350	DA-15C	MKT	ICH, Open	40-50	1,100	4,825	167
9,100	D5	Delmag	ICH, Open	42-60	1,100	2,730	150
8,800	DE10	MKT	ICH, Open	40-50	1,100	3,100	146
8,750	9B3	MKT	ECH, A/S, D/A	145	1,600	7,000	100
8,750	900	BSP	ECH, A/S, D/A	145	1,600	7,100	94
8,680	HPH 1200	Dawson	ECH, Hyd.	120	2,300	6,614	184
8,200	DA-15C	MKT	ICH, Closed	86-92	1,100	4,825	168
8,100	180	ICE	ICH, Closed	90-95	1,725	4,645	135
7,260	30C	Vulcan	ECH, A/S, Diff.	133	3,000	7,036	107
7,260	2	Vulcan	ECH, A/S, S/A	70	3,000	6,700	139
4,700	700N	BSP	ECH, A/S, D/A	225	850	6,630	89
4,150	7	MKT	ECH, A/S, D/A	225	800	5,000	73
4,000	DGH-900	Vulcan	ECH, A/S, Diff.	328	900	5,000	81
3,630	D4	Delmag	ICH, Open	50-60	836	1,360	93
3,000	600N	BSP	ECH, A/S, D/A	250	500	4,800	86
2,500	6	MKT	ECH, A/S, D/A	275	400	2,900	63
1,815	D2	Delmag	ICH, Open	60-70	484	792	81
1,200	500N	BSP	ECH, A/S, D/A	330	200	2,520	71
1,000	5	MKT	ECH, A/S, D/A	300	200	1,500	55
386	DGH-100D	Vulcan	ECH, A/S, Diff.	303	100	786	50
350	300	BSP	ECH, A/S, D/A	400	68	675	58
-	3	MKT	ECH, A/S, D/A	400	68	675	53
160	200	BSP	ECH, A/S, D/A	500	48	343	33
-	2	MKT	ECH, A/S, D/A	500	48	343	29
-	1	MKT	ECH, A/S, D/A	500	21	145	39

(Continued)

(Sheet 4 of 6)

Table 3-2c. (continued)

Maximum Dynamic Force, Tons	Model	Manufacturer	Frequency, VPM	Eccentric Moment, in.-lb	Max. Extraction Full, tons	Suspended Weight lb	Length in.
			Vibratory Hammers				
390	V-140	MKT	1,400	14,000	150	42,000	174
390	400	APE	1,400	14,000	200	27,300	128
298	300	APE	1,500	10,000	150	19,000	96
287	HBV260.01	Delmag/Tunkers	1,600	7,938	110	22,000	177
244	110H1	PTC	1,350	9,550	132	39,050	121
244	110H2	PTC	1,350	9,550	88	32,900	153
222	1412B	ICE	1,250	10,000	150	28,500	174
218.1	600	HPSI	1,600	6,000	75	16,500	116
200	520	HPSI	1,600	5,200	60	16,500	117
199	60H1	PTC	1,650	5,210	88	29,200	156
182	V-36	MKT	1,600	5,000	80	18,800	157
180	180	APE	1,700	4,400	100	10,000	90
167.2	4600A	Vulcan	1,600	4,600	80	14,000	131
167.2	4600	Vulcan	1,600	4,600	66	16,000	96
166	50H3	PTC	1,650	4,340	44	23,300	168
164	1412A	ICE	1,200	8,000	100	26,900	144
163.5	450	HPSI	1,600	4,500	75	13,600	111
162	23HF1	PTC	2,400	2,000	44	12,850	118
160	V-30	MKT	1,600	4,400	80	15,000	133
160	815	ICE	1,600	4,400	50	15,600	122
151	423	ICE	2,200	2,200	40	9,900	96
148	150	APE	1,800	3,200	100	9,500	90
148	23HF1	PTC	2,300	2,000	44	12,850	118
145	400	HPSI	1,600	4,000	75	13,600	111
143	HBV130.01	Delmag/Tunkers	1,600	3,991	110	12,600	159
134	4150	Foster	1,500	4,166	55	16,495	78
116	V-20	MKT	1,650	3,000	60	12,500	146
110	HVB100.01	Delmag/Tunkers	1,600	2,950	55	10,580	168
100	260	HPSI	1,600	2,600	60	10,750	100
100	2800	Vulcan	2,400	1,200	25	8,300	78
90	612	ICE	1,200	4,400	40	12,700	112
84	13HF1	PTC	2,300	1,130	33	7,150	82
83.6	2300A	Vulcan	1,600	2,300	48	8,550	115.5
83.6	2300	Vulcan	1,600	2,300	33	8,200	93
83	25H2	PTC	1,650	2,170	44	9,350	107
83	223	ICE	2,300	1,100	40	5,350	67
80	V-17	MKT	1,600	2,200	60	12,000	120
80	416L	ICE	1,600	2,200	40	9,900	96
78	HVB70.01	Delmag/Tunkers	1,630	2,065	55	7,050	122
78	V-16	MKT	1,750	1,800	40	9,250	134
69	25H1	PTC	1,500	2,170	44	9,350	107
66	HVB60.05	Delmag/Tunkers	1,500	2,065	28	7,050	63
65	1800	Foster	1,600	1,800	30	11,000	80
59	50	APE	1,800	1,300	40	4,800	61
57.5	416	ICE	1,500	1,800	40	13,100	105
-	H-1700	H&M	1,250	-	30	7,000	96
55	HVB60.05	Delmag/Tunkers	1,000	3,879	28	7,050	63
50	130	HPSI	1,600	1,300	30	7,050	78
50	V-5B	MKT	1,800	1,100	30	7,200	92
49.1	1400	Vulcan	2,400	600	25	4,350	104
46	V-14	MKT	1,500	1,500	40	10,000	133
44	HVB40.01	Delmag/Tunkers	1,500	1,346	28	5,700	116
44	13H1	PTC	1,700	1,085	22	5,400	77
42	7HF1	PTC	2,300	565	17	3,800	69
41.8	1150A	Vulcan	1,600	1,150	32	6,300	95
41.8	1150	Vulcan	1,600	1,150	33	6,500	78
40	216	ICE	1,600	1,100	40	5,350	78
37	1000	Foster	1,600	1,000	20	6,094	71
33	HVB30	Delmag/Tunkers	1,800	712	13	2,100	40
30	V-5	MKT	1,450	1,000	30	5,200	88

(Continued)

(Sheet 5 of 6)

Table 3-2c. (concluded)

Maximum Dynamic Force, Tons	Model	Manufacturer	Frequency, VPM	Eccentric Moment, in.-lb	Max. Extraction Full, tons	Suspended Weight lb	Lengtl in.
			Vibratory Hammers (Continued)				
-	H-75B	H&M	1,900	-	15	4,000	67
-	30	APE	2,500	50	5	175	18
27	HVB24	Delmag/Tunkers	2,100	434	13	2,100	39
23.7	65	HPSI	1,600	650	25	3,100	59
23	6H1	PTC	1,800	-	22	3,180	49
18	V-2A	MKT	1,800	400	8	2,400	44
17	HVB16	Delmag/Tunkers	2,100	278	13	2,000	39
17	400	Vulcan	2,400	200	10	1,100	48
17	400A	Vulcan	2,400	200	10	1,100	48
12	HVB10	Delmag/Tunkers	2,100	191	13	2,000	39

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Table 3-3. Summary of cushion material characteristics.

Name of Material	Description	Advantages	Disadvantages
Wire Rope Biscuits	Wire rope coiled into flat biscuits, then placed into cushion receptacle.	Inexpensive; material plentiful on job site.	Material hardens rapidly into steel mass; extensive hammer and pile damage possible.
Force Ten	Wires braided into steel cloth plates.	Long life	Material hardness rapidly into steel mass; extensive hammer and pile damage possible.
Micarta and Aluminum; Conbest	Phenolic plates alternated with aluminum plates. Aluminum plates used for heat dissipation, frequently omitted with diesel hammers.	Long life; versatile, can be used with many hammers.	Material too hard to be used with some hammers.
Hamortex	Aluminum foil bonded with various plastic and paper materials and spirally wound into disks.	Excellent range of elasticity moduli. Can be used as concrete pile cushion as well as hammer cushion.	Inconsistent life; varies widely from job to job.
MC-904/Blue Nylon	Type of nylon, cast into discs. Sometimes alternated with aluminum for heat dissipation.	Excellent range of elasticity moduli.	Material breaks up and melts easily under hard conditions; best suited for diesel hammers.
Plywood	Plywood sheets stacked and mounted onto concrete pile; used almost exclusively as a pile cushion.	Inexpensive and soft.	Poor coefficient of restitution; absorbs much impact energy.
End grain hardwood (oak, hickory, bongossi, etc.)	Wood cut to use in cushion receptacle as single block or into blocks which are then fit into receptacle.	Good range of modulus of elasticity.	Low coefficient of restitution; sometimes expensive and hard to find; burns during use.

Table 3-4. Advantages and disadvantages of fixed leaders.

Advantages	Disadvantages
Requires only a two-drum crane.	Heaviest and most expensive of the three-leader types.
Superior accuracy in located leader in vertical position and all batter positions.	More troublesome to assemble and maintain.
Rigid control of leader during positioning operation.	
Compound batter angles can be set and accurately maintained.	
Boom can be lowered and leaders folded under (for short haul over the road and railroad travel) when crane of adequate capacity is used. This depends on the length of leader and boom and the configurations of the crane.	

Table 3-5. Advantages and disadvantages of swinging leaders.

Advantages	Disadvantages
Lightest, simplest, and least expensive.	Requires a 3-drum crane (one of leader, one for hammer, one for pile) or a 2-drum crane with lead hung on sling from boom point.
With stabbing points secured in the ground, this lead is free to rotate sufficiently to align hammer with pile without precise alignment of crane with	Difficult control twist of leader if pile. stabbing points are not secured into the ground.
Leaders are generally 15 to 20 feet shorter than the boom; crane can reach out further, assuming crane capacity is sufficient.	Crane positioning is more difficult than with any other type of leader. Operator must rely on balance while center of gravity continues to move.
Can drive in a hole or ditch or over the edge of an excavation.	
For long lead and boom requirement, the leader weight can be supported on the ground while the pile is lifted into place without excessively increasing the working load.	

Table 3-6. Advantages and underhung leaders.

Advantages	Disadvantages
Lighter and generally less expensive than extended type leader.	Cannot be used for side-to-side batter driving.
Requires only 2-drum crane.	Length of pile limited by boom length, as this type of leader cannot extend above the boom point.
Accurate in locating leader in vertical or fore and aft batter positions.	When long leaders require a long boom, working radius of crane is reduced, which reduces the effective capacity of the crane.
Rigid control of leader during positioning operation.	
Relatively short rigging time in setting up and breaking down.	
Utilize sheave lead in crane boom.	

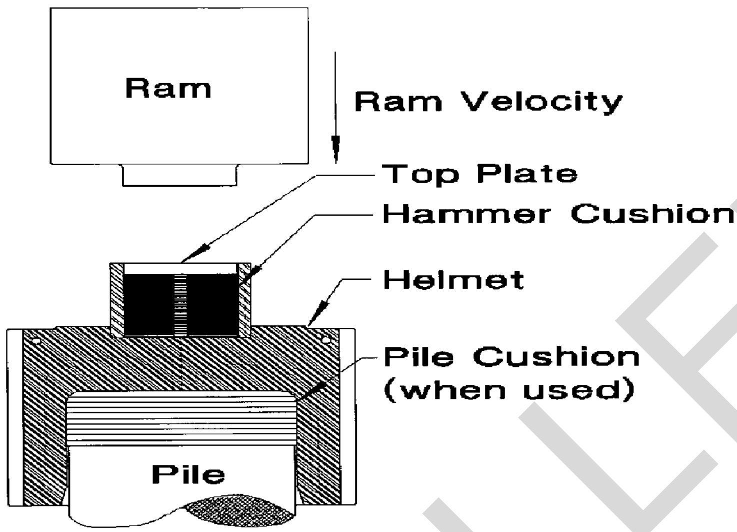


Figure 3-1. Impact hammer system schematic.

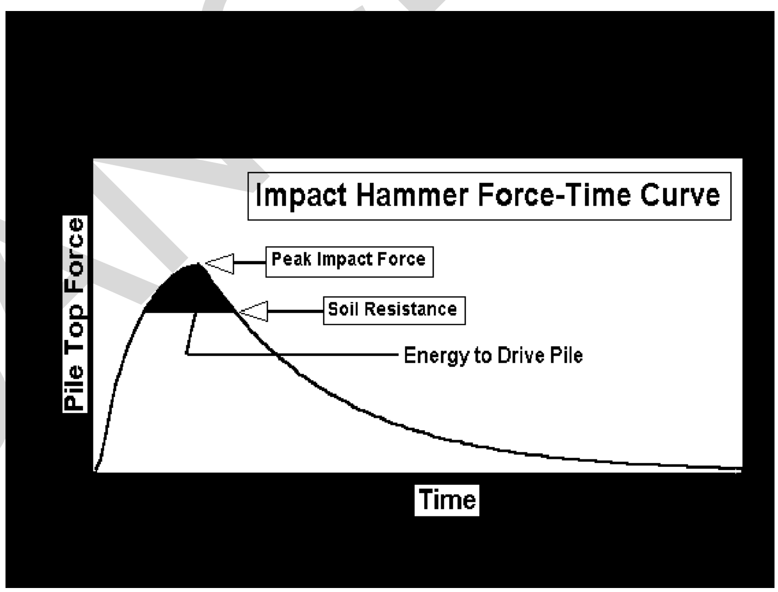


Figure 3-2. Idealized hammer force-time curve.

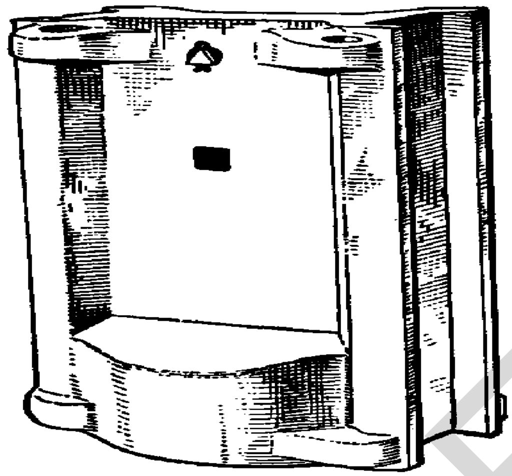


Figure 3-3. Typical drop hammer.

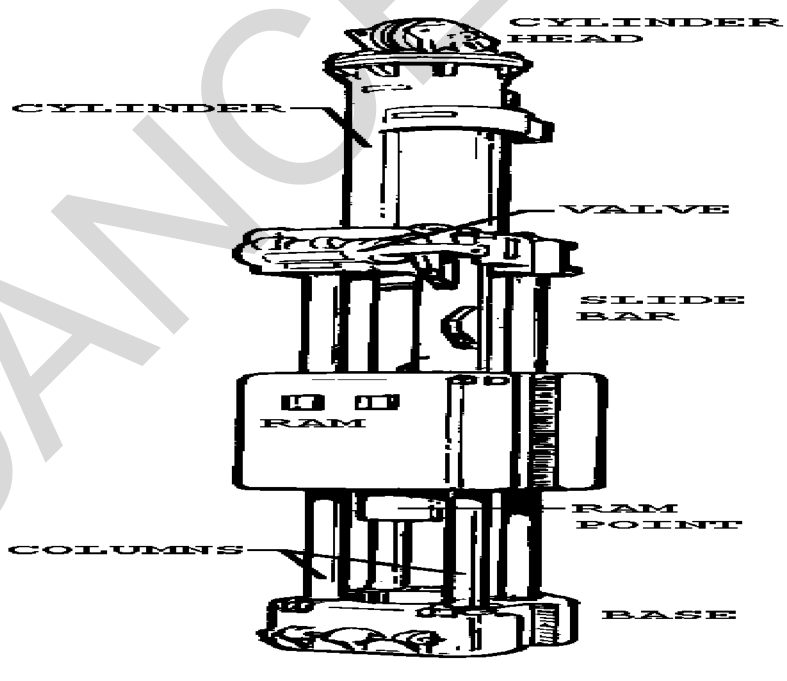


Figure 3-4. Typical single-acting air/system hammer.

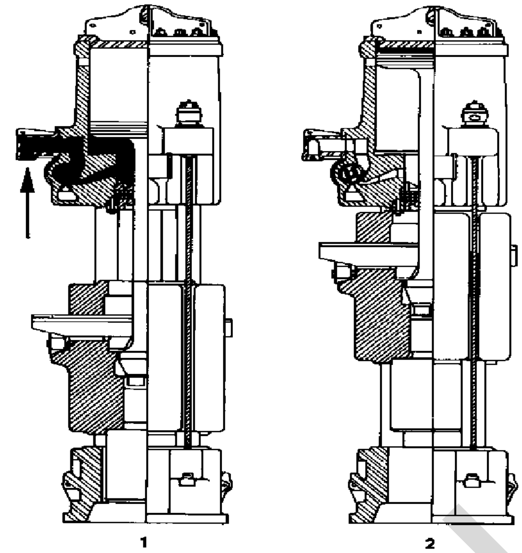


Figure 3-5. Single-acting air/steam hammer operating cycle.

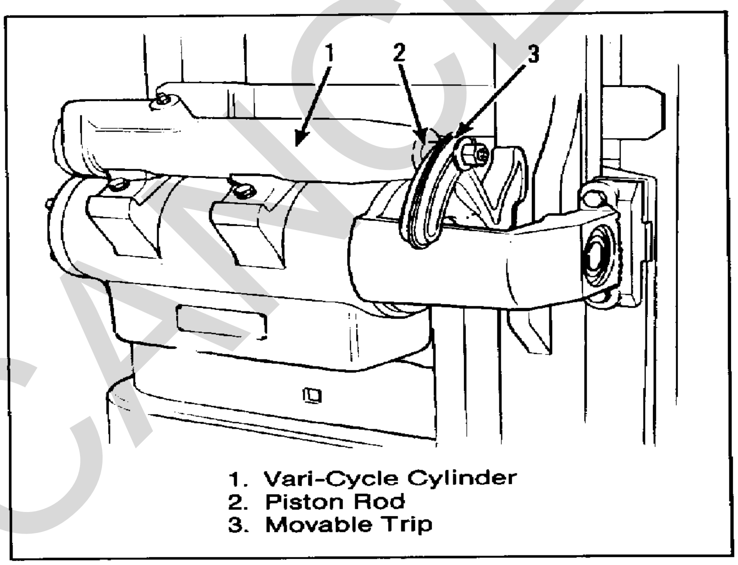


Figure 3-6. Stroke changing device for air/steam hammers.

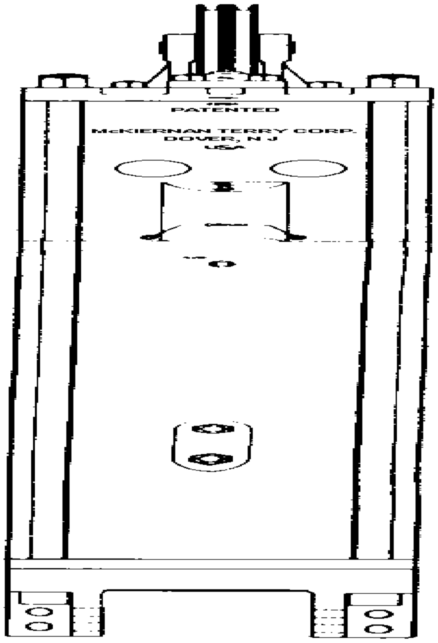


Figure 3-7a. Typical double-acting air/stream hammer.

The ram carries a rigidly connected cam throw (some double acting hammers use a fluid valve) which engages a camrod suspended in the intermediate head of the hammer. In operation, the motive fluid first enters the inlet port and flows through the lower opening of the valve to the underside of the piston. The top opening of the valve completes a path from the topside of the piston to the exhaust port. As the fluid lifts the piston, thus lifting the ram, the lugs of the cam throw slide past the edges of the cam rod until, at the top of the stroke, they engage a spiral portion of the cam rod, causing it to rotate. The valve connected to the top of the cam rod also rotates, allowing the inlet motive fluid on the underside of the piston to escape through the exhaust port. The ram then falls, its velocity increased by the fluid pressure on the top of the piston. The cam throw lugs slide down where another spiral portion of the cam rode is engaged. The cam rod rotates; rotating the valve to the original position, and the motive fluid path reverses.

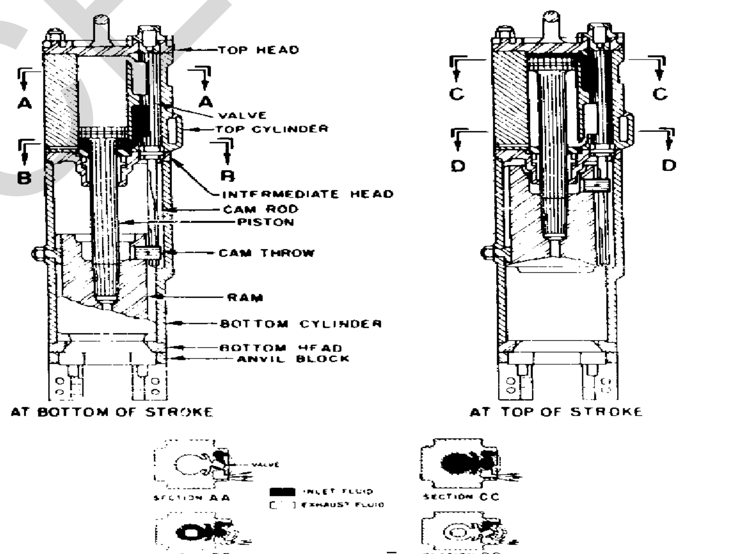


Figure 3-7b. Interior view of air/ateam hammer.

