HIGHWAY RESEARCH RECORD

Number 333

Pile Foundations

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Foreword

All of the papers in this RECORD except two were presented at a Symposium on Pile Foundations held at the 49th Annual Meeting of the Highway Research Board. Two sessions were sponsored jointly by the Committee on Substructures, Retaining Walls and Foundations and the Committee on Foundations of Bridges and Other Structures. The purpose of the Symposium was to review current practice and knowledge in the design and behavior of pile foundations. Topics were chosen to cover the broad area of interest, and authors were invited to make presentations because of their expertise and professional competence in the particular subareas.

The first 4 papers deal essentially with problems associated with design and construction of pile foundations, such as choice of pile types and techniques for driving and estimating pile bearing capacities. These 4 papers are summarized by Kapp. The next 4 papers, summarized by Leonards, are directed generally at behavior of the pile-soil system.

The final 2 papers were not presented at the Symposium but make important contributions to the subject. Gerwick's paper addresses the subject of the use of high-capacity piling for highway bridges. Emphasis is placed on the need to account for both the structural capacity of the pile and the capacity of the soil as well as the desirability of integrating design and installation procedures in order that maximum benefits of high-capacity piles can be made more widely available to the bridge engineering profession. The paper by Goble and Rausche presents a method for predicting pile static capacity different from dynamic measurements made by impact driving. The method was applied to 24 statically load-tested piles with good correlation of predicted capacities. By use of a special purpose computer, a predicted capacity can be computed and displayed within 2 milliseconds after each hammer blow.

Contents

Summary and Review of Part I of the Symposium on Pile Foundations

MARTIN S. KAPP, Port of New York Authority

•IN HIS PAPER on the various types of piles and their characteristics and general use, Grand has the task of setting the stage for the remaining papers. His responsibility is to tell what we thought we already knew about piling. He has traced the historical development of pile usage and the purpose of pile foundations. He cautions that there is a need for complete subsurface investigation and then lists pile types with their physical characteristics. His list, although not fully complete, is comprehensive enough to cover those piles that constitute the majority of installations. Grand also includes in his paper suggestions as to where and for what purpose the numerous types of piles are to be used and also where they are not to be used.

Many practicing engineers will want to know the relative cost of the different types of piling and that magic formula for picking the right pile to do a specific job. This is just too dependent on factors such as locality, specific soil conditions, previous area practices, available equipment, size of job, local contractors' interests, and competition. The engineer will always need his past experience to make that last decision, but Grand's paper is helpful in showing the available alternatives. I only wish he had started his paper with the following comment: The most important decision to be made concerning piling is whether it is needed in the first place.

Mosley and Raamot compare the values of ultimate resistance for 3 dynamic formulas with that computed by the wave equation. This work was confined to the Engineering-News formula, the Hiley formula, the Eytelwein formula, and the wave equation. While elaborating on the limitations of dynamic pile-driving formulas (and who can say that they do not have major limitations), the authors feel that, of solutions currently available, the wave equation offers the only reliable method. They have plotted results with variables of pile materials, size, length, soil resistance, and pile hammers. Can any engineer read this paper and still go back with confidence to his dynamic formula?

However, one must recognize Mosley and Raamot's recent dedication to development of the wave equation (could this explain their partiality?) and remember that dynamic pile formulas have served useful purposes in many areas for many engineers (usually because of past experience and with moderate-weight piles and low-energy hammers). We have always recognized the shortcomings of the various dynamic pile-driving formulas. Mosley and Raamot tell us to do something about it and point the direction.

The heart of any pile-driving system is the pile hammer, and in his paper on hammers and driving methods Gendron describes the most commonly used pile hammers and speculates on driving systems of the future. Because of the simplicity of its operation, drop hammers are still in use, although the economical demands placed on the modern pile contractor do not encourage their use.

Gendron describes the single-acting, double-acting, and differential hammers. The single-acting hammers are simple and reliable, developing consistent energy when the stroke is controlled. From the contractor's point of view, they have a major advantage of a large ratio of weight of ram to total weight of hammer and are mechanically reliable and low in maintenance. Because a faster hammer was desired the double-acting

Paper sponsored by Committee on Substructures, Retaining Walls and Foundations and presented at the 49th Annual Meeting.

hammer was developed (100 to 250 blows per minute compared with 60 to 75 blows per minute for a single-acting hammer). It strikes a relatively high-velocity blow that some theoretical studies have shown to be inefficient in the driving of heavy piles. The differential hammer overcomes the deficiencies of the single- and double-acting hammers, yet maintains some of their advantages. They are the hammers of today according to the author.

Gendron also describes the diesel hammer, which is a single piece of equipment combining the hammer and its power source. Much is yet to be learned about the measured output of a diesel hammer; but contractors are increasing their use of it, and it is here to stay. However, foundation engineers must become more "comfortable" with this hammer before it has universal acceptance.

In addition to his discussion of hammers, Gendron also covers the associated equipment such as cap blocks and cushions and highlights the Micarta-aluminum cushion, which is widely used today. The author discusses vibrators, hydraulic hammers, and the probable need for very high-energy hammers to handle the ever-growing need for larger and heavier piling.

More experience and results are needed on vibratory hammers before one can predict whether their use will either lessen or increase soil vibrations adjacent to the pile installation. In addition, there are many "tricks of the trade" for increasing or decreasing hammer energy output, and a quality installation requires quality inspectors trained in the use and mechanics of the pile hammers.

Hirsch, Lowery, Coyle, and Samson state, "The numerical computer solution of the one-dimensional wave equation can be used with reasonable confidence for the analysis of pile-driving problems." That statement appears to me to be a little too optimistic. If they limit their statement and say that this equation is a tremendous aid to the selection of pile hammer, related equipment, and pile material and size, then I have no argument. In fact, I am amazed at how much help we can get from the wave equation and wonder why it is not used more. I believe that a large number of pile installation problems will disappear with its general understanding and adoption.

The authors have developed a computer program based on Smith's procedure to provide the engineer a numerical solution of one-dimensional wave equation. The computer solution will also predict the impact stresses during driving as well as the soil resistance of a pile at the time of driving. The authors also find from the Michigan pile study that the accurate energy output for pile-driving hammers can be obtained. They demonstrate that driving accessories significantly affect the piling behavior. For this reason their selection should be carefully considered and analyzed whenever possible.

It was found in the investigations that stress-strain curves for a cushion block were not linear as was assumed by Smith. However, it was noted, surprisingly, that for a given material the dynamic curves during the loading of the specimen were almost identical to the corresponding static curves. As such, static can best be used to determine cushion stiffness but not for coefficient of restitutions. It was also fortunately found that, even if a linear force-deformation curve were assumed for a cushion, the wave equation predicts accurately the shape and magnitude of the stress wave as long as the loading portion is based on the secant modulus of elasticity of the material.

A comparison of the results of field test and numerical solutions was encouraging. The results of experiments in the laboratory compare accurately with numerical solutions. The effect of pile dimensions on the ability to drive the pile varied greatly. It was found that the stiffer the pile is the greater soil resistance to penetration it can overcome.

It does appear that the wave equation has demonstrated its usefulness in picking the optimum combination of hammer, cap block, and cushion for a particular application. However, some engineers may question whether it has as yet proven to be a reliable indicator of the "dynamic" pile resistance. We must remember that the variables that that go into the equation have a significant effect on its results. For example, it is critical that we know the characteristics of the soil, the hammer, and the cushion. Consideration should also be given to what stresses should be allowed for the various pile materials.

Types of Piles: Their Characteristics and General Use

BERNARD A. GRAND, Hardesty and Hanover

This paper presents a review of the current practice and usage of the numerous types of pile in general construction. Information on this subject was obtained from a review of existing literature and from field experience. The paper reviews the purpose of pile foundations and the various factors involved in the selection of a type of pile. Emphasis is placed on the general, physical, and structural characteristics of the piles as well as durability and fabrication. Data are presented on the inherent advantages and disadvantages of the various types of piles and on corresponding optimum pile length and load range. Information and data are presented on the field problems of pile installations and the proper method of handling and treatment to avoid damage or failure of critical pile sections. The fundamental information is supplemented by case histories.

•PILE FOUNDATIONS of timber were in use in ancient times. In its earliest form, a pile foundation consisted of rows of timber stakes driven into the ground. Pile foundations such as these were used by the ancient Aztecs in North America. The Romans made frequent use of pile foundations as recorded by Vitruvius in 59 AD. Pile foundations for ancient Roman dwellings have been found in Lake Lucerne. It is reported that during the rule of Julius Caesar a pile-supported bridge was constructed across the Rhine River.

The durability of timber piles is illustrated in the report of the reconstruction of an ancient bridge in Venice in 1902. The submerged timber piles of this bridge, which were driven in 900 AD, were found in good condition and were reused.