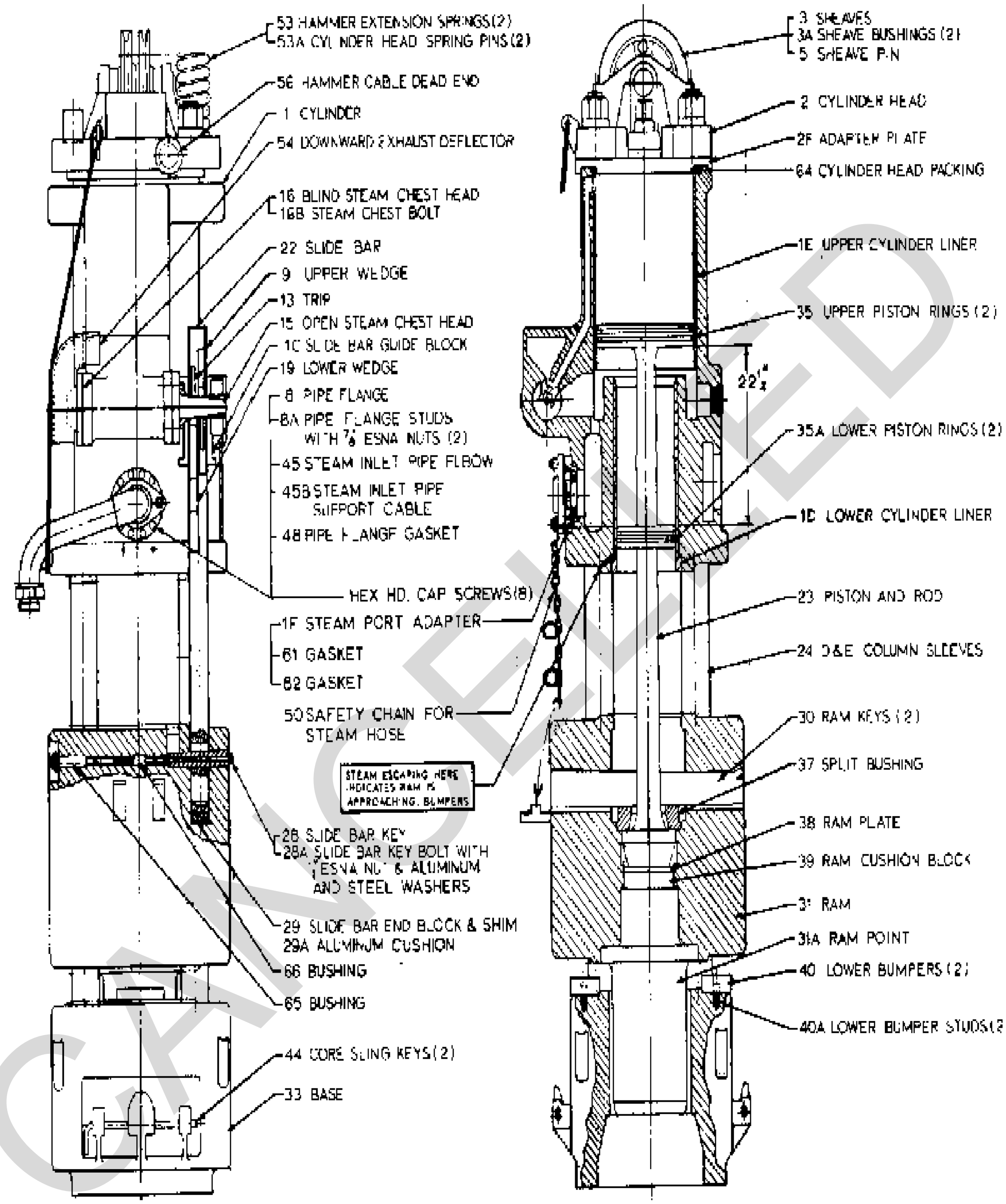
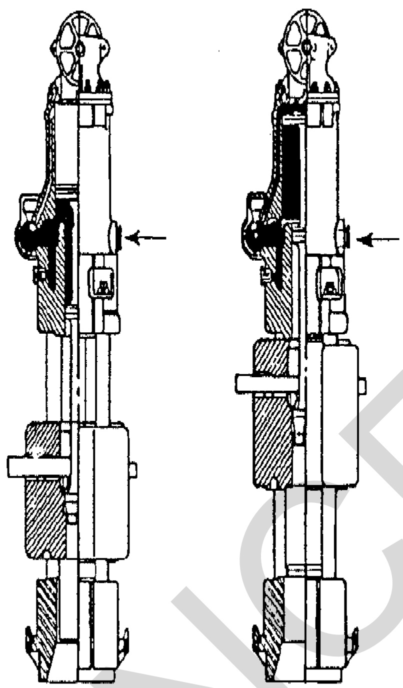


Figure 3-8. Typical differential-acting air/steam hammer.



The cycle begins at impact, the valve rotated so that the area above the large piston is open to the atmosphere and exhausting the compressed air or steam from the previous stroke, as shown in View 3. The area in the cylinder between the large and small pistons is always pressurized, and, as in the beginning of the cycle, when there is only atmospheric on the top of the large piston, this creates an unbalanced force on the piston and the ram accelerates upward. As the ram moves upward the intake wedge actuates the trip, rotating the valve and admitting steam to the cylinder above the large piston as shown in View 4. This produces an unbalanced force downward on the ram, bringing the ram to a halt at the top of the stroke. The ram is then forced downward, gaining kinetic energy both from gravity and the downward acting steam or air force, to impact. Just before impact the exhaust wedge rotates the valve once again to exhaust the compressed air or steam above the large piston and the cycle starts once again.

Figure 3-9. Operating cycle for differential-acting air/steam hammers.

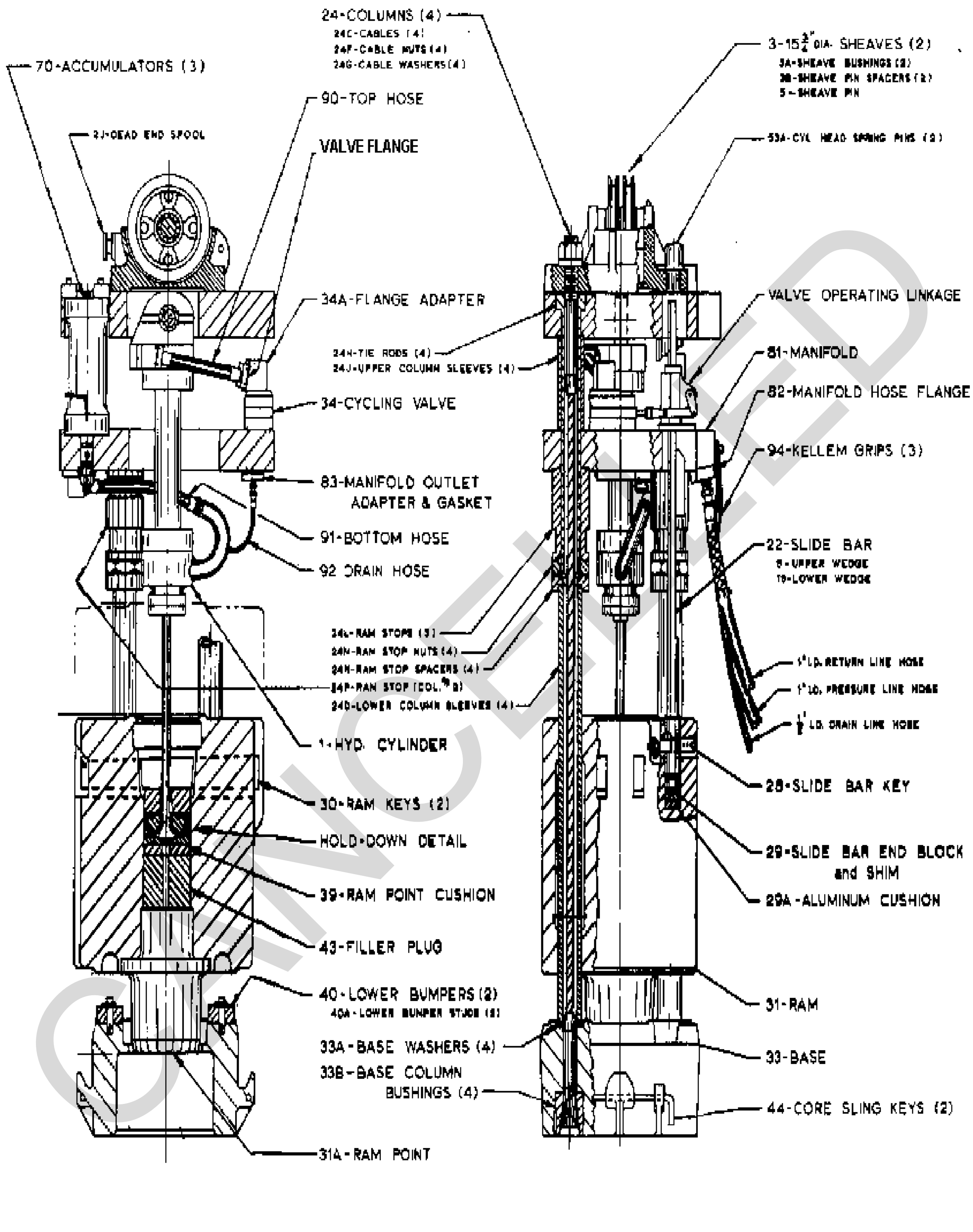
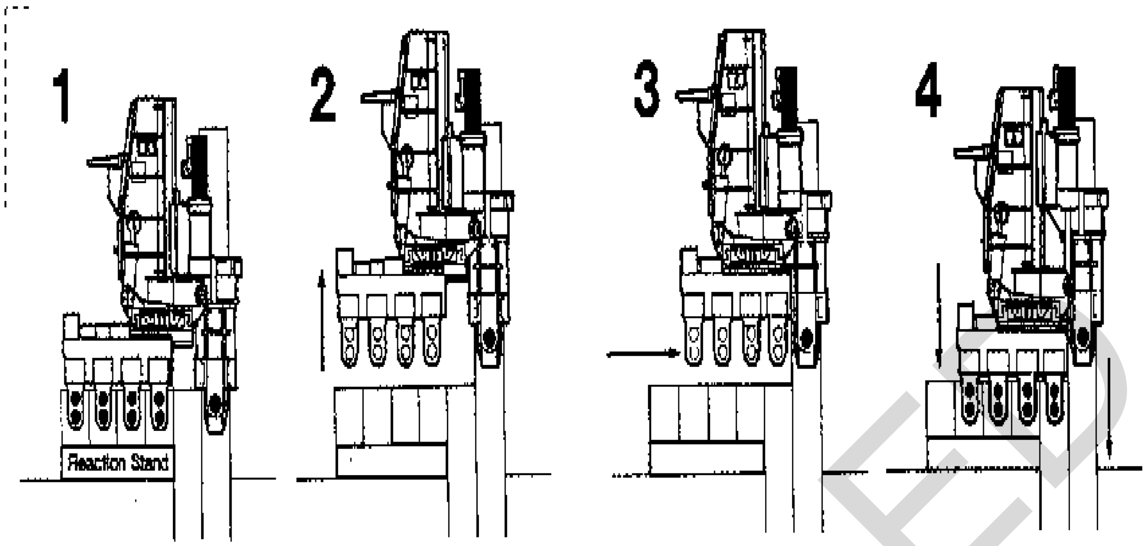
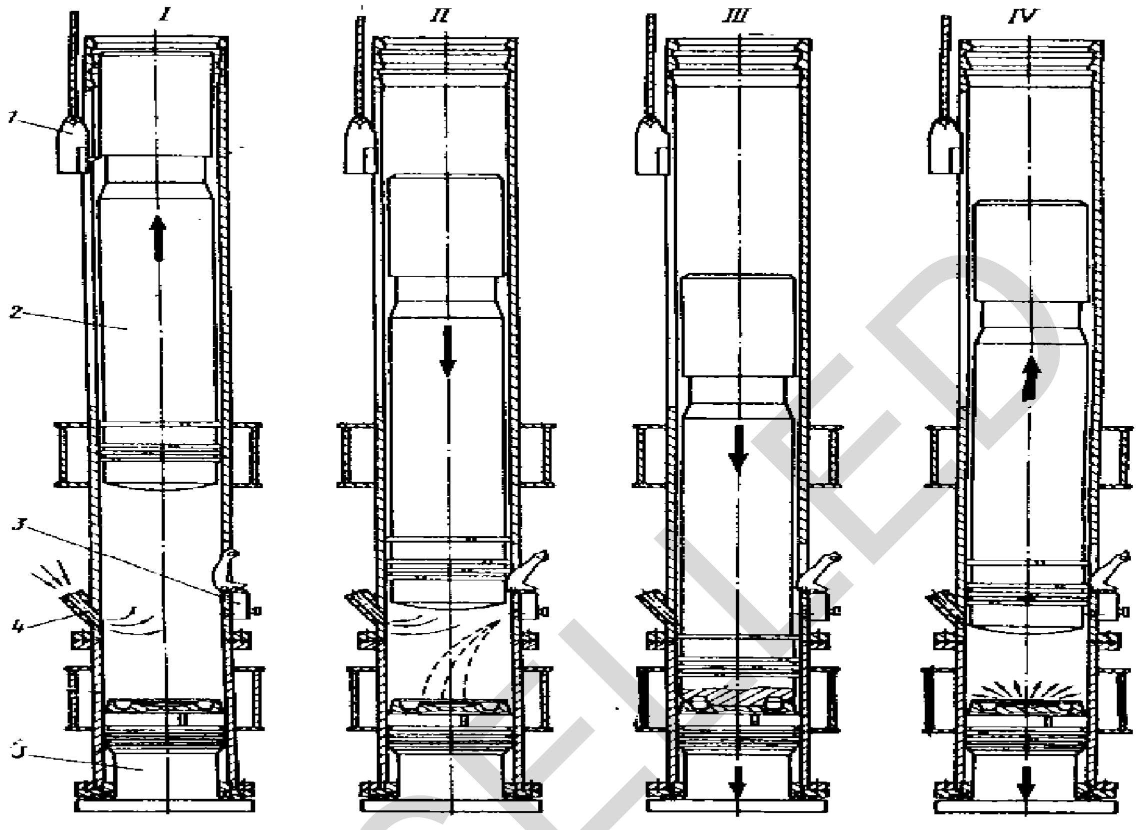


Figure 3-10. Typical hydraulic hammer.



Operating Cycle:
(1) The jack is set on the reaction standard for the installation of the first two sheet piles.
(2) The jack moves by elevating its travel carriage while supporting itself on the last installed pile.
(3) The travel carriage then slides forward.
(4) The travel carriage lowers itself and drops onto the installed sheet piles and continues its hydraulic installation process. After the third or fourth piles are driven, the jack moves off the reaction stand and travels independently on the piles.

Figure 3-11. Pile jacking device.

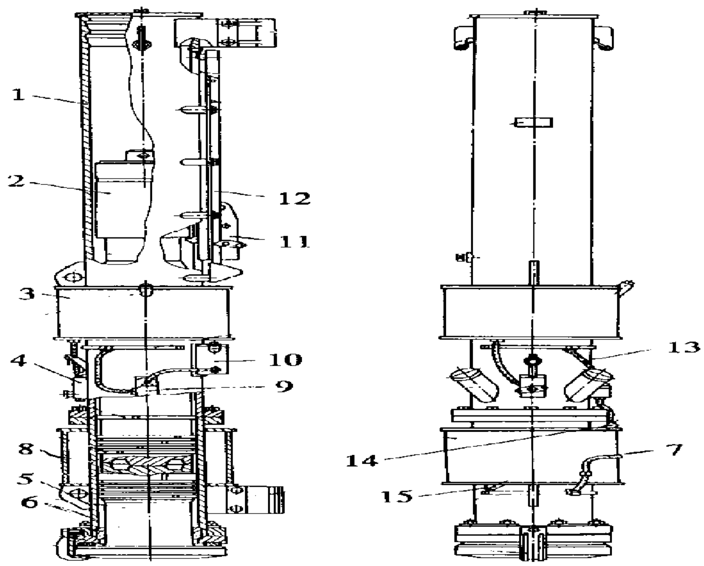


Hammer Parts: (1) starting device (2) Piston
(3) fuel pump (4) inlet
(5) cylinder (6) anvil

Stages in Cycle:

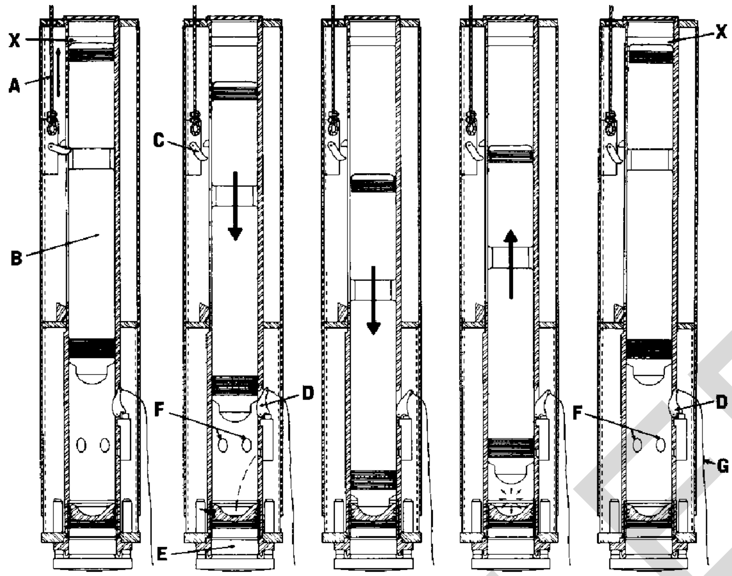
I. Ram up (start), scavenging
II. Termination of scavenging, fuel feed
III. Termination of compression stroke, blow deliveries on anvil block, fuel combustion
IV. Termination of fuel combustion, exhaust, beginning of scavenging.

Figure 3-12. Operating diagram of diesel hammer.



Parts:
(1) Upper Cylinder
(2) Piston
(3) Fuel Tank
(4) Fuel Pump
(5) Lower Cylinder
(6) Anvil Block
(7) Oil Hose for Anvil Block Lubrication
(8) Water Tank
(9) Oil Pump
(10) Oil Tank
(11) Starting Device or Crab
(12) Staring Device or Crab Guide
(13) Oil Hose for Ram Rings Lubrication
(14) Filling Throat Plug
(15) Drain Throat Plug

Figure 3-13. Typical open-end diesel hammer.



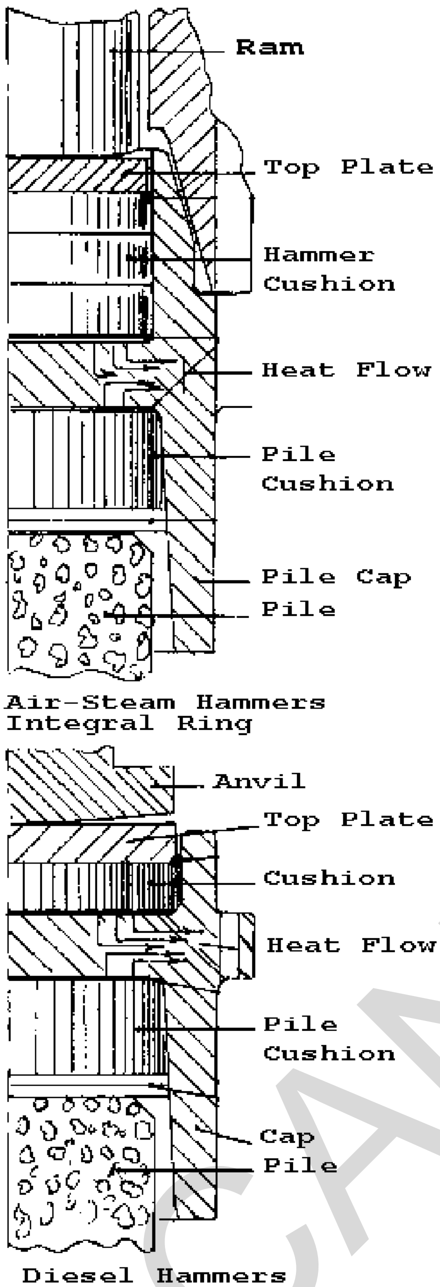
A. Lifting Line from Crane
B. Ram
C. Starting Device
D. Fuel Pump
E. Anvil
F. Exhaust Ports
G. Fuel Pump Lever Rope
X. Compression Chamber

Figure 3-14. Typical closed-end diesel hammer.

The figure slows typical cushion configurations for both air/steam and diesel hammers. Pile cushion is only included in concrete and plastic piling; with steel and wood piling, no pile cushion is normally needed.

In some instances, no hammer cushion is required. Some hydraulic hammers have no hammer cushion. With wood piling, some air/steam hammers can be equipped with special bases to drive wood piling without cushion material.

The air/steam configuration to the left is shown with an integrally cost cushion pot. Most air/steam hammers can use as an alternative a capblock follower or shield, where micarta and aluminum cushion material is stacked into a piece separate from the pile cap. Such a configuration is shown in the below right figure.



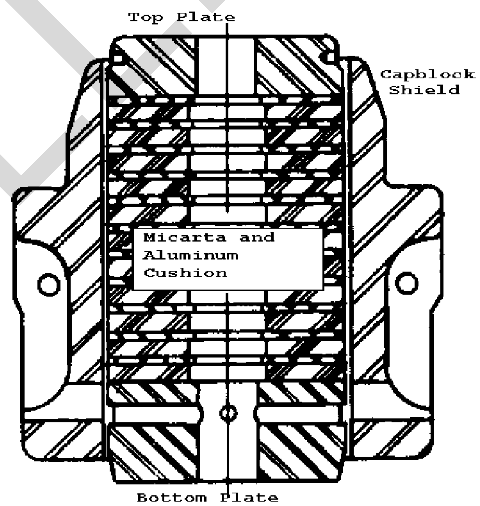


Figure 3-15. Cushion configurations.

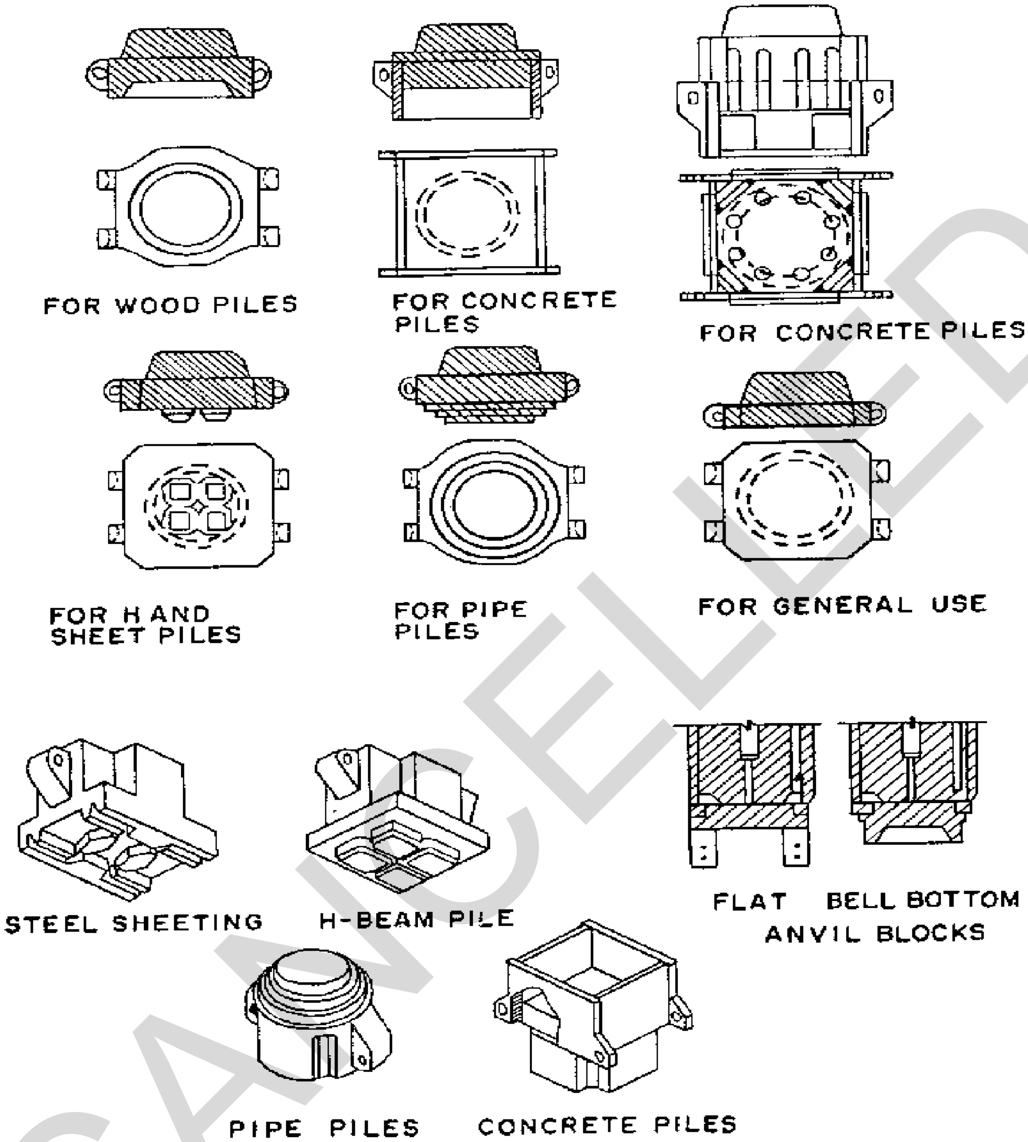


Figure 3-16. Typical driving helmets.

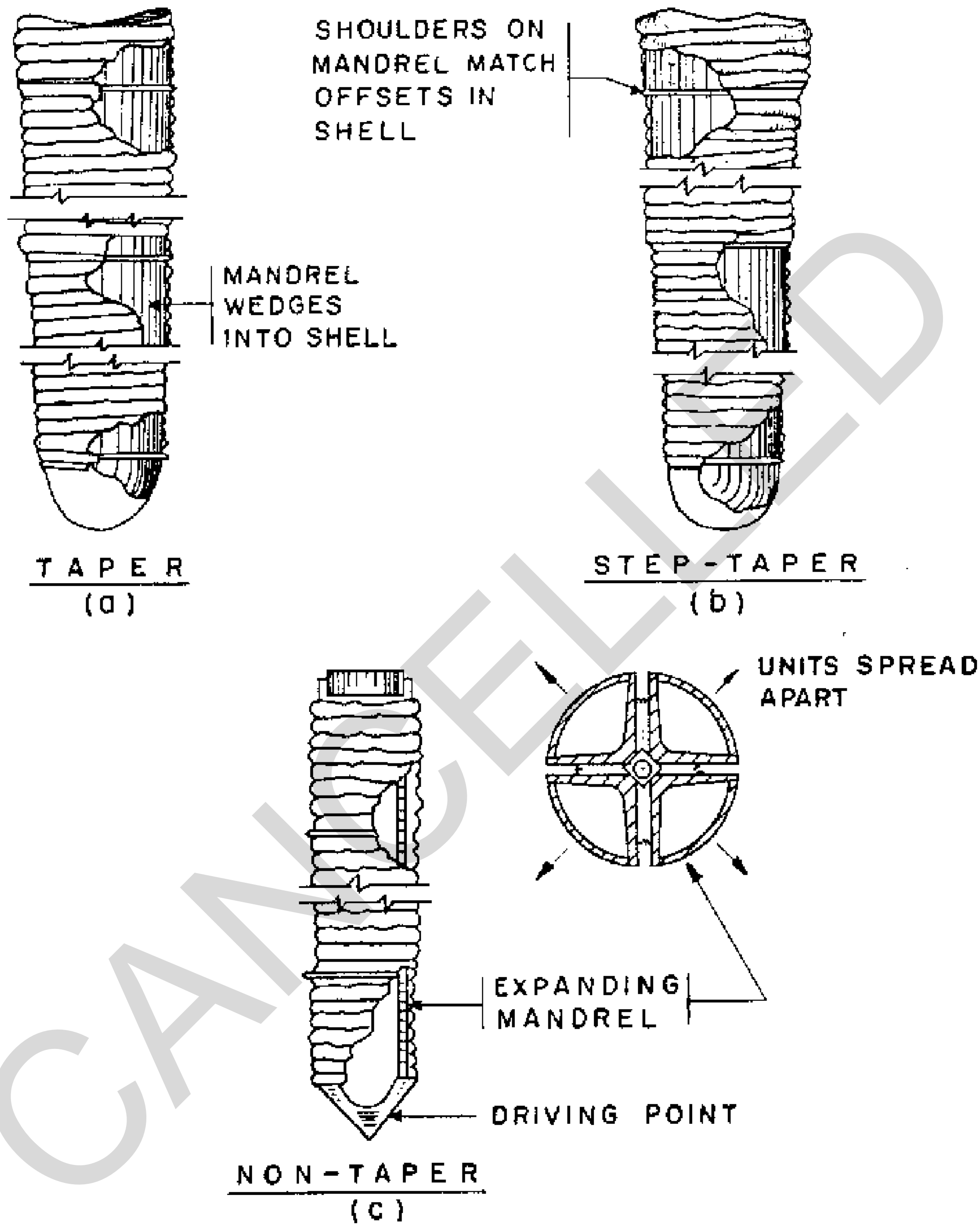


Figure 3-17. Mandrel for shell piles.

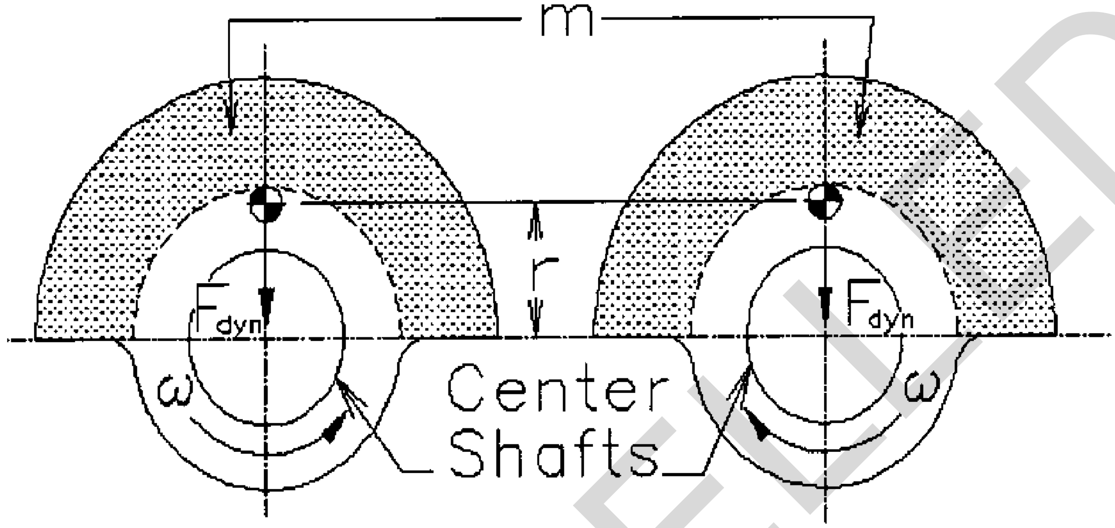
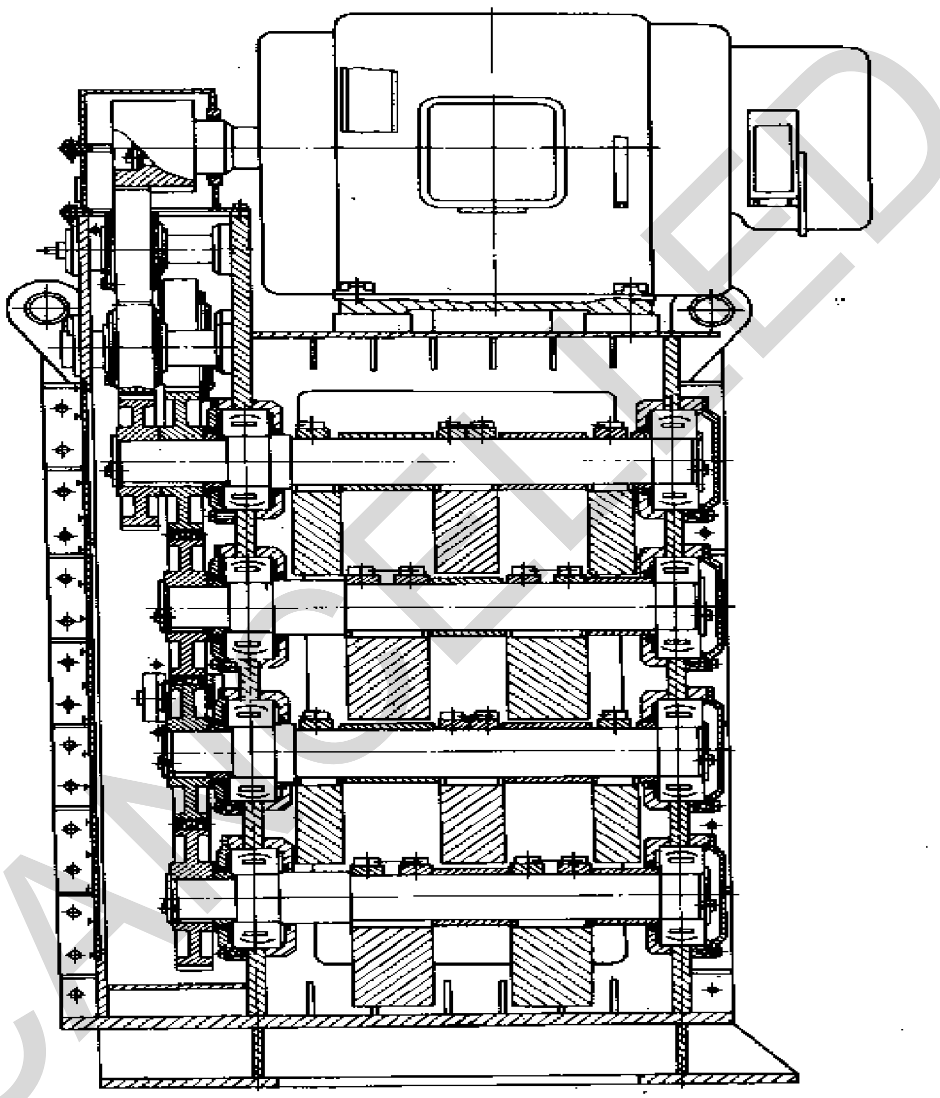


Figure 3-18. Eccentrics for vibratory hammers.



ис. 4.18. Низкочастотный вибропогружатель ВПМ-170

Figure 3-19. Low-frequency vibratory hammer.

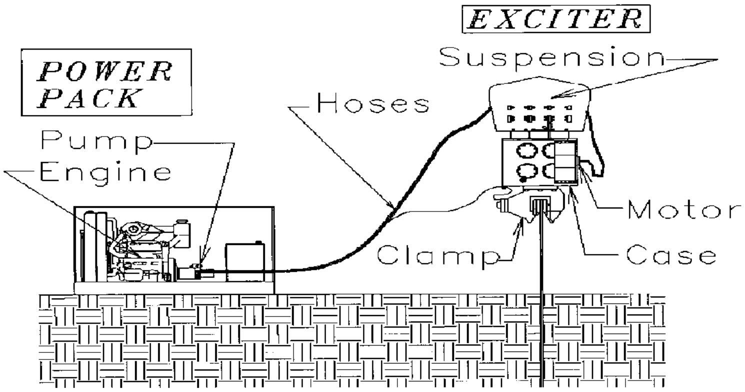


Figure 3-20. Medium frequency vibratory hammer.

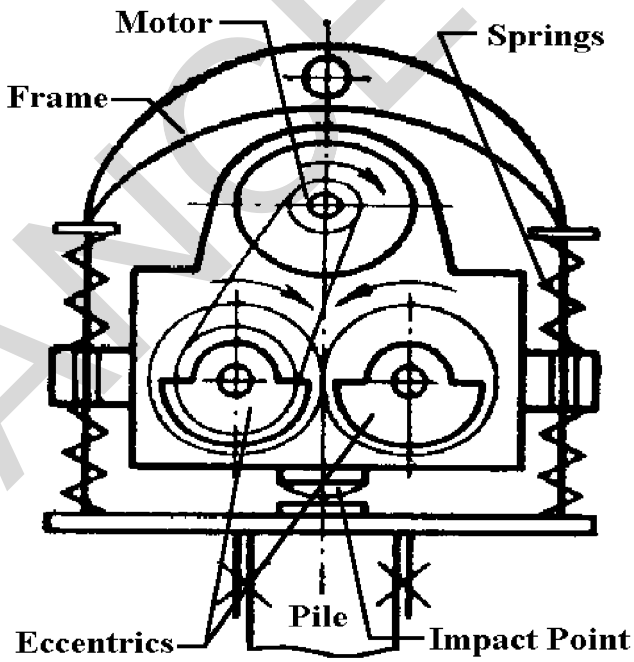


Figure 3-21. Impact-vibration hammer.

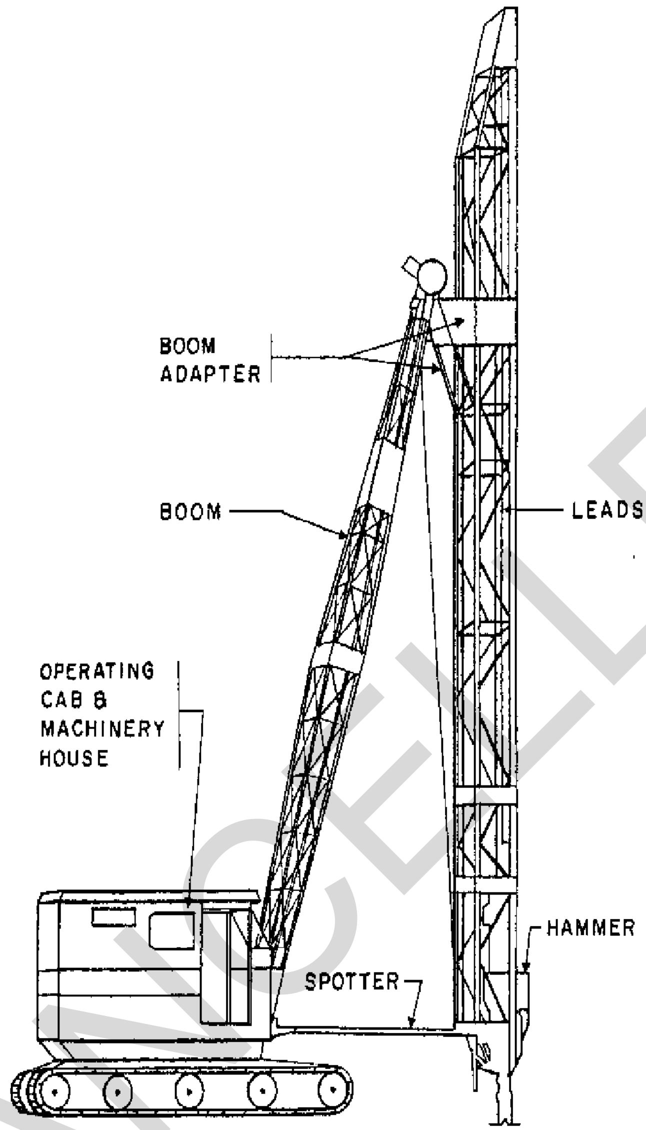


Figure 3-22. Commercial crane pile driving rig.

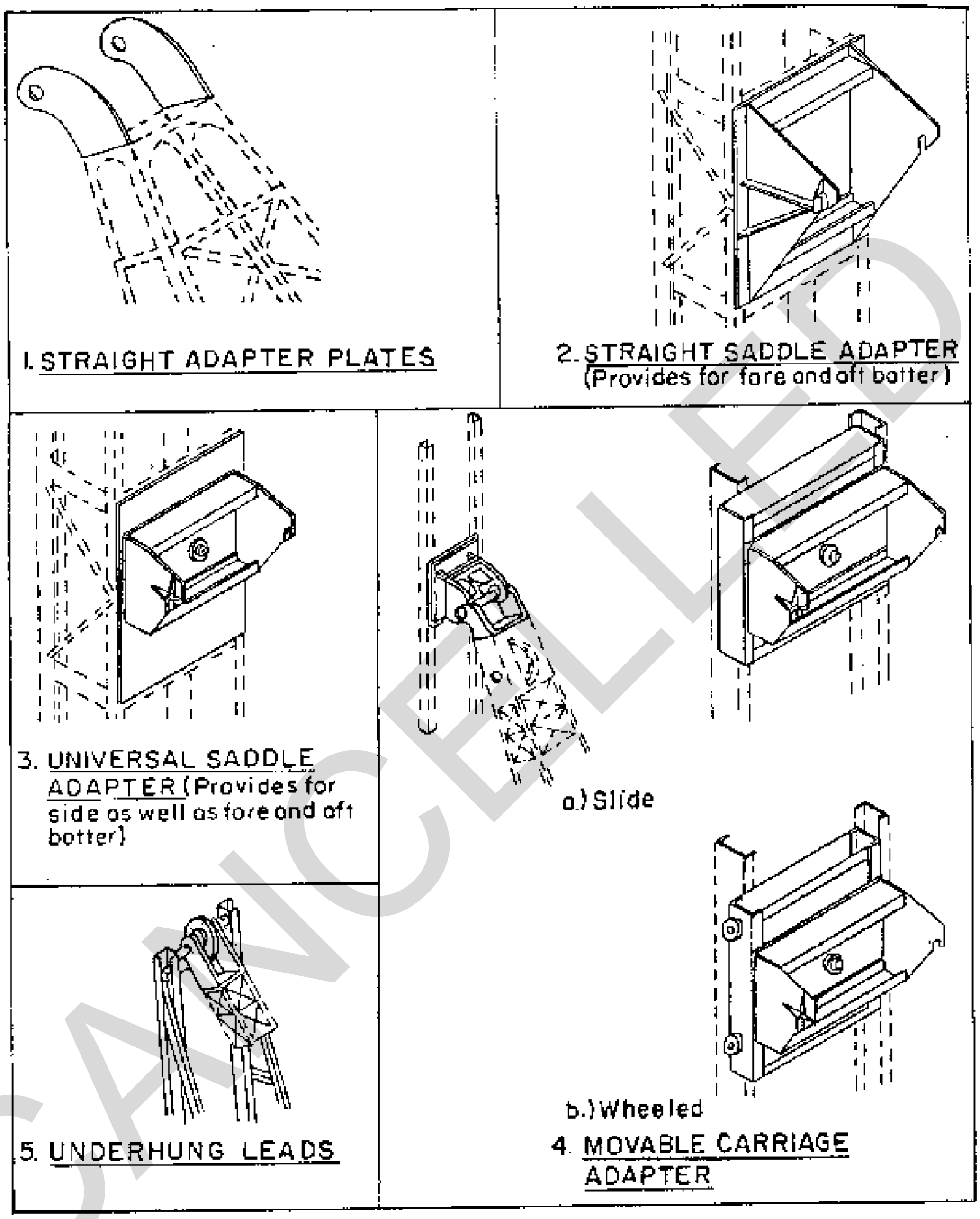


Figure 3-23. Boom point connections.

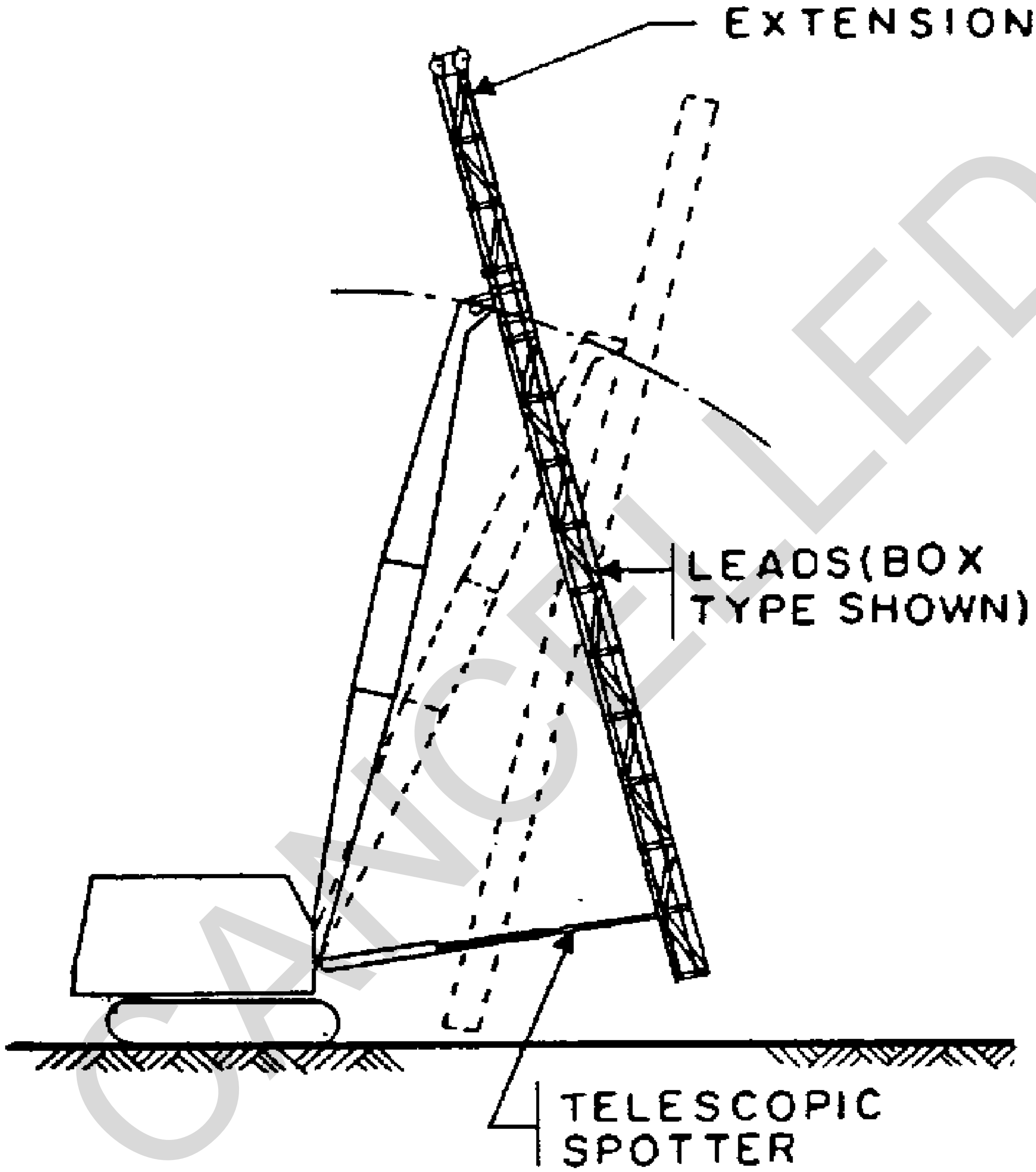


Figure 3-24. Fixed leaders.