In the years immediately preceding the turn of the twentieth century, several types of concrete piles were devised. These early concrete piles were the cast-in-place type. Further development of the concrete pile led to the precast pile and, relatively recently, to the prestressed concrete pile. The need for extremely long piles with high bearing capacity led to the use of concrete-filled steel-pipe piles about 50 to 60 years ago. More recently, steel H-piles have come into common usage. Their ease of handling, fabrication, splicing, and relatively easy penetration hastened their acceptability in foundation construction.

THE PURPOSE OF A PILE FOUNDATION

The primary function of a pile foundation is (a) to transmit the load of a structure through a material or stratum of poor bearing capacity to one of adequate bearing capacity; (b) in some instances, to improve the load-bearing capacity of the soil; and (c) to resist lateral loads and to function as a fender to absorb wear and shock. In addition, piles are also used in special situations (a) to eliminate objectionable settlement; (b) to transfer loads from a structure through easily eroded soils in a scour zone to a stable underlying bearing stratum; (c) to anchor structures subjected to hydrostatic uplift or overturning; and (d) to serve as a retaining structure when installed in groups or in a series of overlapping (cast-in-place) piles.

Paper sponsored by Committee on Substructures, Retaining Walls and Foundations and presented at the 49th Annual Meeting.

NEED FOR SUBSURFACE INVESTIGATIONS

The length of the pile and the method of pile installation are dependent on the nature of the subsurface conditions. Thorough subsurface explorations are necessary to determine the stratification of the foundation elements, including the depth to bedrock and the density of granular materials measured by the number of blows recorded on a standard split spoon sampler, and to obtain undisturbed samples of cohesive strata to evaluate the shearing strength and compressibility characteristics by laboratory testing. The desirable number of exploratory borings depends on the size of the foundation area and the degree of uniformity of the foundation materials. In areas of glacial deposits, the foundation materials tend to be nonuniform, whereas the soil conditions are generally more uniform in marine or alluvial deposits.

Ideally, subsurface explorations should extend to a depth of 100 ft or to a depth of $1\frac{1}{2}$ times the width of the structure, unless bedrock is encountered at a shallower depth.

Groundwater conditions are pertinent in a pile foundation project from the standpoint of the probable permanency of the groundwater level, which is relevant to preserving the permanency of untreated timber piles. The condition of the groundwater is also relevant to steel and concrete piles where acid, alkali, or other injurious solutions may be present.

CHOICE OF PILE TYPE

The initial and primary consideration is the evaluation of the foundation materials and the selection of the substratum that will provide the best pile foundation support. In certain situations involving cohesive subsoils, the pile lengths will be dictated by the necessity to minimize settlement of the foundations rather than the need to develop load capacity. The selection of a type of pile for a given foundation should be made on the basis of a comparative study of cost, permanency, stability under vertical and a horizontal loading, long-term settlement, if any, of the foundation, required method of pile installation, and length of pile required to develop sufficient point bearing and frictional resistance assuming that there is a great depth to bedrock or other hard bottom.

The selection of a pile type and its appurtenances is dependent on environmental factors as, for example, piles in seawater. Environmental factors to be considered are the possibility of marine borer attack, wave action causing alternate wetting and drying and ultimate deterioration, and abrasion due to moving debris or ice. Piles located in strong water currents could be subject to gradual erosion of the pile material due to scouring by abrasive river sediment. Strong chemicals in rivers or streams or alkali soils could adversely affect concrete piles. Steel piles in an electrolytic environment near stray electrical currents could suffer serious electrolysis deterioration.

Foundation materials consisting of loose to medium-dense granular soils would favor a tapered displacement pile for efficient transfer of load along the surface of the pile by friction. If the granular soils were in a very compact state, the piles would probably have to be installed with the aid of water jets. Foundation materials consisting of cohesive soil underlaid by a granular stratum would favor a straight-sided pile to develop the greatest possible skin friction area along the pile and point bearing area at the base of the pile. Piles to be driven through obstructions to bedrock with the least driving effort and soil displacement would favor a steel H-pile or open-end pipe pile. Foundations subject to large lateral forces such as pier bents in either deep or swiftly moving water or both require piles that can sustain large bending forces. Precast, prestressed concrete piles are suitable for such load conditions. The large-diameter Raymond cylindrical prestressed piles have large vertical load and bending moment capacity and are frequently used in such installations.

TIMBER PILES

Timber piles have a wide range of sizes and strengths. The usual timber pile is a tree with a straight trunk and trimmed of branches. The butt diameter ranges in size from 12 to 20 in. and the tip diameter from 5 to 10 in. Their availability depends on transportation facilities and distance from lumbering regions. In North America the most commonly used trees for piles are southern yellow pine, Douglas fir, spruce, and oak. Southern cypress from the Atlantic and Gulf coasts are also extensively used in piling. Cedar piles, although decay resistant, do not find extensive use because of their relatively low strength. From Central America, some greenheart and angelique are used. They are hardwoods and have considerable resistance to marine borers.

Physical Characteristics

The maximum obtainable length of timber piles is of the order of 110 ft, but lengths over 80 ft are scarce. The normal length of available timber piles is 30 to 60 ft. The elasticity of timber makes wooden piles easy to handle. Timber is well adapted for use in dolphins and fenders for the protection of structures in water because of its resilience, wearing qualities, and ease of replacement. Timber piles are comparatively light for their strength, and they can absorb normal driving stresses to develop their design load. However, they are vulnerable to damage in hard driving. Timber piles are also vulnerable to deterioration and to destruction by marine organisms as described later.

Durability of Timber Piles

Timber piles are subject to deterioration caused by decay, insect attack, marine borer attack, and abrasive wear. Decay is caused by growth of fungi that need moisture, air, favorable temperature, and food. Decay can be prevented if wood can be kept dry, rendered unsuitable for food, or entirely embedded in earth and cut off below groundwater level or submerged in fresh water. Thus, untreated timber piles are subject to decay and insect attack where they project above the water table or above the ground surface, and to marine borer attack where they project above channel bottom in saltwater.

Reasonable protection against decay and insect attack, such as termites, can be attained by poisoning the pile by impregnating the wood with pentachlorophenal or with creosote. Treatment with pentachlorophenal is not recommended for marine piles. Creosote treatment by a pressure process is the most effective method of poisoning wood piles for long-term protection. However, this treatment will not prevent ultimate damage by certain species of marine borers, notably the liminora.

Mechanical protection of wood piles in waterfront structures has been used successfully to protect new piles and to repair piles damaged by abrasion or by marine borers. Mechanical devices include Gunite encasements and precast concrete jackets grouted to the piles. Intrusion-Prepakt concrete placed inside of forms fitted to timber piles has also been used. Such encasements generally extend from a few feet below the mud line to some distance above the high water level.

Fabrication

It is the general practice to remove the bark from wood because timber piles generally carry load by skin friction. A decomposed weak film ultimately develops between the bark and the wood creating a plane of weakness.

The butts of timber piles are cut square and the edges chamfered. The chamfering tends to reduce the tendency to split during pile-driving. When piles are to be driven without the aid of water jets, it is standard practice to trim the pile tips to about a 4-in. diameter when driving through relatively firm foundation materials. In driving through gravelly soils, it is frequently the practice to point the pile tips and clad them with steel shoes to prevent brooming.

Timber piles can be spliced when long piles are unavailable; however, it is time-consuming and rather difficult. Sleeve joint splices have been fabricated with 8-in. and 10-in. diameter pipe, 3 to 4 ft long. Bolted splices have been made by using timber and steel splice bars. Gunite splices 6 ft long have been made by utilizing spiral reinforcement surrounding $^3/_4$-in. diameter longitudinal reinforcing bars covered with a 5-in. thick mortar section. In current practice, splicing of timber piles is an infrequent occurrence.

Structural Characteristics

The normal design load for a timber pile is 15 to 25 tons with a maximum permissible load of 30 tons. A number of load tests on timber piles embedded for their full length have indicated a safe load capacity of 40 tons. Timber piles are vulnerable to damage in hard driving, and a water jet is frequently utilized in the installation of piles in dense granular materials. A single jet pipe strapped to the pile is generally used to install the pile to within 2 to 3 ft of the desired tip elevation, and the pile is driven to its final position to the prescribed driving resistance.

Timber piles, designed to develop their load by end bearing, are sometimes driven butt down to utilize the larger end bearing area. Timber piles installed as dolphins are occasionally driven butt down to take advantage of the larger pile section in the zone of maximum bending produced by lateral loads.

In fender pile systems, it is good practice to avoid the use of bolted connections between piles, sheeting, bracking, and struts, because such fixed restraints tend to be destroyed when deflected by lateral impact.