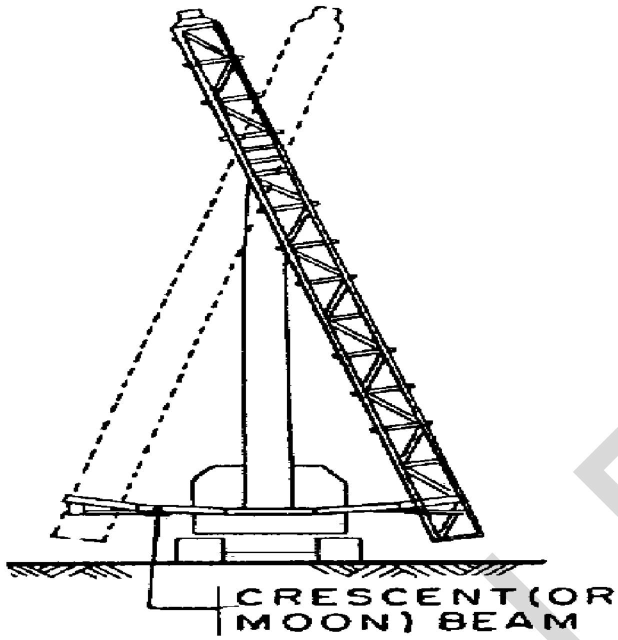


Figure 3-25. Moonbeam spotter leader.

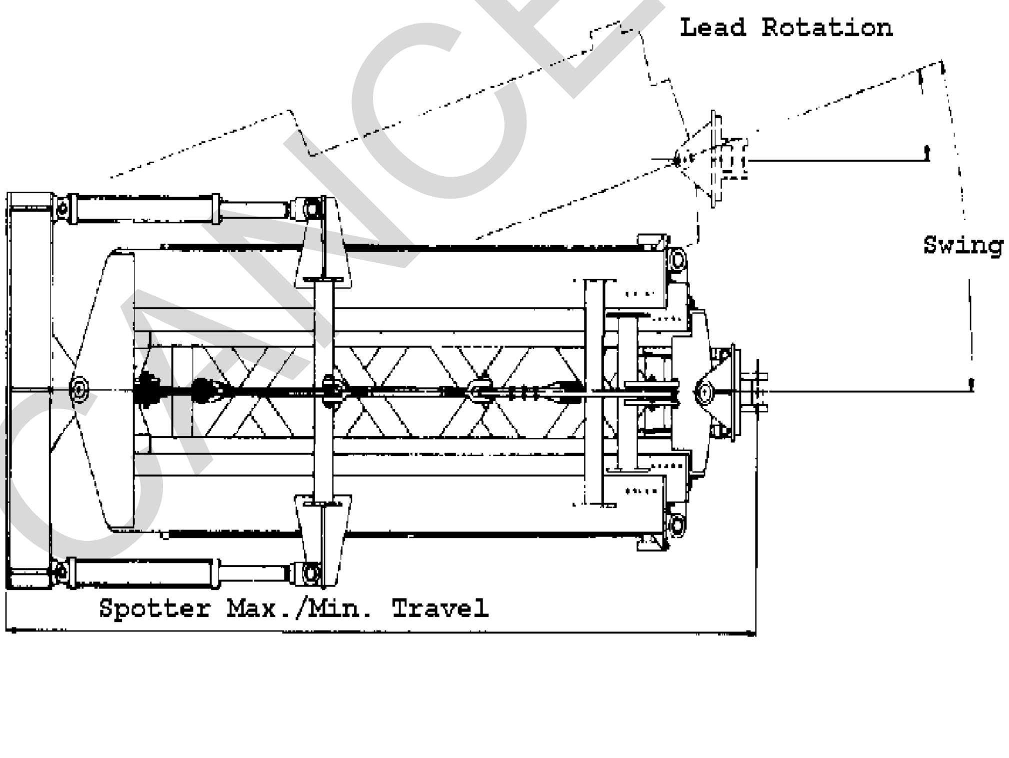
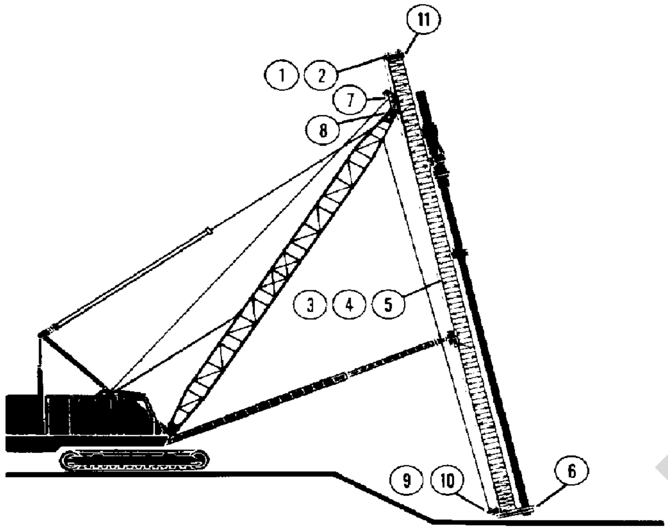


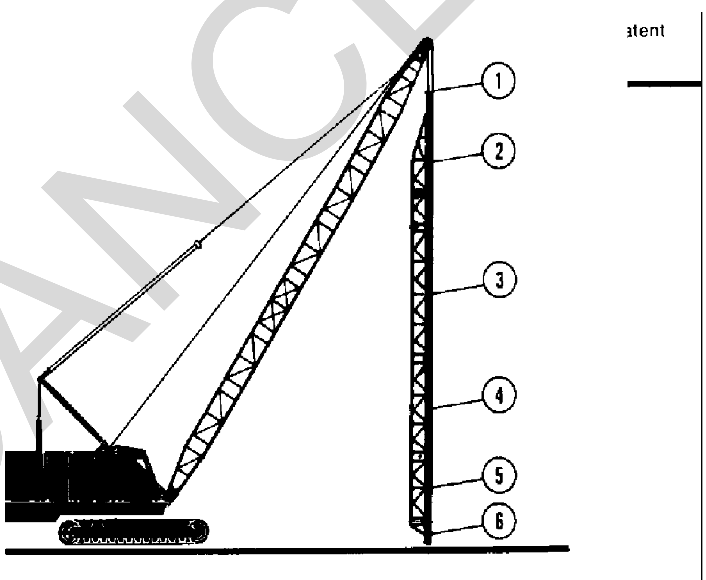
Figure 3-26. Hydraulic spotter.



Parts of Vertical Travel Leaders:

(1, 2, 11) Head section with sheaves
(3, 4, 5) Leader Sections
(6) Pile Gate
(7,8) Boom Point Connection
(9, 10) Connection of wire rope for leader lifting.

Figure 3-27. Vertical travel leaders.



Parts of Swinging Leaders

(1) Taper top head section
(3, 4) Intermediate Section
(6) Stabbing Point
(2, 5) Section Connectors
(5) Bottom Section

Figure 3-28. Swinging leader.

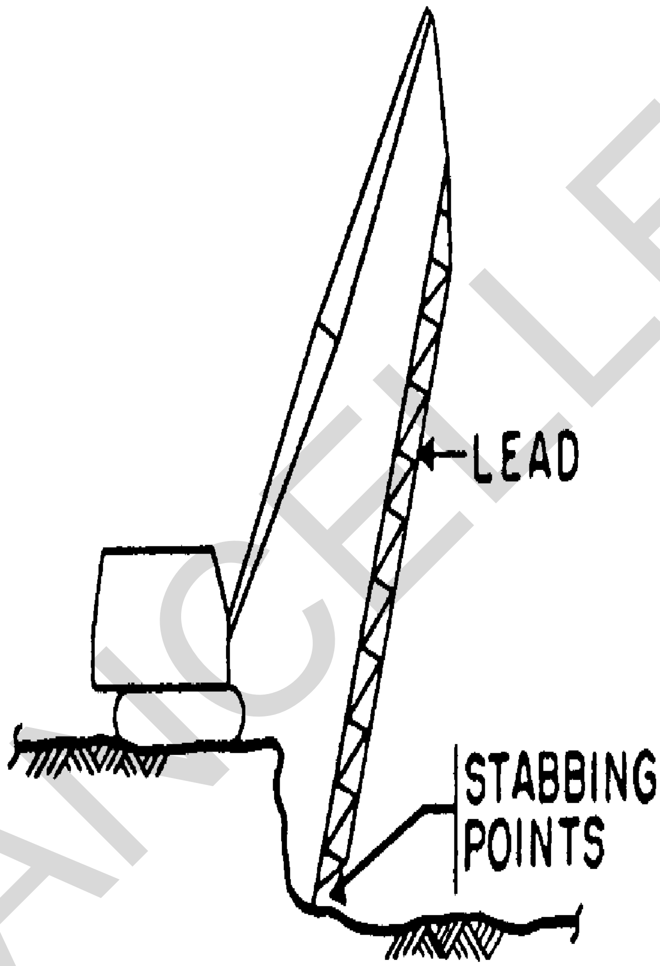


Figure 3-29. Swinging leader with batter piles.

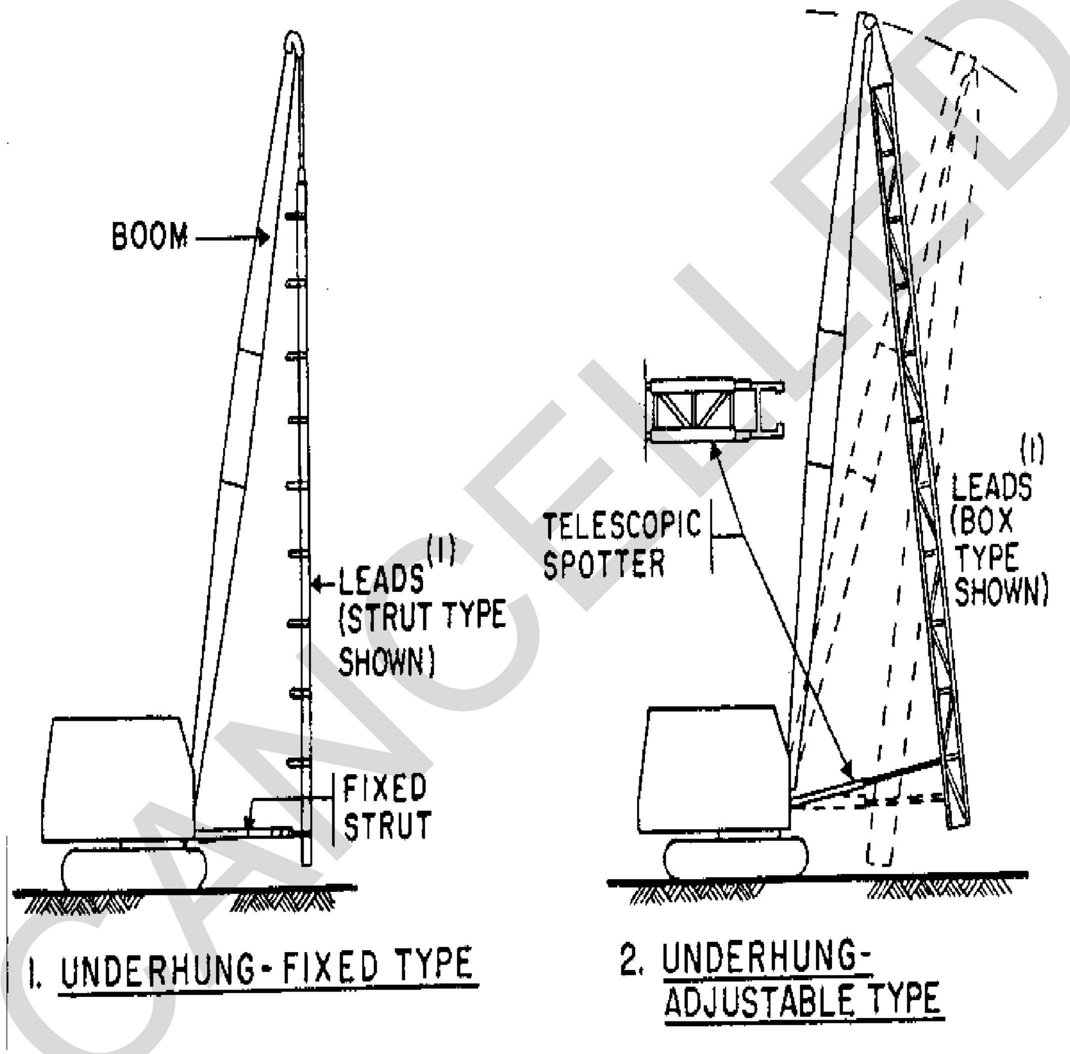


Figure 3-30. Underhung leader.

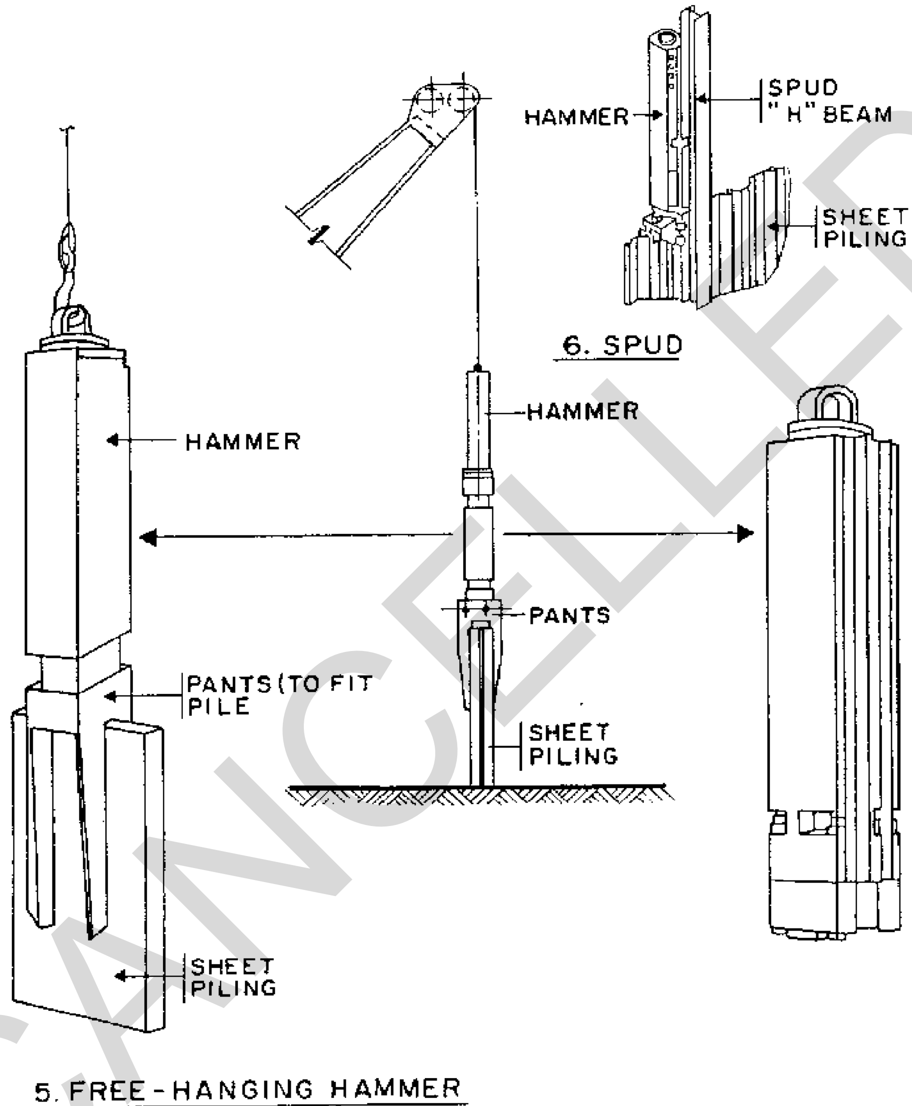


Figure 3-31. Impact hammer with pants.

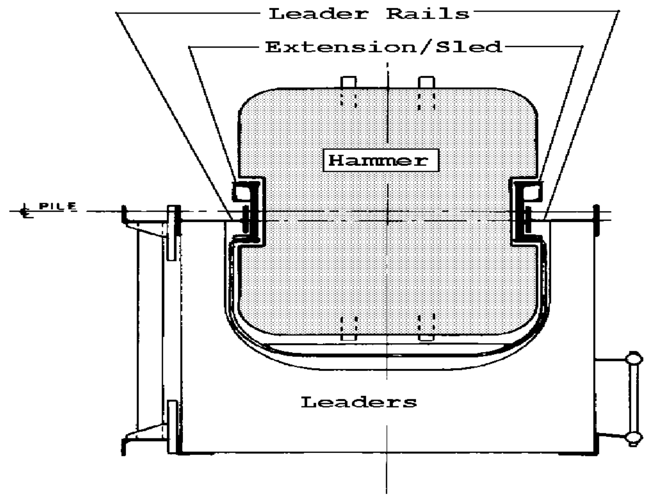


Figure 3-32. Hammer extension (top view).

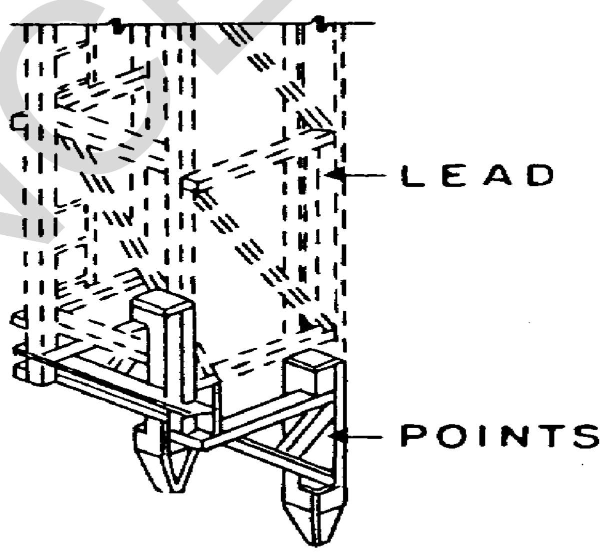


Figure 3-33. Stabbing points.

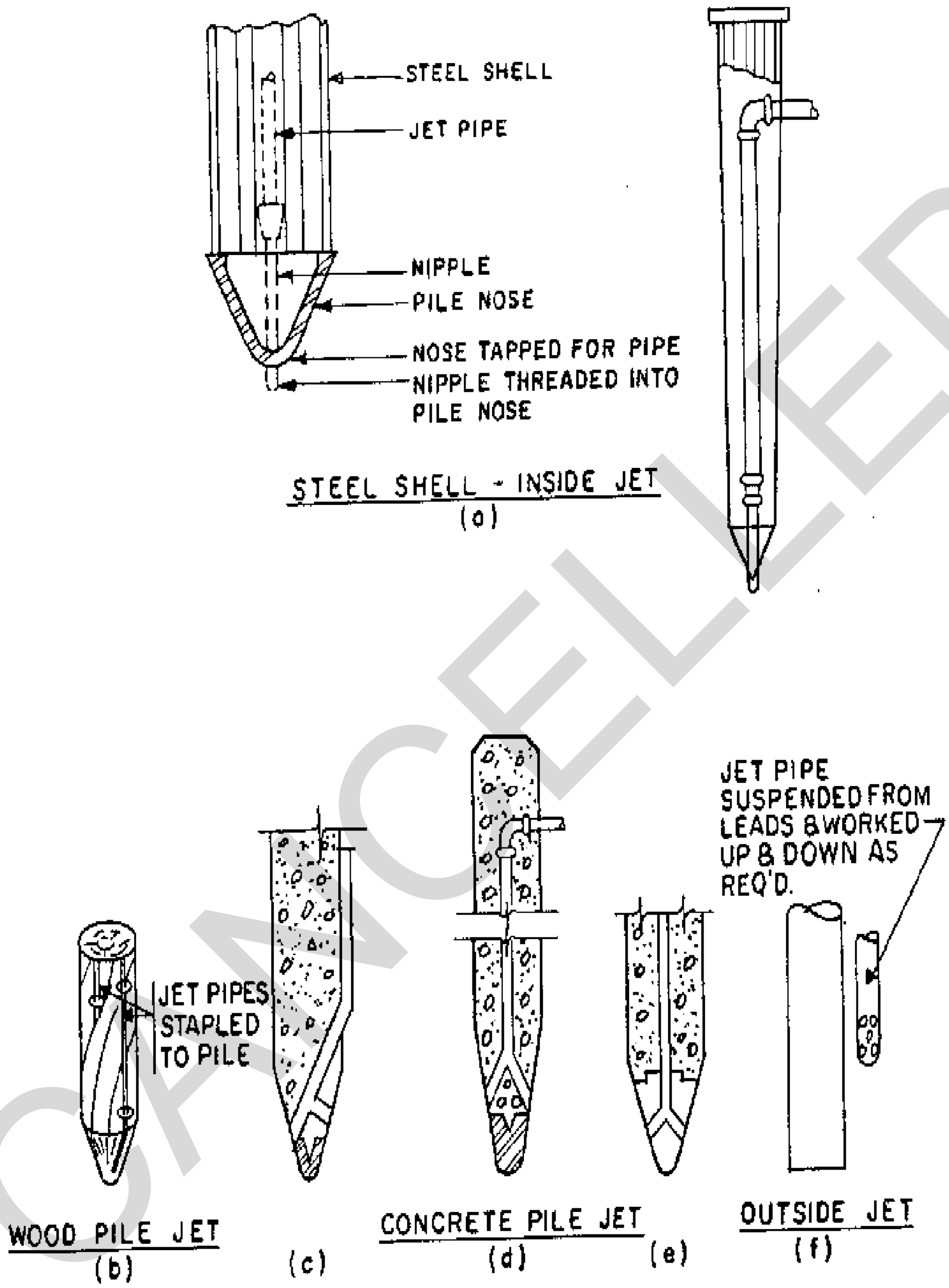


Figure 3-34. Jet configurations.

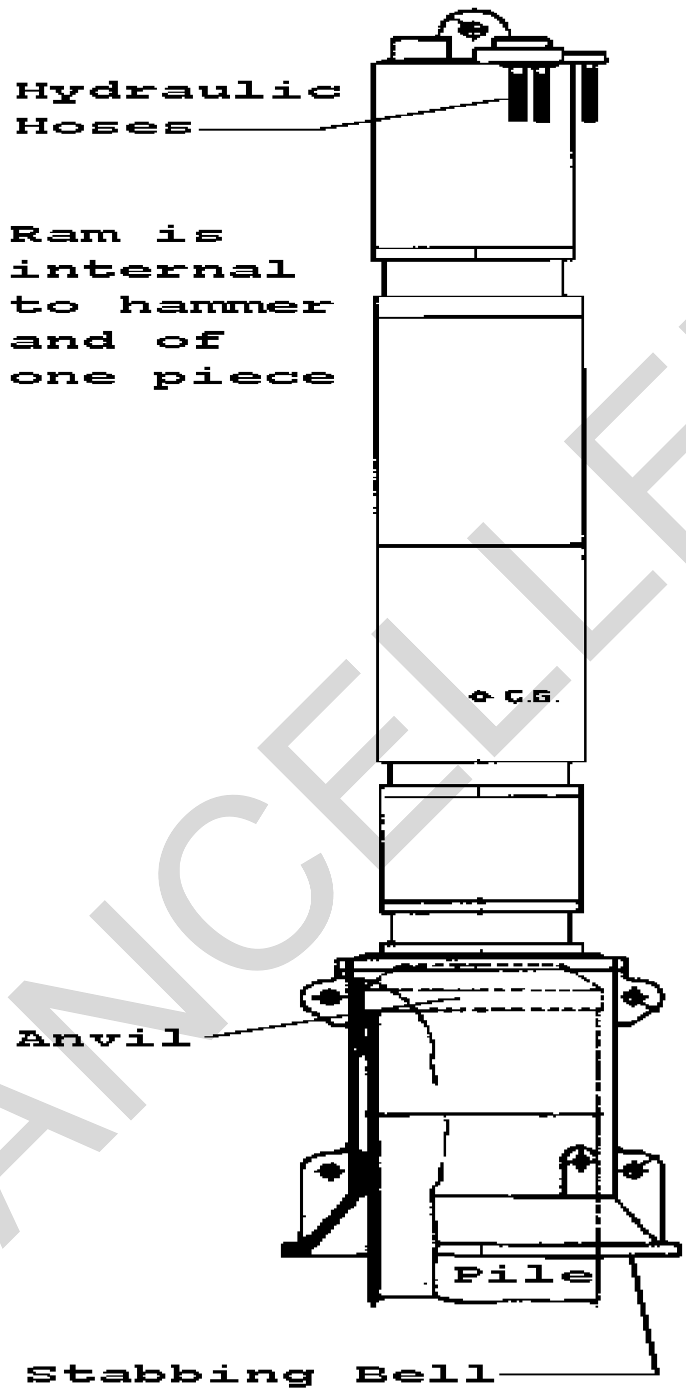


Figure 3-35. Underwater hydraulic impact hammer.

To use a vibratory hammer underwater or, for that matter, under the power pack, it is important to both ensure that water does not seep into the case and to limit the motor case drain pressure relative to the main vibratory case. The pressure of the motor case must be no higher than 2.75 bar (40 psi) above the vibratory case, lest the motor shaft seal blow oil into the vibratory case.

To determine the effective driving depth of the hammer, measure the vertical distance from the motor case drain outlet up to the power pack reservoir. If this distance is less than 9-12m (30' - 40") and the hammer is not submerged underwater, the hammer should operate satisfactorily as it is. Should the depth be in this range and the hammer be submerged, the case vent should be removed and a hose be attached that runs to the surface, and all leaks of the vibratory hammer must be completely stopped and sealed.

For depths greater than 12m (40'), the motor shaft seals must be protected. To do this, remove the case vent and attach an air hose to the exciter case. This hose must be pressurized by an air compressor on the surface, and the case must be pressurized to the pressure of the water surrounding the vibrator at the depth the vibrator is used; however, the vibratory case pressure in any case must not exceed the motor case pressure by more than 1.4-2 bar (20 - 30 psi) or the shaft seal(s) will rupture.

Figure 3-36. Techniques for using hydraulic vibratory hammer underwater.

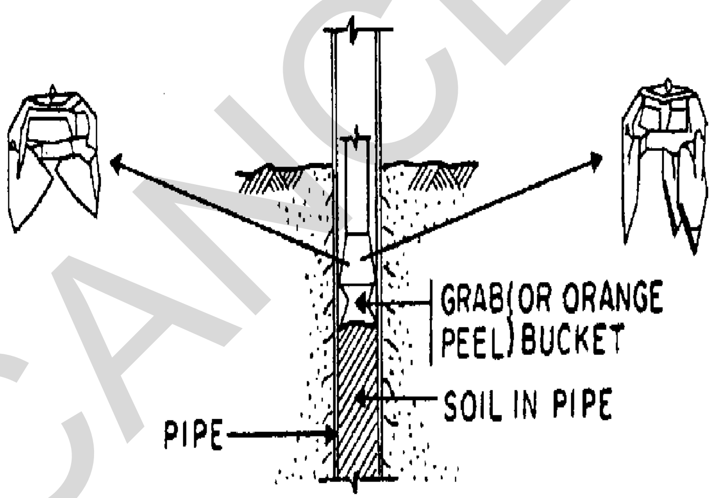


Figure 3-37. Bucket for plug removal.

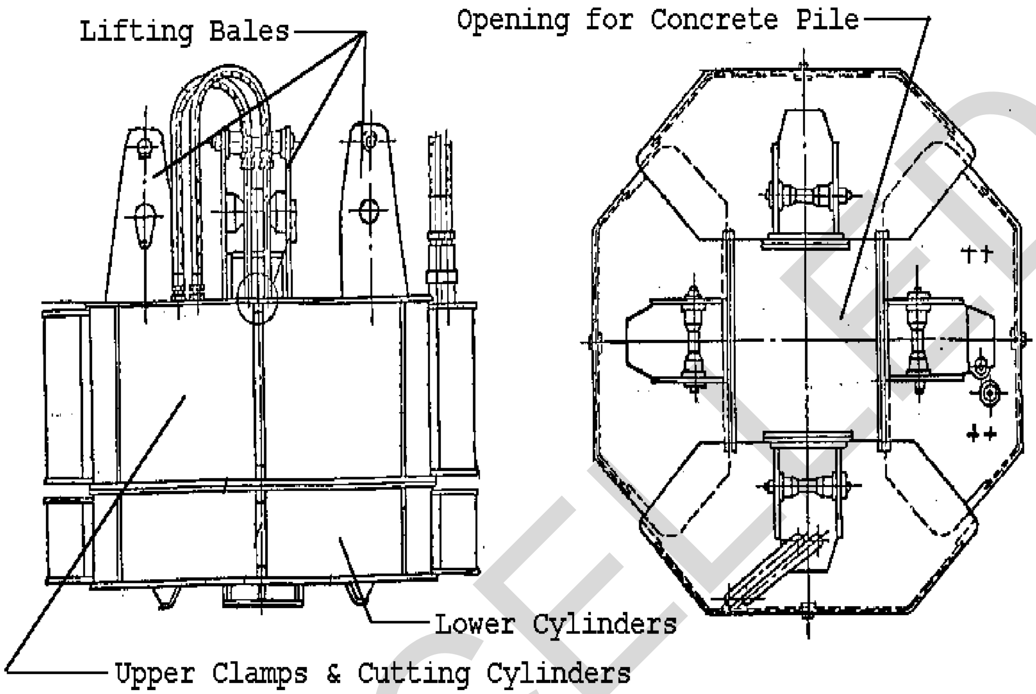


Figure 3-38. Pile beaver.

CHAPTER 4

MONITORING OF PILE INSTALLATION

4-1. EQUIPMENT OPERATION. Successful pile driving is dependent upon both the condition of the equipment and the operating technique used. Failure to observe the proper procedures with the equipment can result in loss of energy imparted to the pile, failure to reach the desired penetration depth, property damage, personal injury, and equipment breakdowns, which lead to downtime, job delays, and contract disputes.

a. Condition of Equipment. Equipment should be maintained and operated in accordance with manufacturer's recommendations. This includes proper preparation of the equipment, correct setup before driving, good operation during driving, and proper transport and storage procedures.

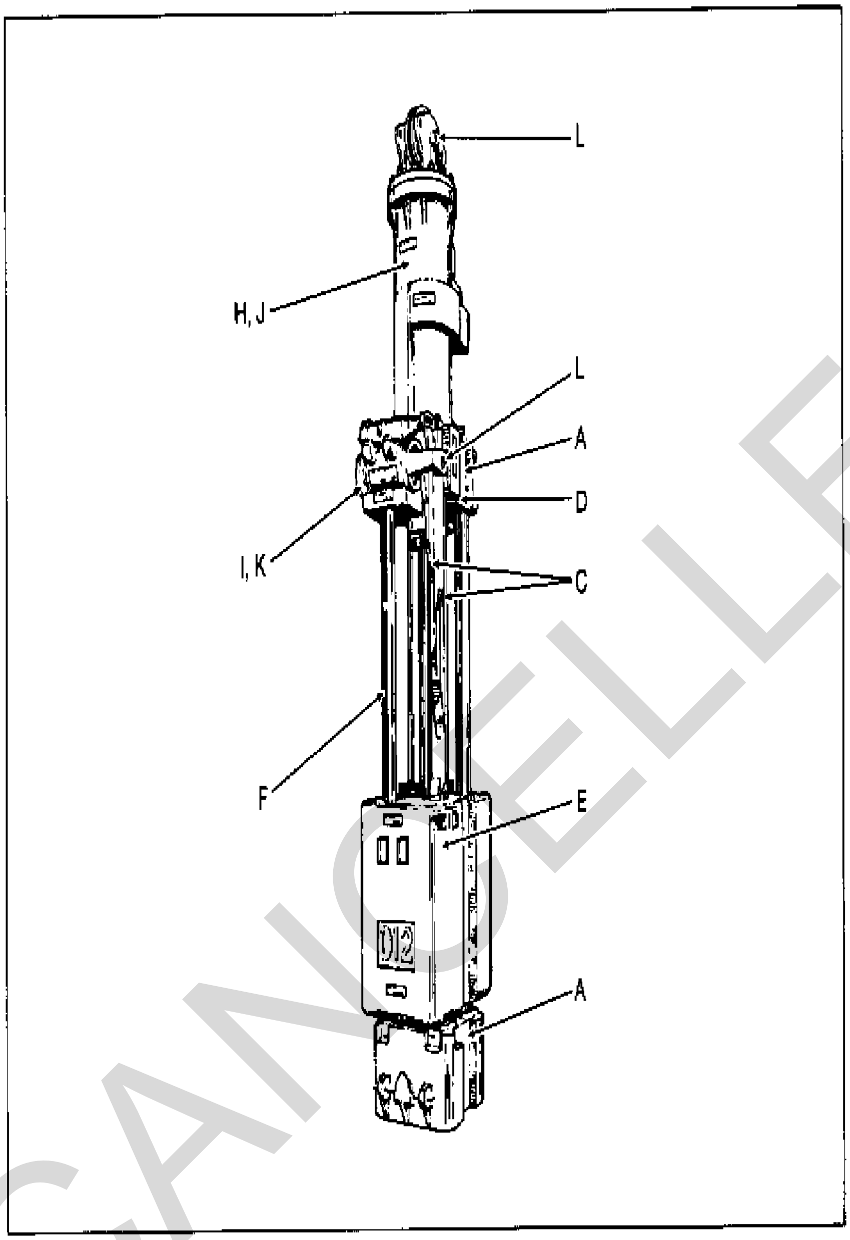
(1) Hammer. The hammer is the element that directly interfaces with the pile. Thus, it transmits and receives the most vibration and impact during driving. Although most pile drivers in use today are relatively simple machines, attention to their basic maintenance is still important. It is also essential that this equipment, in common with most construction equipment, must be operated by personnel with the proper training and experience in the use of the equipment.

(a) Impact Hammers. The most important element in the efficient operation of impact hammers is proper lubrication. All wear surfaces must be kept lubricated by the means provided for either on the machine or from the outside. Typical lubrication points for an air/steam hammer are shown in figure 4-1. Diesel hammers vary widely in their lubrication systems, but the entire cylinder assembly must be lubricated in its full length, whether by grease or oil. Hydraulic impact hammers are similar to air/steam hammers except for the hydraulic system and cylinder(s). In these cases, their maintenance is more like that of a vibratory hammer (see next section). Also, due to the high loads during ram deceleration and pile rebound, impact hammers are subject to dynamic stresses. All fasteners must be kept tight, and parts which are worn, cracked, or broken must be replaced.

(b) Vibratory Hammers. A chart of basic items to watch for with vibratory hammers is shown in table 4-1.

(2) Driving Cap. The driving cap transmits the impact force from the hammer to the pile. Its proper setup and maintenance are central to successful pile driving.

(a) Cushion. Both hammer and pile cushion material should be configured and installed so that there is adequate cushioning for the ram and the pile. Materials which are too hard, or less than complete cushion stacks must be avoided; cushions which are added piece by piece during driving must also be avoided. Cushion material is an expandable item; it degenerates during its life with heat and plastic deformation; it is essential that it is replaced when it is burnt or deformed to its compressive limit. Cushion whose stack height is 75 percent of the original, uncompressed height should be replaced. Also, when piling are designed using the wave equation analysis, it is essential that the cushion configuration assumed during the wave equation analysis be a realistic one in the field and that the configuration assumed during the analysis be replicated in the field. This is especially important with a concrete pile cushion; inadequate cushion may lead to tension cracking during driving and excessive material may lead to unanticipated energy losses. If the pile cushion is plywood and the pile concrete, a new pile cushion should be used with every pile.



Lubrication Points: (a) Jaws
(b)Trip Faces
(c) Slide Bar
(d) Slide Bar Dovetail
(e & f) Columns
(g-j) Air or Steam Ports

Figure 4-1. Lubrication points, air/steam hammer.

Table 4-1. Do’s and do not’s of VIbratory Hammers.

Things that Always Need to be Done

Always store oily rags in containers. If these get into a hydraulic system, there can be problems.

Always remove all tools from unit before starting.

Always be sure that, with hydraulic systems, all pressure is out of the system and that all pressure gauges read zero before working on the hydraulics of the system. The high pressure fluid in hydraulic lines can be very dangerous if released.

Always make sure that all hose hittings or connections are very tight when reassembled. Failure to do so can result in the hoses coming loose, flying around, hydraulic fluid spraying everywhere, and injury or death.

Always make sure that electrical systems are properly grounded during operation. Also, make sure that they are not connected to any power source in anyway and that there is no voltage of any kind in the system before servicing same.

Always make sure that electrical connections and wiring are tight and completely insulated to prevent shock if accidentally touched. This is especially important in waterfront or marine situations; uninsulated wire can result in electrocution.

Always be sure to wear gloves and protective clothing while working on any part of the system. It is even better to wait until the system has cooled down. Hydraulic components, electrical wiring and switchgear, and the engine get very hot during operation.

Always make sure that the pile is firmly gripped by the jaws when clamping.

Things that Never Need to be Done

Never allow unauthorized or unqualified people to operate, maintain, or come within 30 meters of the equipment.

Never allow anyone to stand directly under or within at least 3 meters of the hammer or pile being driven during operation. Failure to do so could result in injury or death caused by falling parts, rocks, or dirt on the hammer.

Never operate the power pack’s engine in a in a closed area. The breathing of the fumes can be fatal.

Never smoke or use an open flame when servicing batteries. Proper ventilation is necessary when charging batteries. On units with a power pack enclosure, all of the doors of the unit must be open during battery charging.

Never smoke when filling fuel tank or hydraulic reservoir or, for that matter while anywhere near exciter, hoses, or power pack. Diesel fuel, gasoline, and hydraulic fluid are all very flammable.

Never adjust or repair the unit while it is in operation, except with the main motor and clamp controls provided for that purpose. If you need to make any other adjustments, shut the entire system down first.

Never store flammable liquids near the engine.

Never unclamp the exciter from the pile when there is any line pull on the suspension or when the hammer is still vibrating.

(b) Anvils and Helmets. Anvils and helmets must be of a construction that will withstand the high loads and stresses of impact pile driving. They should be inspected thoroughly before use. Any helmet or anvil that has any cracks, broken pieces, or thin wall sections that might break in use should be rejected before driving begins. During use, a cap should be inspected frequently for developing cracks or broken or missing pieces, and removed from the job if any of these is discovered during inspection.

(c) Mandrel. Mandrels also transmit high loads. They should be inspected during use for any cracks or other mechanical failures, and these should be repaired before driving is resumed.

b. Hammer Operation. There are several important performance parameters to monitor during driving.

(1) Hammer Stroke. The single most important parameter to look for is the stroke of the ram.

(a) Single Acting Hammers. With single acting hammers of any kind, the only source of energy for the ram on the downstroke is its drop through the gravity field. So it is absolutely essential that its stroke be what is required by the piling being driven, whether that be full or partial stroke. This stroke should be achieved when the pile is at refusal; some hammers (especially diesel hammers) cannot achieve full stroke without the pile energy rebound of refusal driving.

(b) Other Impact Hammers. Although the stroke is not as important with hammers that have assistance on the downstroke, it is important to establish what stroke is required for the particular job.

(c) Monitoring of Stroke. If the ram is visible from the outside of the hammer, then visual monitoring of the stroke is possible. With air/steam hammers, as the ram moves it wipes most of the lubricant off of the columns. The length of this wiped area will determine the stroke of the hammer. With open-end diesel hammers, a striped pole can be mounted on top of the hammer. As the ram raises up, the stroke can be determined by comparing the maximum height of the ram with the highest point of the ram raising.

(2) Speed. Although the hammer speed is a general indicator of the performance of the hammer, except for single-acting diesel hammers, it cannot be used to determine the energy output of the hammers. This is because there are simply too many variables influencing the motion of the ram, especially the variations in energy rebound from the pile. With single-acting diesel hammers, devices exist that can estimate the energy of the hammer based on its blow rate.

(3) Pressure. With external combustion hammers, the pressure of the operating fluid is important in the energy output of the hammers. With single-acting hammers, although no direct correlation can be made between pressure and output energy, monitoring the pressure can be a useful tool in diagnosing problems with the driving system. For other hammers, fluid pressure of the motive fluid or trapped dashpot air is part of the downstroke assistance and is an important factor in hammer energy. For proper readings, any pressure readings should be taken at the hammer by a pressure gauge at the hammer or by remote sensor.

(4) Direct Energy Monitoring. If the ram is not visible, or for more complete information on the energy output of the hammer, devices exist that can determine the energy output of the hammer, either from the hammer itself or after the stress wave has passed through the cushion material and drive cap. These can be either obtained from the manufacturer or third-party vendors and consultants. It is very important that these devices be properly installed, calibrated, and operated by experienced and trained personnel who are very familiar with their use; incorrect installation, calibration, or operation can result in erroneous readings. Complete reports on the use of these devices should be required if they are used.

4-2. PILE INSTALLATION. Installation of the production piles, those piles that are intended to support the structure under construction, will be monitored and records kept to provide data to assess the capacity and quality of the pile foundation.

a. Pile Driving Records. A complete pile driving record will be prepared for each pile driven to its embedment depth. These records provide important data concerning the nature of the soils that are penetrated, provide evidence of any soil disturbance that is a result of driving, and indicate the penetration resistance. The penetration resistance, pile driving analyzer data recorded during driving of indicator and some of the production piles, and recorded later during restrikes, can be analyzed to assess the driving stresses, capacity, and quality of the pile.

(1) Impact Hammers. The make, model, energy, stroke, pile, and hammer cushions should all be recorded on the header portion of the record. The remaining portion of the pile driving record is used to record the penetration resistance (blows applied to the pile within a unit distance such as 1 foot.

(a) A wave equation analysis should be performed if blow counts that are recorded are not similar to those observed during driving and restrike of the indicator piles at the start or prior to construction.

(b) The blow counts should be plotted as a function of depth to determine depths where the soil may be strongest and to indicate if the pile had been driven to the proper embedment depth and bearing stratum.

(c) The average blow count should be computed for each pile and the average of each pile plotted versus time. This plot will indicate if the penetration resistance is changing as additional piles are installed. Increases in the penetration resistance with time may indicate that the soil is getting stronger, perhaps from densification of the soil as a result of driving. Increasing penetration resistance with time may also indicate a temporary drop in pore water pressure as a result of the disturbance of dense sands or stiff clays. Decreases in the penetration resistance with time may indicate driving into weaker soil or perhaps the temporary buildup of excess pore pressures if the soils are loose and fine grained with low permeability.

(d) Abrupt changes in penetration resistance will be investigated to determine the cause. Abrupt changes can indicate a variety of problems that will cause the pile to be rejected. The pile should be replaced if driving damaged the pile. The installation equipment may have to be changed to install a different type of pile and pile length.

(2) Vibratory Drivers. The make, model, weight, dynamic force, frequency or range of frequencies, maximum eccentric moment, and clamping method will be recorded on the form. The remaining portion of the form will be used to record the depth, the time to reach a given depth, and the frequency to reach the depth.

(a) The ultimate piling capacity cannot yet be determined from vibratory driver records. An impact hammer is required to drive the pile to the embedment depth.

(b) A static load test is typically required to verify that vibratory driven piles have adequate pile capacity. Specifications should require a static load test for each type of pile and soil.

(c) Steel H-piles and pipe piles can be easily clamped to a vibratory driver. Concrete and timber piles may require bolts to secure the vibratory driver.

(d) The vibratory driver is nearly always the most efficient tool for extracting a pile and should be used for steel, wood, or concrete. The vibratory driver first breaks the pile loose from the soil and then extracts the pile. A line attached directly to the pile can aid extraction when the pile is too heavy for the driver.

(e) The dynamic force must be sufficient to maintain an adequate amplitude to continue penetration of the soil. This is especially important if the soil is clay. Driving is still possible if the driver is sufficiently large. A rule of thumb for vibratory drivers is a small driver for piles up to 30 feet, medium driver for piles up to 50 feet, and a larger driver for piles over 50 feet in length.

b. Pile Driving Analyzer. The PDA is recommended during pile installation or during some part of the pile installation operation to calibrate wave equation analysis and to evaluate the quality of the driven pile. Calibration of wave equation analysis increases reliability of the bearing graph and driving stresses calculated by wave equation analysis.

(1) PDA analysis will be performed at the start of any significant project and later during pile installation.

(a) The number of static load tests can be reduced to just one or two when PDA is performed. More static load tests are required for variable or erratic soils.

(b) The PDA can be expected to provide good estimates of pile capacity in uncemented sands.

(c) The PDA can provide good estimates of pile capacity in silts or clays, but careful analysis is required because of high damping forces.

(2) The pile is expected to have sufficient capacity if the bearing graph calculated by wave equation analysis indicates an ultimate bearing capacity of at least twice the design load for the measured penetration resistance at the embedment depth.

(a) Sample production piles will be tested by PDA to ensure that the pile foundation will have sufficient capacity.

(b) PDA will be performed if the pile driving record indicates a significant reduction in pile capacity so that there is doubt that the piles can adequately support the structure.

(3) Results of the PDA and wave equation analysis performed by the PDA technician should indicate the load-displacement behavior of the pile and provide a value of the pile settlement at the design load.