STEEL PILES

Durability of Steel Piles

Steel piles embedded in relatively impervious earth, at least 2 ft below ground surface, will generally be free of corrosive effects because of insufficient atmospheric oxygen. Embedded steel piles may be subject to corrosion if the surrounding medium consists of coal, alkaline soils, cinder fills, or wastes from mines or manufacturing plants. Steel piles protruding from the ground are subject to rusting at and somewhat below the ground line. Steel piles protruding into fresh water are generally subject to little deterioration but usually experience severe deterioration in seawater. Corrosion is severest in the splash zone.

Corrosion of steel piles by electrolytic action is uncommon. Local electrolytic action and subsequent corrosion may occur in a saltwater environment where the steel pile forms one pole of a battery with the other pole in a dissimilar metal in close proximity. However, when steel piles are embedded in a concrete footing, and thereby insulated from stray electric currents from the superstructure, electrolysis is generally not a problem. Electrolytic deterioration of steel piles can be minimized or prevented by the application of a protective coating such as epoxy coal tar paint or by positive cathodic protection using either electrolytic or galvanic anodes.

Steel piles can be protected against corrosion failure at critical zones by an increase in the steel cross section, or by encasements. Steel pile encasements have been made of poured-in-place concrete, precast concrete jackets, or Gunite applied before or after pile-driving.

Steel H-Pile

Steel H-piles are rolled steel sections with wide flanges so that the depth of the section and width of the flanges are of about equal dimension. The cross-sectional area and volume displacement of the H-pile are relatively small; consequently, they are well adapted to driving through compacted granular materials and into soft rock. Steel H-piles, because of their small volume displacement, have little or no effect in causing ground swelling or rising of adjacent piles.

The maximum length of steel H-piles is relatively unlimited. Unspliced pile lengths of 140 ft and spliced lengths of more than 230 ft have been driven. The optimum pile length is 40 to 100 ft. The recommended design stress for fully supported piles is 9,000 psi. The normal load range is 40 to 120 tons. Piles with heavy reinforced flanged sections have been driven to design loads of 200 tons and test loaded to 400 tons.

Steel H-piles are easy to splice. Splices can be either riveted, bolted, or welded, the latter being the most common procedure followed. It is desirable to keep splice material on the inner faces to avoid creating a hole in the ground larger than the pile section. This may result in a loss of frictional resistance. For hard driving conditions, splices should develop one-third the full strength of the section. Splices, in long piles with no lateral support, should develop the full strength of the section.

Caps are not usually required for steel H-piles embedded in concrete. Comprehensive tests conducted by the Ohio Department of Highways in 1947 indicated that uncapped H-piles embedded for only 6 in. into concrete proved as effective in transferring load as H-piles with cap plates.

The points of steel H-piles are sometimes tapered and generally reinforced when hard driving is anticipated or when they are to be driven to bedrock. Points are usually reinforced by welding plates to increase the thickness of the original section by a factor of $2\frac{1}{2}$ to 3.

Devices can be attached to a steel H-section to increase the bearing capacity of the pile to be driven into firm materials. Some devices that have been used consist of short sections of straight or wedge-shaped H-piling welded to the sides of the pile to increase the cross-sectional area at or just above the point.

Steel Rail Pile

Old rails have been used as piles by welding 3 rails together at heads or bases. The usual length of rail piles is about 30 ft. Sections of these rail piles have been butt-welded to fabricate a pile 90 ft in length. Rail piles are generally made of abandoned steel rails and are not considered normal steel production piles.

Steel Box Pile

Box piles have been fabricated from sections of steel sheeting in the form of a closed rectangular section. Because of their relatively large exterior dimensions, such piles can sustain large lateral loads and have been used to stabilize sliding banks. Box piles can be cleaned out and filled with concrete for additional bending strength.

Disk Pile

Disk piles have been fabricated of cast-iron pipe with a plate or casting of enlarged size connected to the base of the pipe. A disk pile has been fabricated with a pipe size of 9 in. and a disk diameter of 36 in. Such piles are usually jetted into position for end bearing on a firm stratum. Disk piles are rarely used today.

Screw Pile

Screw piles were used more extensively in the past than they are at present. The pile consists of an open-end pipe section to which is attached a number of turns of a helical shaft or screw at the base of the pipe. The pile is screwed or augered into the ground. Water jets are generally used to facilitate the advancement of the screw pile into the ground. A relatively recent screw pile installation involved a 42-in. diameter and $\frac{7}{8}$-in. thick shell to which was attached an 8-ft diameter helix at the tip of the pile. The steel shell was fitted with a conical point. Such piles were installed mechanically in 20 to 65 ft lengths. Screw piles can be installed with little or no disturbance to existing structures.

CONCRETE PILES

Concrete piles fall into 2 basic categories: precast and cast-in-place. Precast piles can be divided into the 2 general classes of normally reinforced piles and prestressed piles. Cast-in-place piles can be further subdivided into piles with casing and piles without casing. There are a number of variations of both of these basic types including a variation of cross-sectional area and longitudinal shape. Concrete piles are essentially unaffected by biological organisms or decay as are timber piles. They are thus used in foundations where the piles extend above groundwater or are immersed in river water or seawater. Depending on the foundation conditions and the type of concrete pile selected, the load carrying ability of the pile can be developed in either skin friction or point bearing or a combination of the two. Concrete cast-in-place piles, and more particularly prestressed concrete piles, can sustain high bending stresses and are frequently used in viaducts and trestle type of structures with the pile extending above ground or channel bottom level.

Durability of Concrete Piles

Plain or reinforced concrete piles embedded in earth are generally considered not subject to deterioration. The water table, if free from deleterious substances, does not affect their durability. In extremely infrequent situations, there is the possibility that concrete piles embedded in permeable soils may be damaged by groundwater saturated by either acids, alkalies, or chemical salts. These commercial agents can result from wastes discharging from manufacturing plants, sewer leakage, leaching from alkali soils, or leaching of acidic compounds from coal or cinder fill. The use of dense rich concrete with sulfate-resisting cement is a means of minimizing the effects of a deleterious environment. Concrete piles should not be used where severe deterioration could possibly result.

Concrete piles extending above the surface of a body of water are subject to damage from the abrasive action of floating objects, from ice where such exists, and from sand scouring. Damage can also result from frost action, particularly in the splash zone, and from internal corrosion of the reinforcement causing spalling of the concrete. The principal factors involved in these frequent types of failures are (a) composition and density of the concrete, (b) porosity of the aggregates, and (c) concrete cover over the reinforcing steel. Normally reinforced concrete piles are more vulnerable to spalling failure than prestressed piles because of inherent fine cracks in the concrete that develop from shrinkage, from handling of the piles, and from tension and shear loads.

The deterioration of concrete piles can be minimized by careful formulation of the concrete mix, use of sound, hard aggregates, and proper mixing, placing, consolidating, and curing to achieve hard dense concrete. The reinforcing steel should have a minimum cover of 2 in., and the use of galvanized reinforcing is advisible where economically permissible. Prestressing reduces cracks in concrete and should be used whenever possible.

Piles can be protected against some agents of deterioration by use of coatings and jackets applied to vulnerable areas. On a project under way in New Jersey where prestressed cylinder piles will be exposed to seawater, the interior and exterior surface of the piles are to be coated with an epoxy bonding compound immediately following sandblasting of the surface. The epoxy bonding compound is to provide a tight seal on the pile surfaces.

On a recently completed project in Long Island involving the use of prestressed concrete pile bents, the pile surfaces in the tide zone were protected by wrought-iron pile jackets grouted to the piles. Right-angle sections of $\frac{1}{4}$-in. thick wrought iron were bolted together to form a square jacket, and the $1\frac{1}{2}$-in. annular space between the jacket and pile was filled with grout put into place by a tremie.

CAST-IN-PLACE CONCRETE PILES

In general foundation work, the cast-in-place pile is more commonly used than the precast pile. Cast-in-place concrete piles generally need no storage space, are made in place to correct length, do not require special handling, and are not subject to damage from handling. Cast-in-place piles can be subdivided into 2 basic types: those that are formed in a steel shell in the ground and those that are uncased. Cased piles are the more positive type in that they permit an inspection of the pile prior to placing concrete, and allow for more accurate control in placing concrete. Uncased piles are generally more economical; however, they bear a great inherent risk in their installation.

Cast-in-Place Uncased Concrete Pile

Cast-in-place uncased concrete piles are load-carrying elements formed in the ground wherein the concrete is in direct contact with the soil. Such piles are recommended for use where soil or water will not fill or squeeze into the formed hole following the withdrawal of the forming mandrel or shell prior to the placement of concrete, or where the installation of adjacent piles may eventually damage the green concrete of piles already in place.

The following are types of cast-in-place uncased concrete piles that have been used in the past in this country and abroad but that are not often used at the present time.