(a) The settlement for the design load should usually be less than 0.5 inch.

(b) Calculation of load-displacement behavior with PDA equipment requires calibration with a static load test.

(4) Observation of the wave forms on the oscilloscope of the PDA equipment by the PDA technician will be performed to indicate the quality of the pile and to determine if any damage occurred to the pile as a result of driving.

(a) The PDA data will tell if there is any probable damage to the pile top.

(b) The PDA will show the effect of changes in the helmet, cushions, or stresses induced at the pile top.

(5) PDA data will permit analysis of hammer performance.

(a) The analysis can show the effect of reduced throttle settings or steam pressure.

(b) The analysis can calculate the maximum energy actually transmitted to the pile. This energy should be compared with the rated hammer energy to determine the efficiency of the hammer assembly. Low efficiencies indicate problems in the driving system and that maintenance is required.

(c) The analysis can assist in determining where the energy losses are occurring in the driving system.

c. Restrikes. Restrikes are useful for determining effects of soil freeze or relaxation and to evaluate long-term pile capacity. Restrikes of previously driven piles should be accomplished using a swinging leader to minimize time required for positioning the rig and pile driving system.

(1) Restrikes are recommended during installation of the production piles if restrikes performed prior to or at the start of pile installation indicated more than 25-percent change in the penetration resistance after a delay exceeding 16 hours.

(2) Restrikes are required when pile driving records indicate significant changes in penetration resistance, i.e., > 25 percent compared to those piles driven earlier.

(3) Restrikes are required when pile driving records indicate penetration resistances that are not compatible with adequate pile capacity.

4-3. PREVENTION OF PILE DAMAGE DURING DRIVING. Damage can be prevented by eliminating overstressing (overdriving) of the pile. Overstressing occurs when driving stresses exceed the allowable limits.

a. Pile Damage. Overstressing is frequently attributed to using a hammer too large for the pile being driven, misalignment of the pile and hammer, material failure within the drive cap such as loss or insufficient cushion, or malfunction of the equipment.

(1) General Indicators of Damage. The pile is probably damaged if any of the indicators given in table 4-2a are observed.

(a) Hollow piles that can be inspected after driving may show overdriving from crushing or distortion of the pile material.

(b) Any piles suspected of being damaged will be promptly evaluated to determine its effect on the overall foundation design.

(c) Repetitive damage to piles may require modification of the installation equipment or installation method.

(2) Specific Indicators of Damage. Table 4-2b illustrates types of damages observed in timber, concrete, and steel pile.

Table 4-2. Indicators of pile damage.

Observation	Pile Damage
General Indicators of Damage	
A sudden increase, decrease, or irregular penetration resistance	Pile breakage if the soil formation has not changed from that when previous piles had been driven.
Drifting of the pile off location as observed at the ground surface.	Pile breakage.
A sudden lateral snap of the pile head.	Pile breakage from bending.
Damage observed at the head.	Similar damage may occur at the tip.
Two or more cycles of decreasing and increasing penetration resistance.	Progressive crushing of the tip.
Vibration and/or noise of Previously driven piles.	Interference with the pile being driven.
Types of Observed Damage	
Type of Pile	Damage
Timber	Brooming at the tip or head, splintering, or breaks.
Concrete	Spalling, transverse cracks
Steel	Bending, reduction in cross-section.

b. Prevention of Damage. Overstressing the pile can be prevented by determining how much driving the pile can tolerate. This is accomplished by completing a wave equation analysis to determine driving stresses. These stresses and corresponding penetration resistances will be avoided during pile installation. Experience on how hard the pile can be driven in the field without damage can also assist in avoiding damage.

(1) Wave Equation Analysis. The wave equation analysis calculates driving stresses which, when compared with allowable stresses, can determine when overstressing will occur for the selected pile driving system and the pile.

(a) A preliminary wave equation analysis should be made as part of the foundation design procedure to estimate driving stresses for the selected pile foundation, pile driving system, and cushions. The pile can be expected to be overstressed and to be damaged if the driving energy of the selected hammer is exceeded or thickness of pile cushions is reduced.

(b) Wave equation analysis will be performed as part of driving indicator piles with the PDA. Results of this analysis will confirm the selection of the pile driving system and provide guidance on the limits of the driving energy to be sure that driving stresses will be less than the allowable stresses.

(2) Good Driving Practice. All driven piles are subject to driving limitations.

(a) The piles will be properly aligned with the pile hammer and driving energy limited to maintain driving stresses within allowable limits.

(b) Cushions will be maintained at the thickness determined from results of driving of indicator piles with the PDA.

(c) Timber piles are especially vulnerable to brooming and splitting near the pile top or tip when driven. Driving should stop immediately when hard driving is encountered as indicated by a sudden increase in the penetration resistance or if the pile is observed to drift off location. A vibratory driver can be substituted for an impact hammer to eliminate damage from hard driving.

(d) The driveability of concrete piles is often limited by the tensile strength of the pile, especially long prestressed concrete piles. Concrete piles should be driven initially with low energy that is about 50 percent of the maximum energy to be applied to the pile. The maximum driving energy is applied when the pile is nearly at the embedment depth. This guidance therefore requires that concrete piles should be driven with hammers of variable energy to safeguard the integrity of the pile.

(e) Steel piles, especially open-end pipe and H-piles, are capable of hard driving. Pile tips may be protected with driving shoes, but prolonged hard driving should be avoided.

(f) Piles suspected of being damaged should be extracted and replaced. Vibratory drivers are efficient extractors of piles if the pile can be clamped to the driver.

(g) Pile driving should cease immediately when sudden increases in penetration resistance are observed to avoid overdriving. Timber piles are especially vulnerable to brooming or splitting at the top or tip of the pile when hard driving occurs. Prolonged hard driving will be avoided.

(3) Restrike of Selected Piles. Selected piles will be restruck with PDA equipment to measure the quality of the driven piles and to assess pile capacity.

(a) Piles that should be restruck include those that have some sign of possible malfunction, but not judged serious enough to extract. Examples include penetration resistance that is not as high as expected and where the pile capacity is in doubt, unexplained changes in penetration resistance, penetration resistance not as high as previously driven piles, and piles with some damage at the pile head.

(b) Driving stresses evaluated by the analysis during restrike will not exceed the yield or ultimate strength of the pile material. This provides a slight margin of safety against damage because the duration of the peak of the driving stresses is short.

(c) The quality of restruck piles will be evaluated by the PDA technician and made a part of the quality assurance record.

4-4. COMPLICATIONS. Driving of the production piles may lead to complications that could compromise the capacity of the foundation and interfere with the operation of adjacent structures. These complications can be caused by penetration resistances that are either too low or too high when driven to the embedment depth, by ground movement, and by environmental problems. Refer to paragraph 4-3 for solution to problems with pile integrity.

a. Problems with Low Resistance. The pile foundation will not have sufficient capacity if the penetration resistances are too low. Abrupt unexpected reductions in the penetration resistance may indicate a damage pile. Guidance for the solution of this problem is considered in paragraph 4-3. Other causes of low penetration resistance include soil freeze, soils with low strength, and driving into a landfill. Materials in landfills will usually be poorly compacted.

(1) Driving of Initial Piles. If the first production piles driven in a group have penetration resistances that are too low, then one must first check to be sure that the hammer and driving energy are adequate and similar to those used for driving near indicator piles prior to or at the start of construction. PDA data must be obtained if the equipment is available, and driving stresses, capacity, and pile quality must be checked. PDA equipment should be available during the driving of the first piles and used to assist the installation.

(a) If pile capacity is not adequate at the embedment depth as indicated by low penetration resistance or the PDA, but the pile is not damaged and driving stresses are satisfactory, several additional piles may be driven as specified in the contract documents. The first piles driven in a group often have penetration resistances that are lower than those driven later. Driving tends to compact the soil and to increase the penetration resistance.

(b) Plot the average penetration resistances of the piles that were driven as a function of time to see if the penetration resistances had increased with time and have reached a satisfactory level. The first piles that were driven in a group should be restruck after driving additional piles to see if the penetration resistances and pile capacity have increased to a satisfactory level.

(c) If the penetration resistances and pile capacity have not increased to an adequate level, then other options should be considered. Such options are driving the piles to a deeper embedment depth, driving additional piles to reduce the spacing between the piles and increase the number of piles in the group, or use a vibratory driver to compact the soil. A cost and efficiency comparison should be made between these options, and the most cost effective option should be selected first.

(2) Piles Driven Later. If piles driven later in a group do not have sufficient penetration resistance, then the cause may be from the effects of soil freeze, from weak soils such as loose sands or soft clays, or from subsurface voids.

(a) If the penetration resistance of the pile remains low when driven, the pile should be restruck after a 1-, 2-, or 5-day delay to determine if soil freeze effects are present. The penetration resistance of restruck piles should be plotted as a function of time to determine the rate of increase in penetration resistance. Adequate capacity may develop after a sufficient delay following pile installation. Soils subject to freeze include loose to medium dense sands, silts, and clays.

(b) If the penetration resistance is low, although driving energy is adequate, there is no evidence of pile damage. If restrike does not provide larger blow counts, then the soils may be weaker than anticipated from results of the exploration program. Driving of additional piles or driving to a deeper embedment depth are options that may be considered.

(c) If the penetration resistance suddenly drops and pile damage is not suspected, the pile tip may have penetrated a subsurface void. The penetration resistance should suddenly increase when the bottom of the void is reached. Driving of piles at nearby locations will aid the determination of the lateral extent of this void. A possible complication with subsurface voids is that they may become larger with time and could eventually compromise the integrity of the foundation. Subsurface voids should be filled with a cement-bentonite grout when they are found. Pile installation should continue at another location of the site until the grout has had a chance to set. If the void in question is an underground cavern (as can occur in limestone), grouting may not be practical because of the size of the void.

b. Problems with High Resistance. Encountered resistance that is too high may be caused by inadequate driving equipment. If the driving equipment, driving energy, and piles are found adequate, then soil relaxation may be a cause of the problem. Relaxation may occur in dense, fine, submerged sand, inorganic silt, or a stiff, fissured clay or by driving a point bearing pile into friable shale or a clay stone. High penetration resistances may also be caused by obstructions such as cobbles or boulders, an unexpected dense or stiff soil, large bodies in landfills, interference with adjacent piles, or bedrock.

(1) Driving Equipment and Piles. Driving equipment that does not have an adequate hammer and driving energy to install the piles will cause high penetration resistances with little or no set of the pile.

(a) The size and type of pile should be checked to be sure that the proper pile is being installed to support the structure.

(b) Contract specifications will provide the minimum driving energy permitted for the work. The driving energy should exceed the minimum permitted energy.

(c) If the PDA indicates that driving stresses are low, efficiency of the hammer is adequate, and pile quality is adequate, energy of the hammer should be increased and driving continued. If the efficiency of the hammer is low, then driving equipment requires maintenance.

(d) If the PDA indicates that driving stresses are high and near the allowable limits and hammer efficiency and energy is adequate, then soil relaxation or obstructions may exist. If soil relaxation or obstructions such as boulders or large bodies are not found, then the soil through which the pile is being driven may be of high strength and assisted installation described in paragraph 3-4 may be required to reach the design embedment depth. Soil data recorded during exploration should be checked to determine if the designers had anticipated high-strength soil at depths above the design embedment depth. Adequate bearing capacity may be obtainable at a lower embedment depth reducing the required length of the pile and save on installation costs.

(2) Relaxation. Penetration resistances that continue to increase as the pile is driven may be caused by soil that increases in strength with increasing depth. Driving may also reduce the pore water pressures and cause the soil strength to increase for a time.

(a) Small displacement piles such as H-piles or open end pipe piles are recommended if relaxation effects are significant. Open-end pipe piles can be cleaned periodically during driving to minimize soil displacement. The pipe pile, after it had been cleaned, can also be filled with an internal core of reinforcement steel and concrete to achieve adequate pile capacity. Consideration may be given to withdrawing the pipe pile as the concrete is being placed if the pipe is not needed to support the concrete or to provide adequate pile capacity.

(b) Driving of the pile should continue until driving stresses reach the allowable limit or until the pile is driven to the design embedment depth. The pile should be restruck after 1 or 2 days, even if the pile had been driven to the embedment depth, to determine if the soil will relax and the soil strength will be reduced following equalization of pore pressures. If the penetration resistance had decreased following the delay, driving can continue until the pile is driven to the design embedment depth or until the driving stresses again exceed the capacity of the pile or the driving system.

(c) Soil strength gain as a result of relaxation can exceed the capacity of the driving system to install the pile. Delays of 1 or more days may be required to drive the pile to the design embedment depth if the capacity of the driving system and/or pile are not increased. The driving energy should be increased until driving stresses reach the allowable limit to optimize driving efficiency. The capacity of the pile may also be increased to avoid excessive overstresses by increasing wall thickness of H-piles or steel open-end piles.

(d) Assisted installation by specialized operations described in paragraph 3-4 is an option that can be used to install piles where relaxation effects are significant.

(e) Several production piles should be restruck after a long term delay such as 1, 2, 5, and 10 days to be certain that long term equalization of pore pressures will not reduce the penetration resistance and pile capacity to excessively low levels. The decrease in penetration resistance recorded during the restrikes should be plotted with time to determine the rate that relaxation occurs. If the decrease in penetration resistance is negligible after 5 or 10 days, then no later restrikes should be necessary.

(3) Obstructions. Obstructions such as cobbles, boulders, and large bodies in landfills will cause significant increases in the penetration resistance leading to refusal of the pile to be driven further and/or the pile can be deflected by the obstruction. The pile will be damaged if driving stresses exceed the allowable limits.

(a) Obstructions near the ground surface are evident by drift of the pile or by abrupt increases in the penetration resistance. Deep obstructions are indicated by an abrupt increase in the penetration resistance or by a significant difference between tip elevations of adjacent piles when the pile had been driven to refusal.

(b) Deep obstructions can deflect the pile without any apparent effect to the exposed portion of the pile above the ground surface. If the deflection breaks the pile, then the penetration resistance may be decreased abruptly. If the deflection caused by the obstruction bends the pile, then the pile capacity may not be adequate. Steel piles can bend through large angles approaching 180 degrees so that the tip is driven toward the ground surface and may even break the surface.

(c) Obstructions may be removed by assisted installation or may be pierced with a spud. A spud is a mandrel, heavy steel pipe or H-pile section driven to provide a pilot hole. The spud is withdrawn and the pile inserted into the hole and driven to the embedment depth.

(d) A production pile may also be pulled when difficult driving is encountered and redriven. However, skin friction may be reduced causing these piles to be unacceptable friction piles.

c. Problems with Ground Movement. Displacement piles such as timber, precast concrete, and closed-end pipe piles displace the ground as they are driven to the embedment depth. Driving these piles in dense sands or saturated cohesive soils will probably cause heave of the ground surface around the driven pile.

(1) Heaved Piles. Piles that heave should be redriven to their original embedment elevation, but

redriving should not begin until all heave had occurred. The penetration resistance required to redrive the piles to the embedment elevation should be recorded and, any significant change in the penetration resistance should be noted.

(2) Adjacent Structures. Increases in surface elevation exceeding 0.5 inch may cause damage to adjacent structures. Therefore, the effect of ground heave from pile installation should be observed on adjacent structures.

(a) The increase in the surface elevation from ground heave should be measured and recorded.

(b) Ground heave near adjacent structures should be controlled if this heave exceeds 0.5 inch. Methods of controlling ground heave include pre-excavation, increasing pile spacing, or by installation of small displacement piles.

d. Problems with the Environment. All environmental requirements will be fully addressed prior to construction and necessary approvals fully documented.

(1) Vibrations and Noise. Pile driving operations produce considerable noise that may adversely influence the well-being of inhabitants of the area.

(a) Vibrations can damage nearby structures. Structures adjacent to the construction will be monitored to assess their integrity prior to and during construction. Assessment of the integrity of adjacent structures is required prior to construction to determine the initial status of the structures.

(b) Noise can interfere with the quality of life and cause hearing loss to people. High noise levels that are part of an impact hammer pile driving operation can and frequently do prevent installation of driven piles where people can be adversely affected. People within range of damaging noise levels will be protected with safety measures. Vibratory drivers produce much lower noise levels than impact hammers and should be used where noise can be harmful.

(2) Hazardous Materials and Gases. Some construction sites may be located in areas that had been used for landfills, storage depots, or contain natural gases.

(a) The environmental requirements for the site must have been fully addressed prior to construction. Therefore, this is a complication that is not expected, and this situation should not occur. This is an important reason for thorough exploration prior to construction.

(b) If hazardous materials or gases are suspected during pile installation, then the construction activity will be delayed until an environmental assessment can be completed to check this complication. Hazardous materials or gases can become evident when piles are extracted or when the soil collected in hollow open-end piles during driving is excavated.

(3) Groundwater. The exploration program is expected to determine the elevation of the phreatic surface and any evidence of artesian and perched water.

4-5. PERMAFROST AREAS. Frozen soil often has high strength similar to rock. Installation of piles with impact hammers or even vibratory drivers may be impractical and alternative installation methods are required.

a. Site Preparation. The site should be prepared to promote equipment mobility and access at a later time in case other work is necessary.

(1) Disturbance of the ground surface during the construction season frequently increases the depth of thaw. This causes the surface to loose trafficability. The ground surface should be covered with a blanket of gravel or broken stone and work performed from on top of this blanket.

(2) Access to all of the equipment should be limited to the immediate vicinity of the area where the installation is to occur. This area should be prepared so that access of equipment to the area is permanent in case work is required later. For example, maintenance work will probably be required at some later date.

b. Method of Installation. Piles in permafrost areas are not driven to resistance, but rather to some specified tip elevation. Refreeze of the soil around the pile must occur as quickly as practical to develop the required pile capacity. This capacity comes from adherence of the soil to the pile and must support the downward loads and have sufficient resistance to frost heave. Methods of installing piles that can accomplish adequate pile capacity is by drilling or by steam jetting.

(1) Drilling. This method consists of the dropping of pile sections into a boring hole that is made dry with an auger or with a rotary drill and slurry.

(a) Truck mounted augers are usually applicable in silts, clays, and some sands. A rotary drill or local prethawing may be required in coarse sands and in soils containing cobbles.

(b) The excavation after placing the pile sections is backfilled with a soil-water slurry that bonds to the pile on freezing. The slurry should be placed in the hole at near freezing temperature to minimize refreezing time.

(2) Steam Jetting. This method consists of thawing a vertical shaft of soil by a steam jet, pressing or driving the pile into the shaft of thawed soil, and allowing the thawed soil to refreeze.

(a) A 0.75 to 1.25 inch pipe, open or slightly crimped at the point to give better jetting action is used with steam pressures from 100 to 200 psi. A chisel bit may facilitate penetration in gravel.

(b) Water should be added to the jetted shaft in dry soil to promote thawing.

(c) If the thawed shaft of material is of small diameter, then the steam jets should be worked alongside the pile as the pile is advanced down the thawed shaft.

(d) The thawed volume of soil should be minimized to reduce refreezing time.

(3) Seating the Pile. The tip of the pile section should be seated firmly on the bottom of the excavation or thawed shaft of soil to obtain partial capacity and to be able to support the weight of the construction while the adfreeze is developing between the soil and the pile.

CHAPTER 5

PILE DRIVING IN CORAL SANDS (CASE HISTORY)

5-1. BACKGROUND. A very specific yet a very important application of pile driving equipment and driven piles is discussed, namely the application of driven piles to coral sand and calcareous soils. To illustrate the potential problems of such installations, a case history of the installation of piles for a drydock at the Kwajalein Atoll will be discussed. Additional details concerning this project, along with a bibliography, can be found in the U.S. Army Engineer Waterways Experiment Station's Miscellaneous Paper GL-92-23, "Kwajalein Drydock Pile Foundation Analysis."

a. Overview of Calcareous Soils. Calcareous soils are some of the most challenging types of soils for the design and installation of piling. Because they frequently appear in areas where offshore oil is found (i.e., southeast Asia, the Persian Gulf, Australia, etc.), a great deal of research has been done on these soils. Because of the complex nature of these soils and the variable way in which they are formulated, their properties are complex and not as well quantified as other types of soils.

(1) Definition and Origin. Calcareous soils are those which are composed of primarily sand size particles of calcium carbonate, which may be indurated to varying degrees. They can originate from biological processes such as sedimentation of skeletal debris and coral reef formation. They can also occur because of chemical precipitation of particles such as oolites. Because of their association with coral reefs, these soils appear mostly between the latitudes of 30EN and 30ES.

(2) Important Properties of Calcareous Soils. The brittle, crushable nature of calcareous sands complicates the site investigation. This makes both the site investigation itself and a meaningful correlation of test data to actual soil properties difficult. However, there are some important soil properties to watch for.

(a) Carbonate Content. By definition, these soils have higher than average carbonate content. The calcareous soils most prone to difficulties have a carbonate content by weight above 50 percent. Problems are especially pronounced above 80 percent, where many pile driven into these soils have abnormally low capacities.

(b) Degree of Cementation and Grain Structure. The grain structure of these soils is highly variable due to the diverse nature of the soils. This variability is one of the most important factors in the unpredictability of these soils. This variability can manifest itself in the angularity, size, or void structure of the grains or other factors. Light cementation can lead to both low shaft friction and toe capacity.

(c) Bulk Density. Void ratios for calcareous sands can vary from 0.8 to 1.4 as opposed to 0.4 to 0.9 for noncarbonated sands. The tendency to voids of all sizes is one of the most difficult problems encountered with calcareous sands.

(d) Specific Gravity. This normally varies from 2.75 to 2.85 with these soils.

(e) Friction Angle. This is generally greater than 35 degrees and can be greater than 50 degrees. This may decline with increased confining pressure, and the surface friction angle may decrease with surface roughness.

(3) Loading Response. Calcareous soils are highly compressible under pressure loading and are subject to softening under cyclic loading.

(4) Prediction of Driven Pile Capacity. Although a great deal of research has been done into the nature and engineering properties of calcareous sands, predicting the capacity of piles driven into these soils is highly speculative. Great care must be taken to verify that the soil can in fact support the installed foundation, both in the design process and especially in the field verification process. This should include extensive instrumentation and load testing of both indicator and production piling. With driven piles, a frequent occurrence with these soils is very easy driving; this can be due to temporary conditions (as was the case with this project) or a more permanent condition, in which case remedial action must be taken. Blow count cannot be relied upon as a key jobsite control method.