1. MacArthur compressed concrete pile—The pile apparatus consists of temporary casing and solid mandrel that are driven together to the desired penetration, displacing the full cross section of the casing. The mandrel is then withdrawn and the casing filled with concrete. With the mandrel placed in direct contact with the concrete, the casing is withdrawn. Such piles are known as the MacArthur straight shaft pile and have been made in diameters ranging from 14 to 24 in. and in lengths up to 60 ft.

This pile can be formed with a mushroom base where it is advantageous to increase the bearing area of the pile. The mushroom base is formed by placing a charge of concrete in the casing, placing the mandrel on top of the concrete charge, raising the casing to the bottom level of the mandrel, redriving the casing and mandrel through the deposited concrete, and forcing the concrete into the surrounding soil to form an enlarged base. The placement of the concrete in the shaft of the pile is accomplished in the same manner as for the straight shaft pile.

2. Simplex concrete pile—The pile is formed by driving a casing with a heavy detachable conical point, displacing the full cross section of the casing. The shaft is filled with concrete, and the casing is withdrawn. Piles with 16- and 18-in. diameters have been so installed with load capacities ranging from 45 to 60 tons.

Tamped simplex piles are similar to the standard type except that the casing is struck with the hammer at short intervals as it is withdrawn, vibrating or tamping the concrete for its full length.

3. Francois express pile—These piles are formed by driving an 18-in. diameter steel casing with a removable point to the required penetration and charging it with concrete. The casing is raised, and the concrete is compressed by driving on a solid ram in contact with the concrete. Additional concrete is added, the casing is raised, and the concrete is compacted in stages producing a pile of varying diameter in accordance with the displacement of the soil as the concrete is laterally compacted.

4. Vibra pile—These piles are formed by driving a steel casing with a removable cast-iron shoe of slightly larger dimension than the casing to the required penetration. Concrete is placed in the casing, and the casing is alternately driven upward to extract it and downward to compact the concrete, which flows beyond the casing to the soil limits. This tends to produce a concrete shaft with a corrugated surface. These piles are currently used in Europe in diameters of 13 to 17 in. and carry loads of 40 to 60 tons. Piles up to 70 ft in length have been installed in this manner.

5. Ridley pile—These piles are formed with a steel casing and removable shoe essentially as described in paragraph 4. The casing is partially filled with grout, and a precast concrete pile, with a shoulder designed to a close tolerance with the inside diameter of the tube, is placed inside the casing and driven as the casing is withdrawn. The grout is forced out against the walls of the soil as the tube is withdrawn, filling the annular space between the precast concrete section and the soil surface.

Cast-in-Place Cased Concrete Pile

Cast-in-place cased concrete piles are formed by pouring concrete into a tapered or cylindrical form previously driven into the ground. The form or encasement could be a light-gage metal shell driven with a mandrel, or a steel shell heavy enough to be driven directly without a mandrel. Reinforcement is generally used in the upper section of the pile to take small bending forces that may develop. The use of the thin-shell pile should be carefully evaluated so that it is used in soils that will collapse or in places where it will deform because of soil displaced while adjacent piles are driven. The cased piles have an advantage in that the pile can be examined before it is filled with concrete. The placement of concrete, particularly in tapered piles, should be carefully inspected and controlled. There have been cases where such piles have been improperly filled, resulting in intermittent voids along the pile. Deformities or distortions in the pile shell could constrict the flow of concrete into the pile leading to the formation of intermittent voids.

The following is a description of the various mandrel driven cast-in-place cased piles in general use.

1. Raymond pile (standard type)—This tapered pile with a thin corrugated shell is driven with a solid mandrel bearing on the boot of the pile. The shell sections come in 8-ft lengths, with available shell thicknesses varying from 10 to 24 gage depending on the nature of the foundation materials. The optimum pile length is 35 ft, and the optimum load range varies from 30 to 60 tons. The pile is best suited as a friction pile.

2. Raymond step-tapered pile—This pile embodies the same fundamentals as the standard type with the exception that sections of the pile increase in diameter forming a series of steps. This permits increasing the length of the pile up to a maximum of about 100 ft without an excessive increase in the butt diameter. The piles are driven with a stepped solid mandrel.

3. Monotube pile—This is a fluted pile with a tapered steel shell and is best suited for friction piles of medium length. The shells are furnished in gages ranging from 3 to 11, and sustain direct driving with hammers of comparable size to those used for driving timber piles. The optimum length of these piles ranges from 30 to 80 ft with a load range of 50 to 70 tons. This type of pile cannot take excessive driving because the relatively light-gage steel shell will deform at the head. However, these piles can sustain lateral pressures from adjacent driving considerably better than the thin-shell piles.

4. Cobi pile—The casing of this pile is a thin-gage corrugated shell of uniform diameter driven in lengths up to 60 ft. This pile is driven with a Cobi pneumatic mandrel, which, when inflated, expands to a diameter slightly larger than that of the shell. The mandrel and shell are driven as a unit to the desired depth without a tendency to curve during driving. In this respect, the Cobi pneumatic mandrel-driven pile has an advantage over the standard mandrel-driven pile.

5. West's Rotinoff shell pile—The pile consists of a series of precast reinforced concrete shell sections joined together by steel bands and connected to a concrete shoe. The pile is driven by means of a mandrel bearing on the concrete shoe. Following removal of the mandrel, the pile is inspected and filled with concrete. The pile can be driven at locations with restricted headroom because the pile is assembled in sections. Piles of this type have been installed in lengths up to 100 ft. This pile was developed in Great Britain and has been used primarily in Europe.

6. Button-bottom cased concrete pile—This pile is installed by driving a thick-walled steel casing, usually 14 in. in diameter, plugged with a heavy concrete button having a diameter 1 in. larger than that of the casing, to a stratum of firm bearing. A corrugated shell with a flat plate at its base is placed inside the pipe. The flat plate has a center hole that fits over a bolt cast into the concrete button. To prevent floating or heaving during placement of concrete, the shell is anchored to the bottom by threading a nut over the bolt by means of a long socket wrench. The steel casing is withdrawn, and the shell is filled with concrete. These piles can take heavy driving through obstructions and derive their support primarily by point bearing. Piles 76 ft in length have been installed with design loads in the order of 50 tons.

7. Swage pile—This pile is formed by forcing a light steel casing, usually 11 in. in diameter and $\frac{1}{8}$ in. thick, over a tapered precast concrete plug so that the pipe is swaged out by the taper of the plug, forming a watertight joint. The pile is driven by means of a ram bearing on the plug inside the pipe, pulling the swaged pipe with the concrete plug. Following the removal of the ram, the casing is filled with concrete. These piles have been found to be advantageous in extremely hard driving conditions.

8. Closed-end steel pipe pile—This pile is formed by driving a steel pipe into the ground to the desired penetration, and filling it with concrete. The cylindrical steel pipe is of relatively heavy-gage wall thickness, generally ranging from $^5/_{16}$ to $^1/_2$ in. The pipe diameter ranges from 8 to 36 in. Seamless pipe is furnished in diameters up to 24 in., with spiral welded pipe available in larger diameters. Lap-welded pipe is sometimes used but is not recommended in driving through obstructions. The piles are driven with a flat plate or with a tapered cast-iron or steel point welded to the bottom of the pipe. Additional sections of pipe can be added by means of a cast-steel drive sleeve, permitting easy installation of piles of variable length. The optimum pile length ranges from 40 to 120 ft. The optimum load range is usually 80 to 120 tons. The piles are structurally capable of carrying large loads above ground level; the shell participates in carrying the load. The piles also provide high bending resistance under lateral loading. Pipe piles provide alignment control during installation and are capable of hard driving. This type of pile is used extensively in underpinning work because it can be installed in short sections by jacking. The advantages of this type of pile are offset by its relatively high cost.

9. Open-end steel pipe pile—These piles are similar to the closed-end pipe piles except that no closure is used at the tip of the pile. These piles are capable of being extended through obstructions because interferences can be broken or removed through the open pipe. The piles are used where soil displacements would be objectionable or where driving vibrations should be minimized. The open-end piles can be sunk to great depths to reach bedrock; piles more than 300 ft long have been driven. The open-end pipe piles are usually cleaned out with the aid of water jets and compressed air and then filled with concrete.

On a recently completed long-span bridge project in New Orleans involving very heavily loaded piers, 18-in. diameter open-end pipe piles were driven to a depth of 145 ft through clay strata in order to achieve settlement of the foundations. On this project, the settlement of the piers rather than the design loading on the piles dictated the required length of the open-end pipe piles. Only the upper 75 ft of the pile was cleaned and filled with concrete. The results of a series of pile load tests revealed that the piles were carrying load in skin friction and point bearing, indicating that the soil in the pile acted as a plug.

10. Pretest pile—This pile, which is used extensively in underpinning work, is a closed-end steel pipe that is jacked into the ground in sections by using the existing foundation as a reaction. When the pile has been jacked to the required penetration, it is inspected and filled with concrete. While maintaining the jacking pressure on the pile, the load is transferred to struts that are wedged between the top of the pile and the foundation. The transfer of the foundation load to the pile in this manner eliminates movement of the foundation comparable to the combined elastic deformation of the pile and the soil.

Rammed-in-Place Pile

The rammed-in-place pile is also referred to as the compressed concrete pedestal pile or pressure injected footing. The formation of this pile involves the installation of a casing with a temporary closed bottom to the desired penetration, the placement of a charge of concrete at the bottom of the casing, the lifting of the casing about 18 in., and the ramming out of the concrete to frame a bulb or pedestal at the base. The casing is then withdrawn as concrete is placed to form the shaft. Where soft soil conditions exist to prevent the proper formation of the shaft, the casing can be left in place, or a thin corrugated shell can be placed inside the casing to contain the shaft concrete and to permit the withdrawal of the heavier casing. The MacArthur compressed concrete pedestal pile is of this type as is the cased pedestal concrete pile. The rammed-in-place pile is advantageous where it is desired to spread load on a relatively thin bearing stratum. The Franki pile is also of this type and is extensively used in Europe, Canada, and the United States.

The Franki pile is installed somewhat differently from that described earlier. The casing is set on the ground, and a charge of dry concrete is placed in it. The concrete is compacted by means of a drop ram forming a dense plug that drags the casing into the ground to the desired penetration. The casing is held by cables, and the concrete is pounded, forcing it down and out of the casing to form a bulb. The shaft is formed by adding charges of concrete and ramming each concrete charge while gradually withdrawing the casing. This tends to form a corrugated surface along the concrete shaft. Piles of this type have been installed with shaft diameters ranging from 17 to 26 in. and for lengths of 10 to 60 ft. The optimum pile load ranges from 60 to 120 tons. The base of the pile cannot be formed in cohesive soils, and careful control is required in forming the shaft in soft soils to avoid discontinuities.

Formed-in-Place Pile

Formed-in-place piles are a relatively recent innovation and are generally installed by augering techniques. This type of pile is utilized in emergency repairs, in installations where it is not feasible or economical to utilize a pile-driving rig, or in installations where quarters are too cramped for using standard piles. The formed-in-place pile is currently gaining acceptance on large-scale projects. The advantages of this type of pile are speed and economy of installation. The following are descriptions of several formed-in-place piles.

1. Drilled pile—These piles are formed by augering to the desired depth. Where soil and groundwater conditions permit, the auger is withdrawn and the open hole is filled with concrete. In noncohesive soils and as otherwise required, the hole is formed as described, and the walls of the excavation are maintained by drilling fluid consisting of a mixture of bentonite and water. Such piles can be readily advanced past obstructions by means of chopping or coring with rock roller bits. The hole is filled with tremie concrete deposited through the drilling fluid. Piles of this type are usually 16 to 24 in. in diameter and generally extend to depths ranging from 40 to 60 ft.
2. Intrusion-Prepakt pile—These piles are formed by coring holes with an auger to the desired depth. In soft ground, the hole is lined with casing during augering. A ¾-in. grout injection pipe is centered in the augered hole, and 2½- to ⅜-in. aggregate is placed and tamped in the hole. Grout is injected through the grout pipe to solidify the aggregate mass. These piles have diameters of 12 to 24 in. and extend to a depth of 60 ft.
3. Intrusion grout mixed-in-place pile—This pile is formed in sandy soils by injecting grout through a rotating hollow drill rod with vanes attached at the bottom. The rotating rod mixes the grout with the granular soil as it penetrates the ground to the desired depth. Reinforcing bars can be pushed through the grouted soil mass after removal of the drill. These piles also have diameters of 12 to 24 in. and extend to a depth of 60 ft.
4. Augercast pile—This pile is formed by a continuous hollow-shaft auger rotated into the ground to the specified pile depth. High-strength mortar is pumped through the hollow shaft as the auger is withdrawn. Reinforcement can be placed while the mortar is still fluid. Twelve-in. diameter piles have been so installed to a depth of 60 ft to support a design load of 40 tons.

PRECAST CONCRETE PILES

Precast concrete piles find frequent use in marine installations and in foundations, bents, and viaducts where the piles need to extend above water or ground level. Precast concrete piles are generally cast in square or octagonal shapes. Circular (normally reinforced) precast concrete piles have been manufactured in the past, but are not currently in common use in this country. Precast piles can be manufactured of uniform cross section or tapered. The tapered piles are usually limited in length to about 40 ft. The uniform section piles are generally made with the lower few feet of the pile sharply tapered. Piles of larger cross section are frequently manufactured with a hollow interior to reduce weight. Pile reinforcement consists of longitudinal steel tied to spiral reinforcement or rectangular bars depending on the circular or square configuration of the longitudinal steel. The spiral and tie-bar reinforcement is generally placed at a closer spacing at the tip and head of the pile, relative to the middle section of the pile. The required longitudinal reinforcement is generally governed by the stresses resulting from handling of the pile. When the precast pile serves as a column or is subject to lateral forces, the amount of reinforcement is generally governed by the structure loads. Where jetting is anticipated in the installation of the pile, it has frequently been the practice to form a 3- to 4-in. diameter jet hole along the central axis of the pile.

Precast piles are available in sizes ranging from 12 to 24 in. and in lengths up to about 100 ft. The optimum pile load ranges from 40 to 60 tons and extends beyond 100 tons per pile for the larger sections. Among the disadvantages in the use of precast piles are the difficulty in handling, the cutting off of excess length, and the relatively high initial cost. The primary advantages include their suitability in marine installations, their ability to tend above groundwater level without inherent deterioration, and their ability to carry relatively high working loads.

COMPOSITE PILES

Composite piles were developed about 60 years ago to provide an economical pile of relatively long length. Composite piles consist of a lower section of one material joined with an upper section of another material. Each of the materials selected should be suited to the conditions of the in situ medium. Typical combinations of materials include (a) a timber section embedded in the ground below the permanent groundwater level and an upper section of concrete, and (b) a concrete-filled steel pipe or a lower steel H-section and an upper concrete section. All of the various cast-in-place concrete piles can be combined with timber or steel to form composite piles. Composite piles have also been fabricated with precast concrete upper sections. The precast section is cast with a recess in its lower end to receive the stub of the wood pile.

The following is an outline of the method of forming and installing a typical cased concrete and wood composite pile. The butt of a timber pile is tapered to form a tenon, which is wrapped with spiral wire. A casing and solid core are simultaneously driven to a depth well below groundwater level, and the solid core is removed. The wood pile with the wire-wrapped tenon is inserted in the open casing and driven close to the casing bottom by means of a follower. A corrugated metal shell with a reinforcing cage connected to its base is lowered in the casing and placed over the tenon of the wood pile. The shell is held to prevent uplift, and concrete is placed to fill the shell to ground level. The outer casing is removed, leaving in place a cased concrete and wood composite pile.

The optimum length of composite piles is in the range of 60 to 120 ft. Lengths of up to 180 ft have previously been driven. The optimum load range is 30 to 80 tons with a maximum load limited to about 150 tons. The primary disadvantage of the composite pile is the difficulty to attain a good joint between the 2 materials. Its major advantage is its comparatively low cost for the long pile lengths attainable. However, the extra labor involved in the fabrication of a composite pile somewhat neutralizes its economic advantage in this country.

PRESTRESSED CONCRETE PILES

The prestressed concrete pile is used extensively in this country and abroad and is rapidly replacing the standard reinforced precast concrete pile because of its many advantages. Prestressing essentially eliminates open cracks in a concrete pile, and this is most significant in seawater installations. Prestressing permits considerable ease in handling and reduces the tendency to spall during driving. The compression induced in the pile because of prestressing permits such piles to sustain considerable bending stresses.

Physical Characteristics

The design and manufacturing details of square and octagonal prestressed concrete piles and prestressed concrete hollow cylinder piles have been standardized by a joint committee of the American Association of State Highway Officials and the Prestressed Concrete Institute. The square and octagonal piles are made in sizes ranging from 10 to 24 in. The piles can be manufactured with a center void for sizes larger than 18 in. The cylinder piles are manufactured with outside diameters of 36, 48, and 54 in. and with wall thicknesses of 5 and 6 in.

The pile reinforcement consists of high-strength wire in a circular or square pattern, enveloped in mild spiral steel wire or tie-bar reinforcement. The prestressing wire is pretensioned prior to the placement of concrete, and the prestressing force is released when the concrete has reached a strength of 4,000 psi. The 28-day concrete strength usually ranges from 5,000 to 7,000 psi. Concrete hardening is usually accelerated by steam-curing for economy of manufacture. For installations involving extremely long piles, sections of piles can spliced. One type of splice that has been used involves joining the adjacent pile sections with a group of 6 dowels embedded 2 ft into each end. A fast-setting plasticized cement fills the dowel holes and the space between the pile sections.