(5) Remediation. Once a situation has been encountered where the piles installed do not have the required capacity, remediation is necessary. One method, of course, is to drive the piles further. This can be uneconomical depending upon the situation. Another potential solution is to use drilled and grouted piles. These can significantly improve pile capacity by pressurizing the surrounding soil. However, as with any pile of this type comprehensive quality control during installation is essential.

b. Description of Kwajalein Project. Project Overview and Design Requirement. The aim of the project was to construct a drydock at the Kwajalein Atoll. The foundation designed to support the drydock was made up of 12 pile groups, designated N-1 through N-6 on the northwest side and S-1 through S-6 on the southeast side. The layout for these piles is shown in figure 5-1. The piles were to be driven into unconsolidated bioclastic limestone (coral) debris with sizes that ranged from silt to cobble, although the predominant size was sand. Some of the piles in each group were battered. These are shown with their batter orientation noted in figure 5-2. Before pile driving could begin, the drydock area was first dredged to -26 feet mean sea level (MSL). The area was also drilled and blasted during dredging operations.

c. Soil Investigations. Eleven soil borings were originally drilled. Four of these borings, B1 to B4, were offshore (that is, in the harbor). After the drydock was moved to the northwest about 100 feet, two additional borings, B12 and B13, were drilled for a total of 13 borings. Standard penetration test (SPT) results from these borings indicated N-values ranging from 3 to 35 blows/foot as shown in figure 5-3 for both onshore and offshore borings. Depths are shown relative to MSL. These constitute very loose to very dense sands and gravels; most sands were of medium density. The blow counts of borings B5, B6, and B7 in figure 5-3a and are representative of the coral sands in the onshore drydock area. Of the borings whose blow counts are shown in figure 5-3b, borings B2 and B4 were located about 100 ft southeast of the pile plan centerline shown in figure 5-1, and borings B12 and B13 are located as shown in figure 5-1. Assuming that the driving energies delivered to the borings was identical, comparison of figures 5-3a and 5-3b shows that the blow counts in the offshore borings are as great as or greater than those of the onshore borings. Additionally, the borings indicated broadly graded coral sand with gravel and lesser quantities of silt and clay.

d. Pile Capacity and Load Testing. The required design load Q_d for the offshore piles was 120 kips and 160 kips for the onshore piles. These piles were 20 inches square precast prestressed (PCPS) concrete piles, 85 feet long with an embedment depth of 53 feet to obtain the desired capacity. Proof load tests were conducted to twice the design depth on piles C2, E2, and E3, using the ASTM D 1143. These tests indicated adequate reserve capacity with displacements not exceeding 0.1 inch at the design load for the selected piles. Results of these tests are summarized in table 5-1. As for the driving resistance of these piles, for example the E2 pile was driven with a Delmag D46-23 hammer with a rated striking energy of 102,000 foot-pounds and a ram weight of 10,143 pounds. This hammer is a predecessor of the Delmag D46-32 shown in table 3-2. When first driven on 26 April 1991, the blow count was as low as 2 blows/foot, at which point driving was discontinued 6 feet before final toe elevation. When driving was resumed the next day, the blow count during driving increased to 17 blows/foot at a toe elevation of -81 feet MSL.

Table 5-1. Summary of load test results

Pile	Date Driven	Length	Embedment Depth	Design Load	Displacement at Design Load	Ultimate Capacity	Displacement at Ultimate Capacity
C2	13 Oct 1990	25.9 m (85')	24.7 m (81')	712 kN (160 kips)	2.5 mm (0.1")	>1423 kN (>320 kips)	7.6 mm (0.3")
E2	26-27 Apr 1991	25.9 m (85')	16.2 m (53')	534 kN (120 kips)	1.8 mm (0.07")	>1068 kN (>240 kips)	6.4 mm (0.25")
E3	10 Oct 1990	18.3 m (60')	18 m (59')	534 kN (120 kips)	2.5 mm (0.1")	801 kN (180 kips)	8.9 mm (0.35")

e. Construction Events. A unilateral letter contract to construction the drydock was awarded to a contractor 24 November 1989. Demolition of the existing facility was commenced, but was halted on 26 February 1990 because the Environment Assessment (EA) was deemed inadequate. With the signature of a new EA on 31 May 1990, work was resumed. Indicator piles were driven for the onshore end of the drydock. However, those planned for the offshore end were deleted to effect savings with it having been decided that the piles were unnecessary. Pile driving commenced 25 April 1991. Although lower than anticipated, blow counts were encountered, and the production piling was completed in June 1991. Subsequent to an analysis of the situation, the contract was terminated on 8 July 1991. To remedy the situation and complete the project, a supplemental test program was initiated and the tests under this program were conducted in March 1992. These tests indicated that over time the piles had developed the capacity to support the drydock.

5-2. ANALYSIS OF PRODUCTION PILE INSTALLATION.

a. Pile Driving Records.

(1) Pile Hammer. As was the case with the indicator piles, the rest of the piles were driven with the Delmag D46-23 hammer, set for the 66 foot-kip energy setting. This corresponds to a ram stroke of 6 feet.

(2) Embedment Depth. The embedment depth of the piling is shown in figure 5-4; the average embedment was 53 feet. The North group appears to have a slightly smaller embedment depth than the South groups. The embedment depth for one pile in group N-3 was 44 feet, and several piles were embedded about 50 feet; otherwise, these piles were driven to an embedment depth from 51 feet to 61 feet. Matching this depth range with the sampler blow counts from the borings in figure 5-3 indicates that the boring N-value N_{spt} was expected to be at least 10 blows/foot.

(3) Penetration Resistance. The blow counts at final penetration for each group of piles, figure 5-5, indicate that the penetration resistance at the toe depth of the North group is slightly less than that for the South group. The slightly smaller values observed for the North group is consistent with the slightly smaller embedment depth of the North group compared to the South group. Figure 5-5 also shows that the final blow count decreases from a range of 6 to 37 blows/foot near the head wall down to 2 to 12 blows/foot near the far end offshore. The piles driven later in a group have a higher mean blow count than those driven before. The mean blow count of the last pile driven in a group was generally twice that of the first pile driven. There was no discernible difference in the driving due to batter.

b. Ultimate Pile Capacity. As is the case with any pile driving project, there were several ways available to determine the load-bearing capacity of the piles.

(1) Wave Equation Analysis. A wave equation program was used to analyze the bearing capacity of the piles. Using this method, it is necessary to know the configuration of the hammer and pile, the soil tape, the distribution of the bearing capacity of the soil between the shaft and the toe, and also the distribution of the shaft resistance along the length of the shaft including the length of the pile embedment. The driving resistance for the pile can then be varied, and the various blow counts for each case can be calculated by the program. The results of this analysis for the Kwajalein piles are shown in figure 5-6.

(2) Application of Design Methods. Other methods that can be employed to estimate the bearing capacity of the piles are (a) dynamic formulae and (b) methods which are based on the properties of the soils themselves. The dynamic formulae have been largely superseded by the wave equation. A discussion of these for the piles in question is given in Miscellaneous Paper GL-92-23. Capacity estimation from the soil properties themselves is shown in figure 5-7. A more detailed explanation of the Meyerhof and Nordlund methods is given in TM 5-809-7, along with detailed instructions on the use of these methods.

(3) Load Test. Piles E2, E3, and C2 were tested with static loads 1 to 5 days after installation. For pile E2, the results of the load test is shown in figure 5-8. This result is consistent with the estimates given by the analytical methods. The load tests were performed 1 to 5 days after installation, and this could allow for soil freeze, perhaps from dissipation of excess pore water pressure. The effects of soil freeze are seen in figure 5-9, which compares the penetration resistance during the original driving with a restrike 12 to 24 hours after original driving. Results of this restrike are shown in figure 5-9.

c. Settlement. Settlement was estimated using the design load and Vesic's semi-empirical method, where the total settlement is the sum of the axial deformation of the shaft, the settlement at toe from load transmitted along the pile shaft, and settlement at toe from load transmitted at the toe.

(1) Axial Compression. A computation of axial compression is given in figure 5-10.

Table 5-2. Summary of settlement analysis.

Pile	Case	Total Settlement mm	Total Settlement inches
Headwall	50 percent shaft, 50 percent toe resistance	8.1	0.32
	100 percent shaft resistance	2.4	0.095
	Proof Test, Pile E2	1.8	0.07
Far End	50 percent shaft, 50 percent toe resistance	35.0	1.38
	100 percent shaft resistance	5.3	0.21

(4) Comparison with Load Tests. Settlement of the proof load test conducted on Pile E2 near the head wall (figure 5-8) shows about 0.07 inch at the design load of 120 kips for offshore piles. This

settlement is consistent with the calculated settlement for offshore piles near the head wall. Analysis of the elastic characteristics of the piles showed that the shaft friction is significant and appears to provide much of the bearing capacity for piles near the headwall.

d. Discussion of Results. Based on the results of the SPT tests and past experience, the foundation of the drydock should have had adequate capacity both onshore and offshore. The driving of the piles started onshore and proceeded to the offshore end of the drydock. As this took place, the penetration resistance during actual driving decreased. Additionally, penetration resistance decreased within each group as new piles were driving within a group. These events in the driving of the offshore piles cast doubt on the bearing capacity of the offshore piles. The probable cause of the decrease in driving resistance was the buildup of excess pore water pressures in the construction area coupled with the effects of preconstruction blasting, which may have further reduced cementation between sand particles near group N-5 and N-6. Although the presence of these factors was certain, because of them it was impossible to quantify the bearing capacity of the offshore piles without further verification.

5-3. SUPPLEMENTAL TEST PROGRAM. As a consequence of the results of the installation of the offshore piles, a supplemental test program was performed from 24 February to 5 March 1992, about 7 months after installation of the production piles.

a. Repeated Static Load Test. To verify the load bearing capacity of the offshore piles, additional load tests were performed on selected piles. One repeated static load tests was performed on Pile S-5-12. The purpose of this test was to calibrate dynamic monitoring results by the Case method and CAPWAP method (ASTM D 4945). This was to determine the bearing capacity of the restruck piles, and thus obviate the need to statically test each of these piles. The static test was repeated with greater maximum loads to check for the possible breakdown of coral particles and the piles and adhesion between coral particles that could reduce bearing capacity from repeated loads. Additionally, an indicator pile was driven between groups S-4 and S-5 after the repeated static load test. Bearing capacity of this indicator pile was evaluated from the pile driving analyzer and wave equation results. After the dynamic methods were calibrated, 10 restrikes of the production piles were also completed, which included the pile that was load tested.

(1) Procedure. The repeated static compression test was performed on pile S-5-12 in accordance with ASTM D 1143. The pile was embedded to a depth of 53 feet, and it was tested 261 days after installation. Figure 5-3b shows the blow count record of this production pile as it was originally driven in June 1991. The first cycle loaded the pile to the design load of 120 kips, then the load was reduced to zero. The second cycle was conducted to 240 kips, twice the design load, then returned to zero load. The third and final cycle was to be conducted to 360 kips, three times the design load. The standard loading procedure was used for the first and second load cycles, while loading in excess of the standard load option was used to complete the third cycle.

(2) Results. Figure 5-12 shows the results of the static load tests. For the first cycle, there was total elastic deformation after return to the zero load. The second cycle caused approximately 0.03 inch of permanent settlement. Creep recorded during the 12-hour holding period at 240 kips was also 0.03 inch. The permanent settlement equaled the amount of creep during the holding period. For the third load test, the hydraulic pump malfunctioned at the 360 kips load and the test could not be completed. The creep rate observed at this load was approximately 0.016 inch/hour, and total permanent settlement of the pile toe at this load was approximately 0.2 inch. A plunging failure was not achieved. Wave equation analysis using the CAPWAP program and the observed blow count of 34 blows/foot (as opposed to the 4 blows/foot during original driving) determined from the restrike of this pile estimated a failure load of about 440 kips. Ultimate capacity by the Case method was 298 kips. The CAPWAP method better evaluates the soil input parameters and distribution of soil resistance from the PDA results, and was expected to lead to a better estimate of bearing capacity than the Case method.

(2) Pile Toe Settlement. A computation of pile toe settlement is given in figure 5-11.

(3) Summary of Settlement Analysis Results. A summary of the results of the settlement computation (with actual data for Pile E2) is given in table 5-2.

b. Indicator Pile. Following the static load test, an indicator pile of the same dimensions and type as the production piles were driven between groups S-4 and S-5, near pile S-5-1. The pile was driven with the same Delmag D46-23 hammer that was used in the production driving. A comparison of the driving record of this pile with that of test pile E2 or pile S-5-12 is shown in figure 5-13. This figure shows that the blow counts of the indicator pile are substantially larger than those of pile E2 or S-5-12, especially below -40 feet MSL. The blow count at final penetration of the indicator pile at about -81 feet is approximately 20 blows/foot. This value is well within the 6 to 37 blows/foot observed at the toe depths of groups S-1, S-2, N-1 and N-2, but greater than the 3 to 12 blows/foot observed at the toe depth for groups S-4 and S-5. The mean penetration resistance of this pile is the same as or exceeds the largest mean value observed during the driving of any of the piles in the North groups. This observation is consistent with the mechanisms of sand densification during pile installation and recementation during driving, such as the result of sand densification may have dissipated shortly after driving.

c. Restrikes of Production Piles. Figure 5-14 shows the comparison of the blow counts of those pile restruck with their original blow counts. As was the case with the indicator pile, the same hammer used to driving the piles originally was used to restrike them.

d. Results of Supplemental Test Program. This program showed that the existing pile foundation has adequate bearing capacity to support the drydock and that the original specification for the project led to an adequate foundation. During the production driving, excess pore water pressures, sand densification, and increased confining stresses during driving lead to blow counts that were below expectation. However, in this case these phenomena reversed themselves after driving, leading to soil freeze and increased blow counts during restrikes and thus increased bearing capacity over time. Ultimate bearing capacity evaluated by dynamic monitoring using the pile driving analyzer was consistent with results of static load tests.

5-4. LESSONS LEARNED. In addition to the detailed geotechnical aspects of the Kwajalein project, there are some important overall lessons that are to be learned from this experience.

a. Complete Environmental Requirements. Districts must assure that complete, documented approval of the fulfillment of all existing environmental requirements, including those under the National Environmental Policy Act (NEPA), are in place prior to contract award. Failure to do this can result in significant delays and costly modifications to the project. Environmental requirements relating to construction must be addressed and approvals obtained and fully documented prior to the beginning of construction.

b. Complete Thorough Soil Investigation. Subsurface investigation should be thorough, complete, and span the full range of site investigations.

c. Complete Driving of Indicator Piles. Any indicator or test pile program should be planned from the start, locating the piles to reflect the range of potential site conditions. The piles should be driven before the production piling is started and load tests be applied to the piles after a minimum waiting period, usually 1 day. The driving of these piles, along with all other testing, should be handled with a separate contract from the production piling.

d. Establish Criteria for Restrikes and Setup Time. If piles are in coral or other cemented sand strata, a setup time should be considered and a set duration time established in the design. The load test program for the indicator piles should reflect the specifications of the design.

e. Establish Criteria for and Complete Load Tests. In addition to the tests on the indicator piles, load testing on production piles should be planned for during design and contracted for separately from the piling. Any use of dynamic testing in conjunction with static load testing should be specified and properly correlated with any statically load tested piles, whether they be indicator piles or otherwise.

f. Establish Pile Lengths from Results of All Tests. Pile lengths should be finalized only after the indicator pile testing is complete and the analysis has been performed on the results.

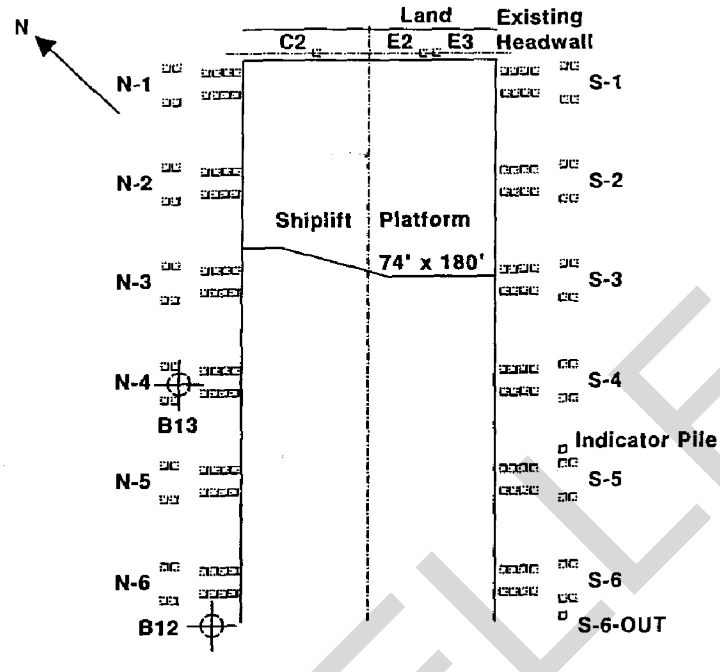


Figure 5-1. Drydock pile layout.

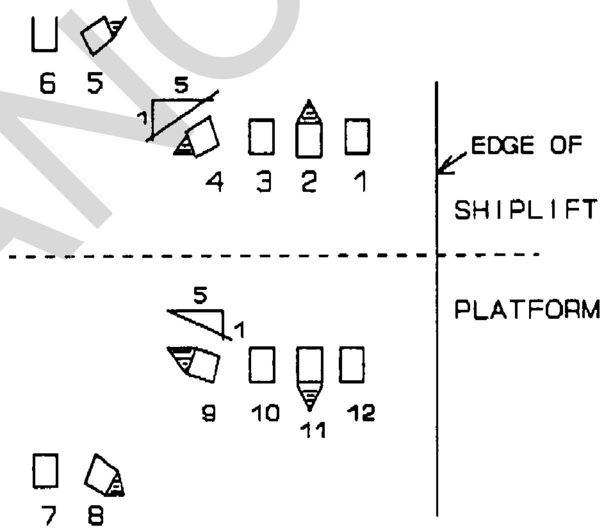
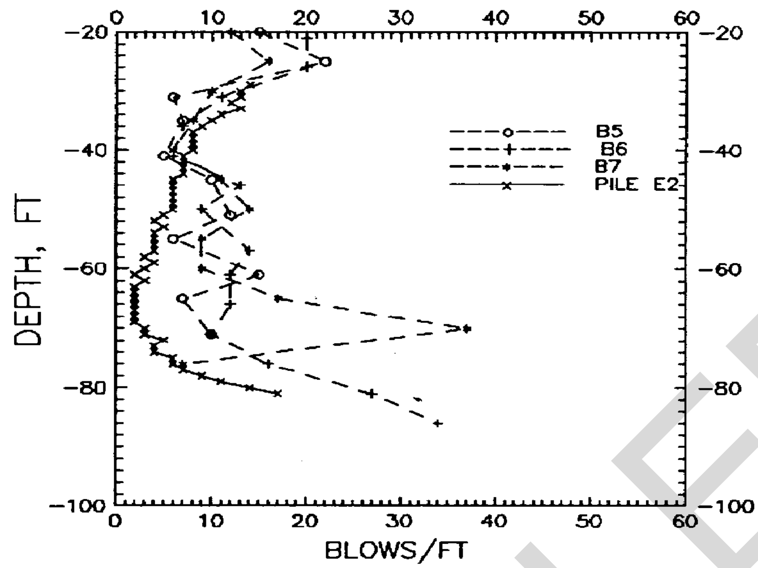
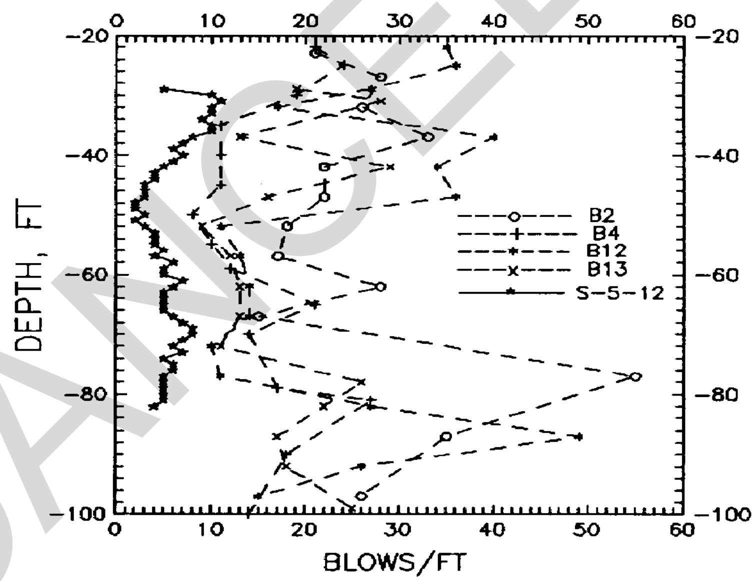


Figure 5-2. Pile group layout with batter pile orientations.



(a) Onshore borings.



(b) Offshort Borings

Figure 5-3. Standard penetration resistance results.

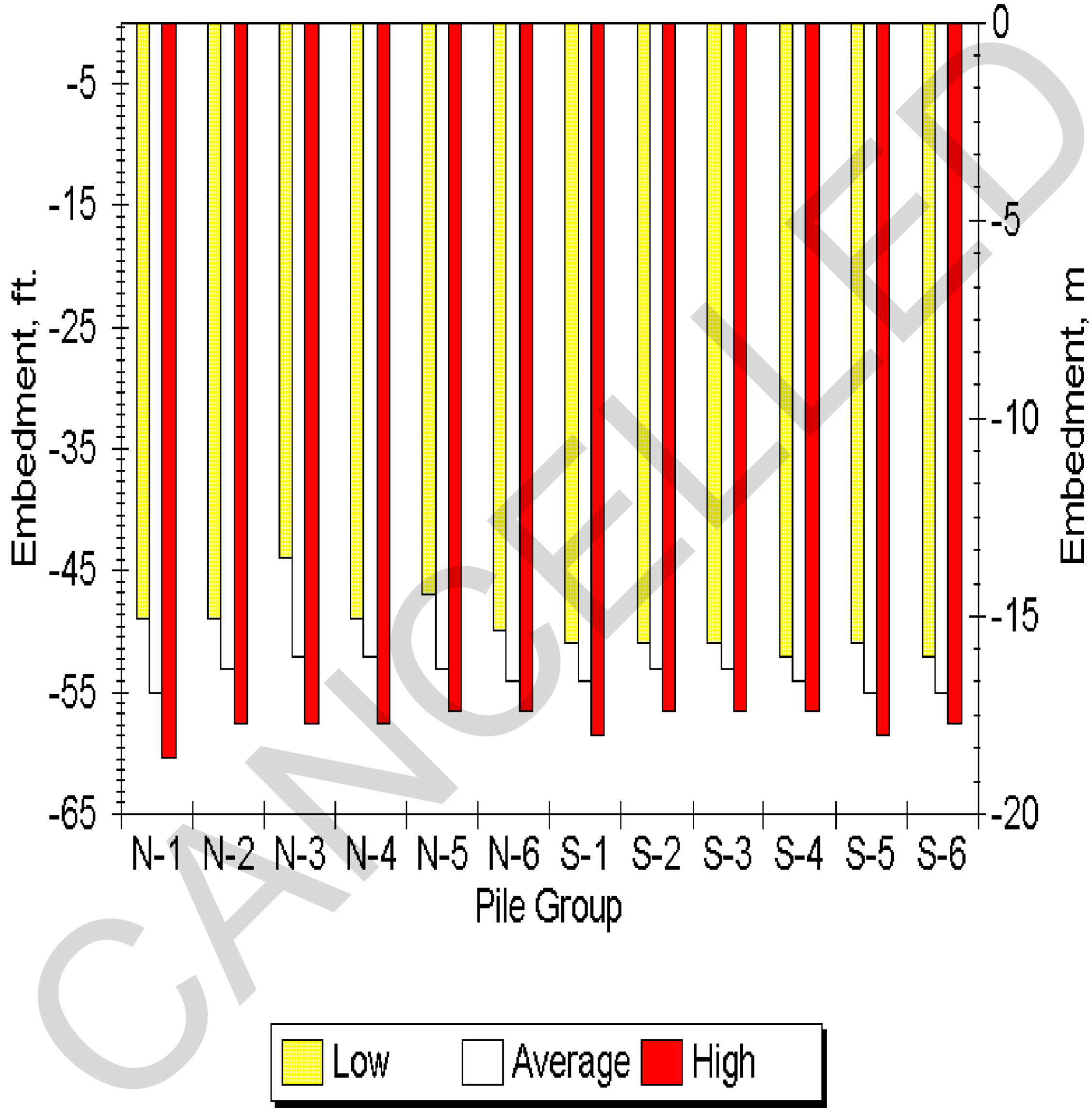


Figure 5-4. Piling embedment depths.