Another type of frequently used prestressed pile is the Raymond cylinder pile. This pile is manufactured by joining together a series of hollow cylindrical precast sections, each section reinforced with a small amount of longitudinal and spiral steel to facilitate handling. Longitudinal holes for the prestressing wires are cored in the walls of the sections. These piles are fabricated to the desired length by joining the sections and post-tensioning them by stressing the cables running through the cored holes. The ends of the sections are previously sealed with a plastic joint compound, and the cable holes are subsequently pressure-grouted with cement. Stress is transferred to the pile by releasing the external cable-pull after the cement grout surrounding the cables has attained the proper strength. The tensioning cables usually consist of twelve 6-gage, high-tensile strength wires. However, the number of tensioning cables and wire strands in each cable can be varied to suit the design loading. The piles can be manufactured in diameters ranging from 24 to 90 in. and with wall thickness ranging from 4 to 7 in. The standard pile diameters are 36 and 54 in. Piles of this type have been made in lengths exceeding 200 ft. The piles are generally driven open-end and can carry loads exceeding 200 tons. The piles can be subjected to bending moments of considerable magnitude and find extensive use in structural bents over land and water.

An unusual pile installation in New Jersey involves the use of 36-in. diameter Raymond cylinder piles in a fender system. The piles are joined together by a heavily reinforced cap, and the connection between the cap and piles is designed as a flexible system. Thus, the fender system permits the participation of all the piles in resisting lateral load and in dissipating the impact energy by deflection of the entire system.

Problems in Installations of Prestressed Piles

Prestressed concrete piles are inherently weak with respect to radial stresses, with resulting tensile stresses being carried only by the nominal spiral reinforcement. An illustration of this occurred on a project where the contractor elected to use a sonotube (cardboard) form to create a 4-in. tubular jetting void in the interior of a 24-in. square pile. In the steam-curing of the piles, the sonotube distorted, producing a nonuniform opening. In the jetting process, the form collapsed and caused a sudden blockage of water resulting in internal tensile stresses that cracked the pile longitudinally. This problem was eliminated with the substitution of metal pipe for forming the void. The internal jetting of the piles worked very well, achieving accurate alignment control in the installation of vertical and batter piles.

The possibility of developing internal tensile stresses applies also to the Raymond prestressed cylinder piles and similar types of thin-walled concrete piles. Care needs to be exercised in the installation of these piles in that jetting and driving operations need to be controlled to prevent the buildup of excessive internal pressure.

An unusual failure has occurred in a pile splice, similar to the one just described, because of excessive driving energy in advancing a long pile through relatively soft foundation materials. The splice failed as the pile was driven by the full driving energy of the hammer through soft materials. The resulting substantial compression wave in the pile not meeting sufficient resistance at the pile tip produced a tensile wave in the pile as the driving energy traveled out of the pile tip. The resulting tensile stress caused failure of the pile at the splice. This problem was eliminated by substantially reducing the driving energy in extending the pile through the soft foundation materials.

CAISSON PILES

Caisson piles can be generalized as large-diameter, cast-in-place, open-end, cased concrete piles. The pile diameters can range from 12 to 36 in., and the casing may or may not remain as a part of the load-carrying element. Casing, where used, is usually thick-walled. Caissons are designed to carry extremely heavy loads to extreme depths. A description of some caisson piles that typify those in current use follows.

1. Western caisson pile—These caisson piles are installed by driving an open-end heavy steel pipe to bedrock or to a firm stratum of high bearing value. The pipe is cleaned and filled with concrete if dry, or by the tremie method if water is present. The pipe is pulled for reuse.
2. Calweld drilled foundation—This caisson pile is formed by a self-contained boring rig that can drill holes 10 to 72 in. in diameter up to 200 ft deep. In advancing through submerged granular materials, drilling mud is utilized to keep the hole open. Where soil conditions permit, bells up to 80 in. in diameter can be cut into the soil above the founding level by a belling bucket. The shaft is filled with concrete in the dry or by means of a tremie pipe through the drilling mud.
3. Drilled-in-caisson—This type of installation comprises a steel casing, steel H-pile core section, and concrete shaft. The drilled-in-caisson is a composite fixed-end column terminating in a rock socket, transferring load by direct bearing and bond to solid bedrock. An open-end casing is advanced to bedrock, and a churn drill is used to remove rock to form a socket. The depth of the socket is related to the magnitude of the load to be carried and the bearing and shear values of the rock. A concrete seal is placed at the base of the socket, the steel H-pile core section is centered in the socket, and the casing is filled with concrete. Drilled-in-caissons are usually 24 or 30 in. in diameter and have been installed in lengths of more than 140 ft. The composite section of steel casing, steel H-pile core, and concrete carry loads of the magnitude of 1,500 tons.

SUMMARY

Only the more significant details of the various types of piles available could be reviewed in this paper. The types of piles discussed do not by any means constitute a complete and comprehensive review of the subject. They are considered, however, to be a generalized representation. An economic comparison of the various types of piles has not been presented because of the variables involved with respect to time, place, availability of materials, and other pertinent factors. A relative economic comparison among types of piles was presented whenever possible.

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Pile Driving: Hammers and Driving Methods

GEORGE J. GENDRON, Raymond International, Inc.

The heart of any pile-driving system is the pile hammer. Modern contractors use impact types ranging from the "ancient" drop hammer, through single- and double-acting hammers, to differential hammers. Steam and air are still the basic sources of power for hammers, but lately diesel hammers and high-pressure hydraulics have gained acceptance. Because a constant energy source is seriously affected by pile cushions of varying characteristics, "permanent" cap blocks are now in widespread use. Low-frequency vibrators are used primarily for driving nonbearing piles and for extracting sheet piles. High-frequency (resonant) vibrators, though currently expensive to purchase and operate, have much wider fields of application including the driving of displacement bearing piles. Pile-driving systems of the future will include larger hammers (250,000 ft-lb or more) with self-contained power sources, both diesel and steam, and simple, less expensive but more reliable high-frequency, high-power vibrators.

•FOR THE PAST HUNDRED YEARS engineers have been struggling to convert the art of pile driving into a science. In the past 20 years or so its growth as a science has caused a demand for the improvement of old tools as well as the development of new, specialized equipment so that the practical side might keep pace with the theoretical. The heart of any pile-driving system is the pile hammer. Its history is as old as pile driving itself, having its beginning in the sledges of prehistoric man and the drop hammers of ancient Rome.

This paper is primarily concerned with the most common types of hammers in existence today, but some attention is given to those special systems that have seen limited use during the past decade and may be the nuclei of the driving systems of the future. Drop hammers are the oldest type of hammer and are still used today because of the simplicity of the pile rig required for their application. From the early beginnings of pile driving little changes were made in hammers until the steam age of the nineteenth century, when the single-acting steam hammer was developed. As the use of single-acting steam hammers became widespread, pile men felt the need for a power hammer that would strike a greater number of blows per minute, delivering energy at a faster rate. This lead to the development of the double-acting hammer. Because of the high-velocity, sometimes impractical blow of these hammers, engineers have been cautious about their application on bearing piles. This desire for driving speed coupled with a low-velocity blow lead to the development, 35 years ago, of the differential-acting hammer. Single-acting, double-acting, and differential-acting are the 3 major types of hammers in common use today. Although primarily designed as steam hammers, they are often used with air or hydraulic fluid as a source of power. The diesel hammer, developed prior to World War II, has come into prominence lately because of its self-contained power source.

This paper also discusses cap blocks, the cushion interposed between most pile hammers and the pile, and vibrators, both the low-frequency type commonly used to drive sheet piles and the high-frequency or resonant vibrators that caused quite a stir when introduced commercially 5 years ago.

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DROP HAMMERS

Pile hammers originated with the sledges of prehistoric man and the drop hammers of ancient Rome. Drop hammers are still in use today primarily because of their simplicity of operation. Their advantages are many. However, so are their disadvantages.

Among the advantages of a drop hammer is the simplicity of the operating system. The hammer requires no specialized power source but uses the main hoist of the pile rig. The lack of mechanical parts makes for the simplest of maintenance requirements. When the hammer is handled by an experienced operating engineer, the hoisting line is the only part requiring periodic replacement.

Except for the Franki pressure-injected footing system, the drop hammer sees little use on domestic pile jobs today. This is primarily because of its low frequency, 5 to 10 blows per minute, and the difficulties attendant to the delivery of a consistent, measurable blow. In the Franki system, where large strokes are common, variations of several inches of stroke have a minor effect on the energy of the delivered blow. When bearing piles are driven, however, experience, and of late theory, has shown that low-velocity blows are desirable, the acceptable maximum being 36 to 39 in. In these cases variations of several inches in the stroke can have a measurable effect on the energy of the blow.

The blow of drop hammers can also be adversely affected by other factors. For one, the drag exerted on the hammer by the handling line can vary from pile to pile depending on the friction in the hoisting system. For another, even an experienced operator will occasionally prematurely engage the friction on the hoisting drum, dampening the blow.

Drop hammers are commonly used overseas to drive bearing piles. The most common application is on precast, prestressed piles. The ability to "tap" the pile when little or no point resistance is present and thereby to avoid excessive tension stresses represents an advantage of this particular hammer.

Regardless of the simplicity of drop hammers, the economic demands placed on the modern pile contractor relegate it, at least in the United States, to a minor role.

SINGLE-ACTING HAMMERS

The single-acting hammer, a product of the steam age, has seen only superficial changes in design since the early 1900's. These hammers are simple and reliable, develop consistent energies when the stroke is adequately controlled, and possess, what from the contractor's standpoint is a major advantage, a large ratio of weight of ram to total weight of hammer. Most of the empirical pile-driving formulas in use today are based on this type of hammer.

The hammers are mechanically reliable. Years of experience have produced a series of low-maintenance designs that produce hammers that require little care on the part of the operator and little concern on the part of the engineer. They are moderate speed devices usually rated in the range of 60 to 75 blows per minute.

The single-acting hammer can be conveniently and economically short-stroked for the driving of precast and prestressed concrete piles. Mechanisms are now available to make it possible to remotely shift from a short to a long stroke in a matter of seconds. There is, however, the ever-present possibility of oversupplying short-stroked hammers with steam or air, and temporarily reduced energy blows are not to be considered reliable as to rated energy. Factors that may contribute to substandard operations of single-acting hammers are as follows:

1. Improper valve timing—This results in premature admission of steam (cushions the blow) and throttling of the exhaust (shortens the stroke). Usually improper valve timing results in decreased frequency of blows of the hammer. It should be noted, however, that there is really no direct relationship between the frequency of the hammer and the energy of the blow.

2. Excessive mechanical friction—There have been cases documented where the hammer ram has actually "hung-up" because of excessive packing friction. Adjustment of the gland beyond that required to just reduce excessive leakage can reduce hammer energy.

3. Variations in the location of the striking point—When the location of the striking point (the top of the cap block or cushion block) is too high, the valve of the hammer might not be thrown completely, and as a consequence the single-acting hammer will short-stroke. When the striking point is too low, the ram has to travel an excessive distance after the valve is thrown at the bottom of the stroke until it strikes the pile, and the blow can be cushioned by the upforce of steam. In spite of its age and shortcomings, the single-acting hammer is still the mainstay of today's pile contractor.

DOUBLE-ACTING HAMMERS

Double-acting hammers use steam or air to raise the striking parts and also to impart energy during the downstroke in addition to that supplied by gravity. The basic design was developed out of a desire on the part of engineers for a greater number of blows per minute. The double-acting hammers in common use today operate in a range of 100 to 250 blows per minute.

To provide higher frequencies double-acting hammers are usually designed with light rams. A large percentage of the energy rating of the hammer is due to steam force. These hammers are, therefore, extremely sensitive to system pressure.

Double-acting hammers strike a relatively high-velocity blow compared to single-acting hammers. Theoretical investigations have shown this to be extremely inefficient in the driving of heavy piles. Although contractors consider these hammers desirable because they have a high-energy rating compared to other hammers of equal total weight, they are not often used for the driving of bearing piles. Their use is commonly limited to the driving of sheet piles or soldier beams.

Because they are usually of the closed design where the ram is not visible, it is extremely difficult, if not impossible, to monitor the stroke of the hammer. Tables are, however, available indicating "rated energy" versus blows per minute for these hammers. Unfortunately, these are extremely unreliable because factors other than the energy of the blow affect the operating speed of these hammers. For example, (a) a hammer that short-strokes will usually produce a higher frequency of blows than one that delivers the rated stroke, and (b) double-acting hammers operating on a springy pile will usually increase in frequency as resistance increases, requiring the operator to throttle the hammer and consequently the blow.

The double-acting hammer has to be classed as a special-purpose tool. However, properly applied it becomes a necessary valuable part of the equipment of the pile-driving contractor.

DIFFERENTIAL-ACTING HAMMERS

Employment of relatively heavy rams in pile driving results in low-impact velocity blows that not only conserve more of the available energy, but also prevent undue damage to the pile. Because of this, as much as possible of the total weight of a hammer should be assigned to the striking parts in order to most efficiently utilize the maximum permissible equipment weight. It is also desirable to have a hammer strike as many blows per minute as possible in order to further reduce the cost of driving.

A single-acting hammer meets the heavy ram requirements. It lacks, however, the desirable high frequency of blows. A double-acting hammer operates with a rapid succession of blows; but, when compared with a single-acting hammer of the same total weight, its much higher velocity impact is less effective.

In the differential type of hammer the deficiencies of single- and double-acting hammers are overcome while the advantages are maintained. This is the result of its steam cycle that is different from that of any other hammer. This cycle makes the lifting area under the piston independent of the downward thrusting area above the piston. Therefore, regardless of how large a portion of the total weight is contained in the striking parts, sufficient force can be applied for lifting and accelerating these parts without affecting the deadweight needed to resist the reaction of the downward accelerating force.

An explanation of this is as follows: The upward steam force in the differential hammer can be increased by increasing the size of the larger piston. The reaction for this force is carried through the hammer frame into the follower and head of the pile. The downward steam force uses for its reaction the entire deadweight of the frame of the hammer.

This produces an interesting characteristic of differential hammers. The maximum energy per blow that can be developed by a differential hammer is the total weight of the hammer times its stroke. The proof of this is straightforward. Hammer energy is equal to the total downforce times the stroke of the hammer. This downforce is made up of the weight of the striking parts times the stroke, plus the weight of the downward steam force times the stroke. The maximum limit for the steam force is that of the reaction furnished, or the deadweight of the nonstriking parts of the hammer. The maximum energy of the hammer is, therefore, the sum of weights of the striking parts and nonstriking parts times the stroke.

Differential hammers may be short-stroked in a manner similar to single-acting hammers. However, if the short-stroking is to be permanent, that is for the life of a job, economical hammer operation can only be ensured by putting a filler or "dummy" under the cylinder head. The reason is obvious if one realizes that in the differential hammer the amount of steam vented to exhaust on each stroke, and thereby "consumed" by the hammer, is the volume above the upper piston at the time the hammer strikes. A short-stroked differential hammer without a filler in the cylinder will exhaust the same volume of steam per blow as a full-stroke hammer. The results will be decreased energy output with the same energy input.

Fillers in the cylinders of short-stroked differential hammers offer another advantage besides economy of operation. They guarantee a mechanical limit for the hammer stroke and prevent accidental overdriving.

Differential hammers are today's hammers. Their economy of operation finds favor with owners and contractors. Their efficiency and reliability find favor with engineers.

HAMMER POWER SOURCES

Steam

Steam has been a prime power source for pile hammers since before the turn of the century. It is becoming increasingly difficult, however, to find qualified firemen, and boiler maintenance is in danger of becoming a lost "art." Coupled with these disadvantages are problems relating to local smoke ordinances that all but rule out contractor's boilers, the difficulties of cold weather operation, and the ever-present need for large quantities of clean water. On a commercial crane pile driver, a boiler represents a second power source, one that idles while the main hoist works; and the crane engine idles while the hammer works. All of this leads contractors and equipment manufacturers to search for other power sources.

Air

Modern air compressors answer many of the objections to contractor's boilers. A few years ago it was difficult to obtain portable compressors of adequate size and pressure rating. However, today, even the largest hammer can be operated by paralleling compressors when necessary. The modern compressor is a clean tool, and its maintenance and reliability are constantly being improved. It can be run by most operating engineers, requires little or no start-up time, and eliminates the need for large quantities of water. Winter operations of compressors have become commonplace. The efficiency of compressors for pile-hammer operation has been increased in recent years by the introduction of an after-heater system that uses the heat of the compressor engine exhaust gases to add energy to the air and thereby increase the output of the system. Compressors are, however, not without their disadvantages. The initial investment required can be 4 to 5 times that of a boiler with similar capacity. Operating costs are higher than those of a boiler, and compressor complexity adds to the contractor's maintenance work load. Finally, air compressors still represent a second power source on a pile driver.