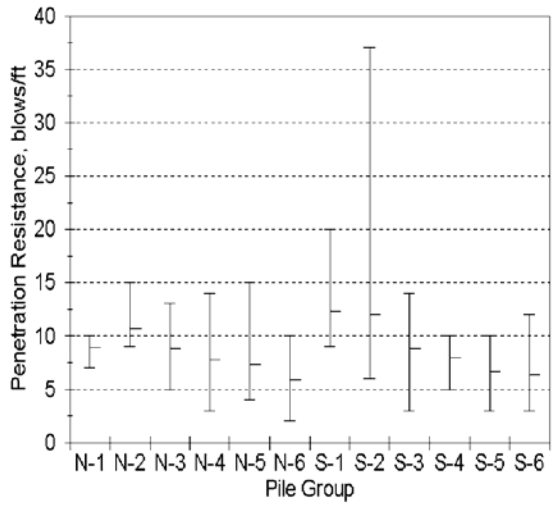


Figure 5-5. Low, average, and high blow counts for pile groups.

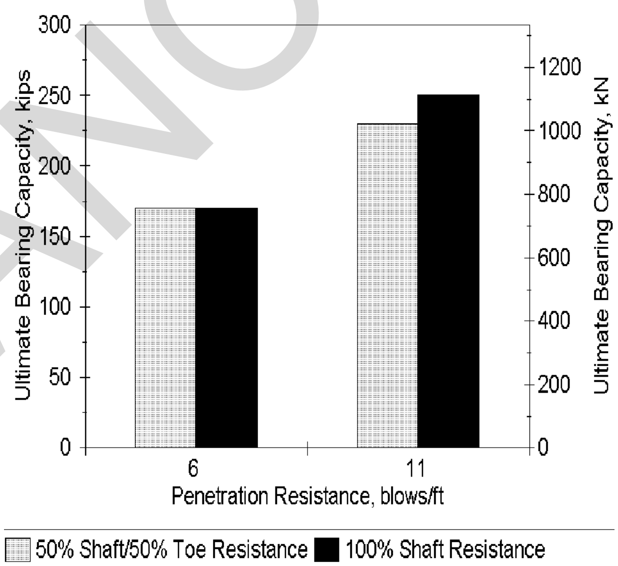


Figure 5-6. Wave equation analysis results.

In order to compute the bearing capacity of the pile by either Meyerhof's or Nordlund's method, it is first necessary to estimate the internal frictional angle for the soil. Based on gradation analysis of the boring sample of the coral sands, 2 is assumed to be 35 degrees at the far end and 30 degrees at the head wall.

To determine the total bearing capacity Q_u of the pile, use the formula

$$Q_u = Q_{bu} + Q_{su} = q_{bu}A_b + C_sLf_s \tag{1}$$

where

Q_u = bearing capacity of the pile, kN or kips
Q_{bu} = shaft friction, kN or kips
Q_{su} = toe bearing, kN or kips
q_{bu} = toe bearing resistance, kPa or ksf
A_b = toe bearing area, m^2 or ft^2 = .258 m^2 or 2.78 ft^2
C_s = shaft circumference, m or ft = 2.032m or 6.67 ft
L = embedded length, m or ft = 16.159m or 53 ft
f_s = maximum mobilized shaft friction, kPa or ksf

Toe Resistance -- Meyerhof Method

The Meyerhof method computes the toe bearing resistance by the formula

$$q_{bu} = F'N_{qp}\,{}^{\cdot}{}_{qp} \# q1 = N_{qptanN} \tag{2}$$

where

F'1 = effective overburden pressure at the pile toe, kPa or ksf
${}^{\cdot}{}_{qp}$ = geometry correction factor = 1
N_{qp} = bearing capacity surcharge factor = 60 for N = 30 degrees and 150 for N = 35 degrees

The effective overburden pressure is limited to the overburden pressure at the critical depth, which is 10 times the pile size or in this case Lc = 5.08 m (16.7'). This can be computed by the equation

$$N'I = L_{c((sat-(w)} = 42 \text{ kPa } (.877 \text{ ksf}) \tag{3}$$

where

$(_{sat}$ = 18.1 kPa/m (.115 ksf/ft)
$(_w$ = 9.82 kPa/m (.0625 ksf/ft)

Solving equation (2) using the N'I from equation (3), q_{bu} = 6,272 kPa (131 ksf) at the headwall and 2,921 kPa (61 ksf) at the far end. Both of these values exceed the q_1 from equation (2); therefore, for the first term of equation (1), the toe capacity by the Meyerhof method is

Q_{bu} = 1,299 kN (292 kips) (4a Headwall, N = 11)

Q_{bu} = 427 kN (96 kips) (4b Far End, N = 6)
d)

(sheet 1 of 3)

Figure 5-7. Pile capacity by Meyerhof and Nordlund methods.

Shaft Resistance, Meyerhof Method

Shaft resistance by the Meyerhof method is given by the equation

$$f_s = \beta f \sigma' i \quad . \qquad (5)$$

where

βf = lateral earth pressure and friction angle factor
$\sigma' i$ = average effective overburden pressure along the embedded length L = 42 kPa (.877 ksf)

The skin friction capacity (the second term in equation (1)) thus solves to

Q_{su} = 1,099 kN (247 kips) (eq 5-6a Headwall, N = 11)

Q_{su} = 276 kN (62 kips) (eq 5-6b Far End, N = 6)

Toe Resistance, Nordlund Method

The end bearing resistance by the Nordlund method is given by the equation

$$q_{bu} = \alpha f N_{qp\sigma' i} \qquad (7)$$

where

af = depth-width relationship factor = .65 for this case
N_{qp} = bearing capacity surcharge factor = 70 at the headwall, 30 at the far end

The toe resistance, by the first equation of equation (1), is

Q_{bu} = 494 kN (111 kips) (7a Headwall, N=11)

Q_{bu} = 163 kN (36.6 kips) (7b Far End, N=6)

Shaft Resistance, Nordlund Method

The shaft resistance by the Nordlund method is given by the equation

$$Q_{su} = KC_{f\sigma' i}C_s L \sin\delta \qquad (8)$$

where

K = coefficient of lateral earth pressure = 2 for headwall, 1.4 for far end
C_f = correction factor = 2
δ = 31.5 degrees for headwall, 27 degrees for far end

This equation solves to

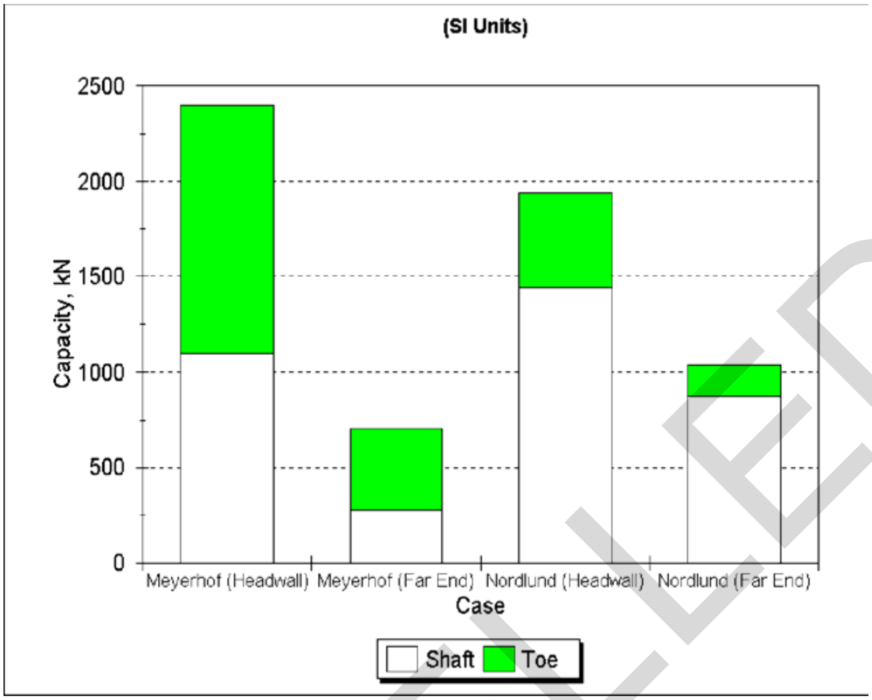
Q_{su} = 1,441 kN (324 kips) (9a Headwall, N=11)

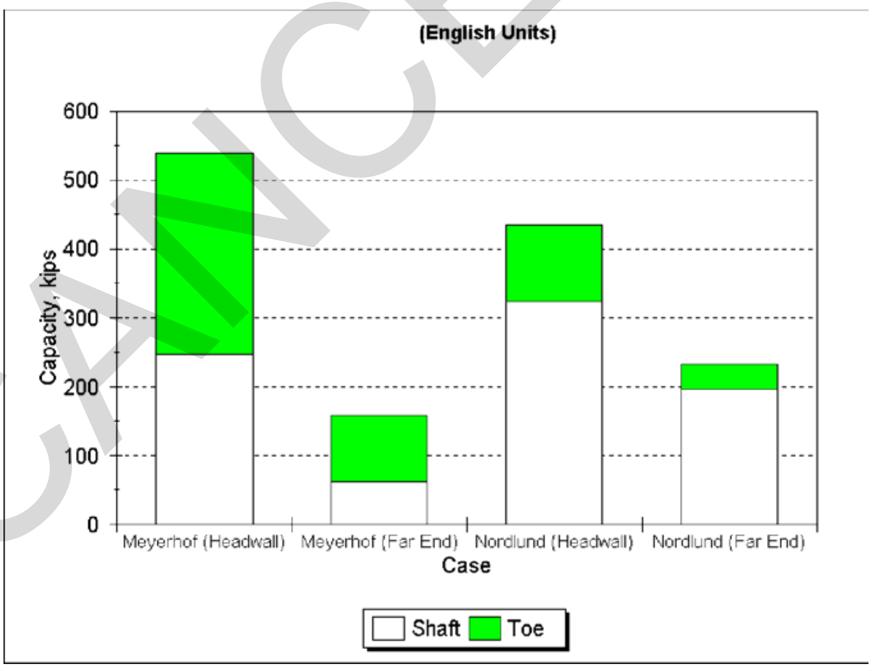
Q_{su} = 876 kN (197 kips) (9b Far End, N=6)

The ultimate capacity, their distribution between shaft and toe, and the two methods are compared on the following page.

(sheet 2 of 3)

Figure 5-7. (Continued)





(sheet 3 of 3)

Figure 5-7. (Concluded)

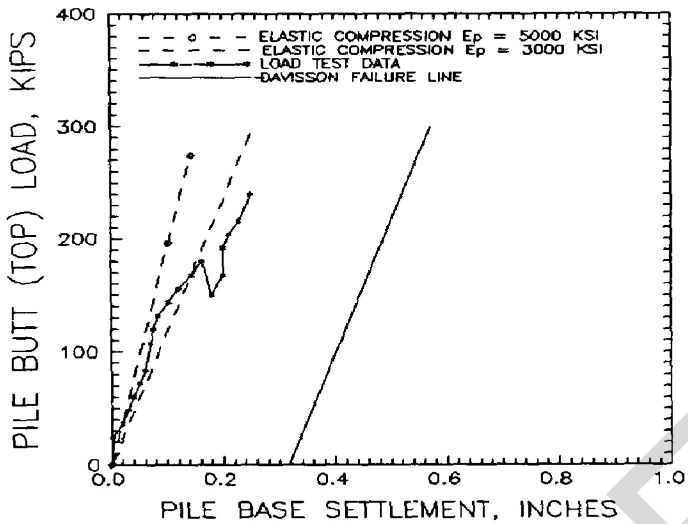


Figure 5-8. Load test results for pile E-2.

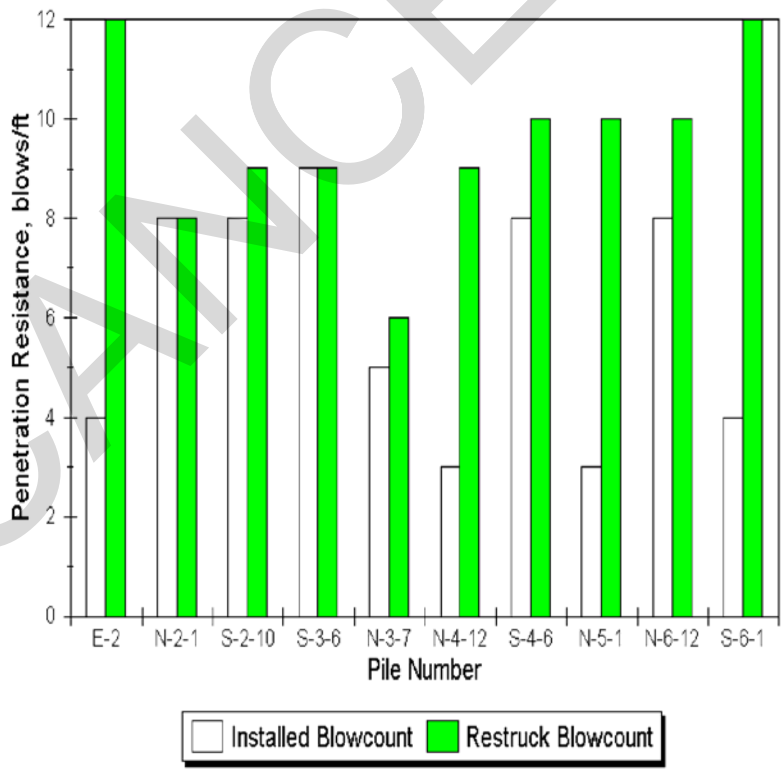


Figure 5-9. Penetration resistance of production piles during restrike.

The axial compression of a pile is given by the equation

$$Dp = ((Q_b + "^{s}Q_s)L + (Q_b + Q_s)(L_t\text{-}L))/(A_bE_p)\text{............}(1)$$

where

Dp	=	settlement from axial deformation of the shaft, m or ft
Q_b	=	design load at the pile toe, kN or kips
$"^{s}$	=	distribution factor for load along the pile length = 0.5 to 0.67; normally assume 0.5
Q_s	=	design load taken by shaft friction, kN or kips
L	=	embedded length, m or ft = 16.159m or 53 ft
L_t	=	total pile length = 25.9m (85')
A_b	=	toe bearing area, m^2 or ft^2 = .258 m^2 or 2.78 ft^2
E_p	=	pile modulus of elasticity = 20,682,000 kPa (432,000 kips/ft^2)

A rough estimate of Q_s may be made by assuming that $Q_s = Q_{su}$, since nearly all shaft resistance will be mobilized before significant toe resistance, unless the pile toe is bearing on a hard stratum. Q_b is then estimated by subtracting Q_{su} from the design load Q_d. If Q_b and Q_s are each assumed to take half of the load or 267 kN (60 kips), then

Dp = 0.00218m (0.0072')...........(2a -- Qb = Qs = 267 kN)

If the design load of 534 kN (120 kips) is assumed to be taken totally by shaft friction,

Dp = 0.00178m (.0058').................(2b -- Qs = 534 kN)

Figure 5-10. Computation of axial pile compression.

The settlement at the pile toe is given by the equation

$\rho^b = C_b Q_b / (Bq_{bu})$ . . . . (1a -- Toe Settlement)

and

$\rho^s = C_s Q_s / (Lq_{bu})$ . . . . (1b -- Shaft Settlement)

where

- ρ_s = settlement at toe from load transmitted along the pile shaft, m or ft
- ρ_b = settlement at toe from load transferred at the toe, m or ft
- B = pile width = 0.504m or 1.67'
- L = pile embedded length = 16.159m or 53'
- C_b = empirical coefficient
- C_s = coefficient = ((0.93 + 0.16(L/Bs)0.5)Cb
- Q_b = load supported by end bearing, kN or kips
- Q_s = load supported by shaft friction, kN or kips
- q_{bu} = end bearing resistant, kPa or ksf

For settlement near the headwall, C_b = 0.02 and C_s = 0.037. q_{bu} is assumed to be 1915 kPa (40 ksf). The settlements can be computed as follows:

ρ^b = 0.0055m (0.018')..........(2a -- headwall, shaft/toe resistance evenly divided)

ρ^s = 0.00033m (0.00108')........(2b -- headwall, shaft/toe resistance evenly divided)

ρ^s = 0.00064m (0.00208')........(2c -- headwall, all shaft resistance, no toe resistance)

For settlement near the far end, Cb = 0.04 and Cs = 0.073. q_{bu} is assumed to be 670 kPa (14 ksf). The settlements can be computed as follows:

ρ^b = 0.031m (0.1025')...........(3a -- far end, shaft/toe resistance evenly divided)

ρ^s = 0.0018 m (0.00592')........(3b -- far end, shaft/toe resistance evenly divided)

ρ^s = 0.0036m (0.0117')..........(3c -- far end, all shaft resistance, no toe resistance)

Figure 5-11. Computation of pile toe settlement.

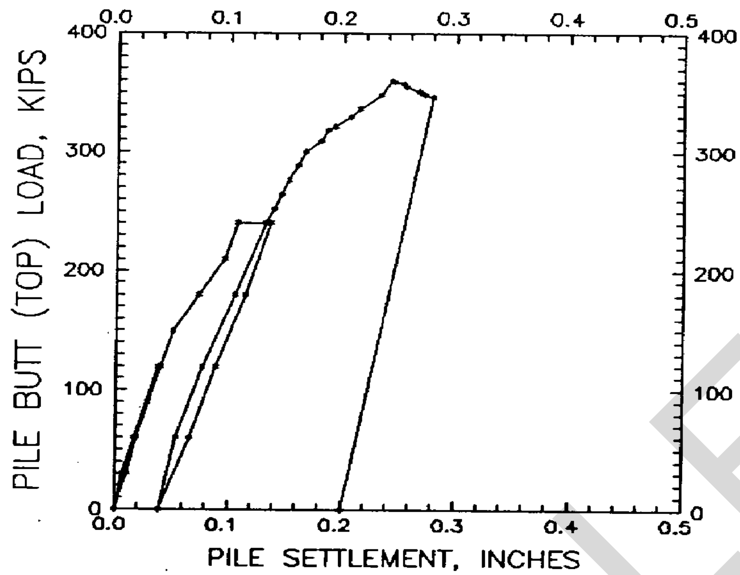


Figure 5-12. Static load test results, supplemental test program.

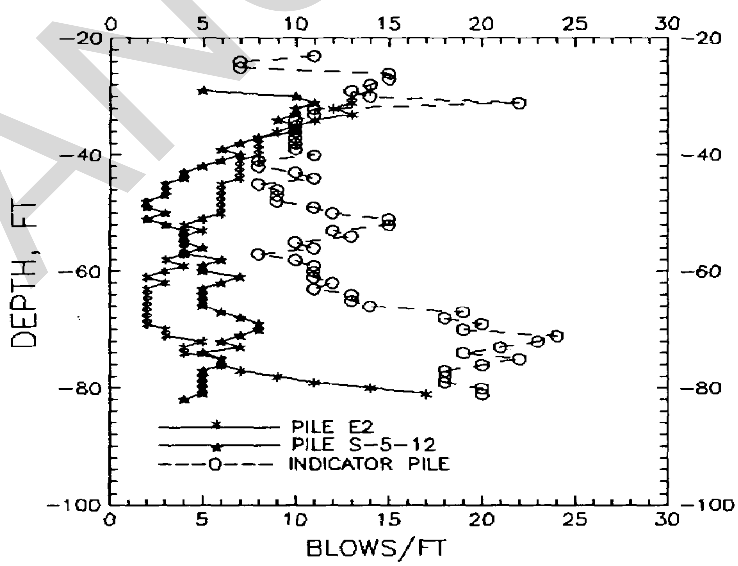


Figure 5-13. Comparison of driving record of indicator pile with piles E-2 and S-5-12.

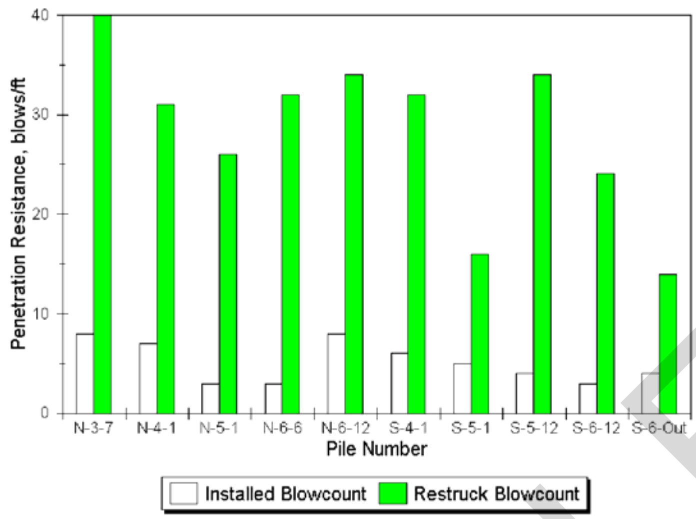


Figure 5-14. Blow count comparison for restruck piles.

APPENDIX A

REFERENCES

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REFERENCES

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A 36 M-93a	Specification for Structural Steel
A 252	Specification for Welded and Seamless Steel Pipe Piles
A 572 M-93	Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
A 690	Specification for High-Strength Low-Alloy Steel H-Piles and Steel Piling for Use in Marine Environments, 01.04
D 25	Specification for Round Timber Piles
D 1143-81	Method of Testing Piles Under Static Axial Compressive Loads, 04.08
D 3689-90	Method of Testing Individual Piles Under Static Axial Tensile Load
D 3966-90	Method of Testing Piles Under Lateral Loads
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APPENDIX B

BIBLIOGRAPHY

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