Hydraulics

In the early 1960's the Raymond Concrete Pile Division of Raymond International, Inc., introduced a line of hydraulically powered differential hammers. Today over 25 of these are in operation. Their hydraulic power source offers the following advantages:

1. The pumps for the hammer system can be driven by the main engine of the pile driver. Because this engine would normally idle during most of an air- or steam-hammer driving cycle, the increased fuel and maintenance costs generated by the pump load are relatively insignificant.
2. The use of 5,000 psi pressure allows for small-sized hydraulic components, making possible completely built-in power pack systems.
3. The "closed loop" hydraulic system eliminates the external exhaust present in other pile hammers. No airborne contamination is generated, and hammer noise is appreciably reduced.
4. The hammer power pack can operate other pile-driver accessories such as drills for pre-excavating, hydraulic spotters, and auxiliary hydraulic hoists.

DIESEL HAMMERS

In recent years German, Japanese, and American equipment designers have produced bigger and better diesel hammers. When the diesel hammer was first introduced just prior to World War II, its mechanical reliability was questionable. Many times the hammer would not start. It was common for it to occasionally skip a blow. Most of these mechanical shortcomings have been overcome.

A diesel hammer is close to an ideal pile-driving package for the contractor. It gives him a single piece of equipment combining his power source and hammer. With a commercial crane, a light set of leaders, and a diesel hammer, he is in business.

There is apparently no limit to the size of diesel hammer that can be designed and built. Hammers with energy ratings of more than 100,000 ft-lb are available. There is, however, still some question about the blow delivered by diesel hammers. Arguments have been advanced for and against its special characteristics.

The blow of a diesel hammer is complex and starts with an initial force induced in the head of the pile by the compression of the air and fuel prior to ignition. The pre-compression force is followed by the actual blow of the ram that starts to accelerate the head of the pile downward. Almost simultaneously with the blow, diesel ignition occurs and the force of the explosion accelerates the ram upward and pushes the pile head downward. A lingering push is applied to the head of the pile as the products of combustion expand and continue to push the ram upward and the head of the pile downward.

Proponents of diesel hammers maintain that all of these factors contribute to a very efficient transfer of energy from a falling ram to a pile and that even the lingering force of explosion keeps the pile in motion and increases the penetration per blow. Opponents of the diesel hammer maintain that pre-ignition may cushion the blow of the hammer and that incomplete combustion can produce erratic hammer action.

Much is yet to be learned about these hammers. Several extensive test programs seem to indicate that the hammers deliver energies close to their rating. There are, however, other comparison driving tests that seem to indicate the contrary and cause many engineers to be extremely cautious about their application and use in connection with most "conventional" pile formulas. There is no doubt, however, that the diesel hammer is here to stay and that its reliability will soon be sufficiently improved and enough experience gained in its use for it to receive universal acceptance.

CAP BLOCKS

No discussion of driving systems would be complete without some mention of the cap block or cushion block commonly interposed between a hammer and the pile. This assembly performs 2 major functions: (a) It protects both the pile and the pile hammer; and (b) it modulates the blow of the hammer, eliminating extremely high, inefficient, and possibly injurious peak forces, and transfers the energy of the moving ram to the pile more in terms of a push than a sharp rap.

Constant cap-block characteristics are almost a necessity when penetration per blow is used as a driving criteria. All of the empirical formulas used to determine the rate of penetration equivalent to a particular dynamic resistance assume that cap-block characteristics are constant.

For many years wood was the mainstay of the industry. It was found, however, that the type and the amount of wood used had an effect on the cushion's characteristic and that this characteristic further varied throughout the life of the block itself. This variable characteristic together with the high cost of the consumable wood block lead contractors to the development of the so-called permanent cap block. Typical of these is the micarta-aluminum combination developed by Raymond International, Inc. It not only possesses the springy constant found desirable in the old "standard" wooden cap block, but also has a higher coefficient of restitution: a measure of the efficiency with which the cushion can transmit the hammer blow to the pile.

VIBRATORS

Low-Frequency Vibrators

In a search for faster and more efficient means of installing piles, engineers began experimenting in the United States with the use of vibrations in the early 1950's. (In Russia and Germany experimental investigations were made prior to 1936.) Little came of this until the early 1960's when several low-frequency vibrators were introduced.

These vibrators operate in the range of 5 to 35 cycles per second and deliver their energy by lifting the entire pile and driving it downward on each cycle. The vibratory input tends to reduce the frictional grip of the soil on the pile and the pile itself is used to impact the soil and overcome point resistance. In recent years these tools have been increasingly used in the driving of "nondisplacement" piles. The application of the tool to closed-end pipe, shell, and precast piles has been very limited for 2 reasons: (a) Displacement piles are usually bearing piles and as yet no dynamic formula has been universally applied to make it possible to correlate either vibrator output or rate of penetration with dynamic pile capacity; and (b) the ability to overcome resistance under the pile point depends on the vibrator's maximum output force, the mass of the pile, and the amount of damping in terms of side friction that the soil presents. For most displacement piles the power required is beyond the capabilities of all but the largest vibrators.

The hammers have, however, been used extensively to drive and pull sheet piles, soldier beams, and open-end pipe. Almost every year new, larger units are available that cannot help increasing the vibrator's area of application.

High-Frequency (Resonant) Vibrators

In the early 1950's Bodine introduced the concept of resonant pile driving. This system utilizes oscillators having an operating range of 40 to 140 cycles per second. These oscillators make it possible to vibrate a pile at its natural frequency. The resonant theory holds that the mass of the pile does not dampen the oscillator's output but that the pile acts as a "transmission line" that maximizes the ability of the tip of the pile to do work on the soil. The vibratory imput also reduces or eliminates side friction.

A number of successful jobs have been completed by using this tool. Displacement piles over 100 ft in length have been successfully driven. The system offers the following advantages to the contractor and the engineer:

1. The ability to drive lighter section piles than can be driven with an impact hammer;
2. Increased speed of installation;
3. Elimination of the impacting noises present in conventional hammers; and
4. Operation far above the natural frequency of the soils on a particular site, eliminating or at least reducing the amount of vibrations felt by adjacent structures.

The system is not without its drawbacks. The cost of the equipment is high. The complex construction of the oscillator increases the maintenance cost of the tool and adversely affects its reliability. It is only a matter of time, however, before less complex high-frequency oscillators are developed that will eliminate these objections and increase the number of applications for resonant driving. Vibrators will probably, however, remain special-purpose tools.

DRIVING SYSTEMS OF THE FUTURE

For the short term the writer expects to see the following:

1. Diesel hammers—As their design is further refined, their reliability is increased, and a background of experience is developed, diesel hammers will be adopted by more pile contractors. Their "all-in-one-package" feature will make them especially attractive for the small jobs and highway work where many equipment moves are required. As diesel hammers of larger energy ratings become available, their use in driving heavy bearing piles for design loads of more than 100 tons should become common.

2. Large steam hammers—Only a few years ago piles of 70-ton design load capacity were considered exceptional. Today loads of 200 tons per pile are not uncommon. In a few years it is to be expected that piles loaded to 300, 400, and even 500 tons will be replacing expensive caissons for high-column loads. This will make the use of larger hammers mandatory.

In offshore construction, loads of 1,000 to 1,500 tons are already commonplace, and hammers with 60,000-lb rams and rated energies of more than 150,000 ft-lb are being used. Hammers with rams weighing 100,000 lb and ratings of more than 250,000 ft-lb are already on the drawing boards and will be introduced during the 1970's. As the design loads of dry-land piles are forced upward for economic reasons, it is only a matter of time before these large hammers move onshore.

3. Hydraulic hammers—The practicality of hydraulics as a source of power for pile hammers has been proven during the 1960's. To date the largest hydraulic hammer has a rated energy of 24,500 ft-lb. A 75,000 ft-lb hammer has already been designed. There seems to be no limit to the size of the hammer to which hydraulics is applicable. If the pile drivers of the 70's are to be as mobile as those of the 60's, and equipped with as many auxiliaries, hydraulic hammers are bound to see wider application.

4. Vibrators—Larger low-frequency vibrators and more reliable high-frequency vibrators are to be expected in the 1970's. Although these tools will probably always have limited application, they are just beginning to be applied. Oscillators of the high-frequency or resonant type will probably make use of linear hydraulics, and this should make for a high force output from a small package and minimize the number of moving parts in the system.

For the long term, who really knows just what driving systems will be developed? Nuclear power systems are a possibility. Another is single-package steam generators and hammers. Chemical generation of steam is now possible with convertors in the 70- to 100-hp range no larger than a roll of plans for some medium-sized job and those in the 250- to 1,000-hp range smaller than a 55-gal drum. Generators of this type directly mounted on a hammer will offer the same advantages in steam now enjoyed by diesel and yet not alter the operating characteristics of the steam hammer.

One thing we are sure of: The equipment conceived in the past and refined in the present will do the bulk of the work in the future.

Pile-Driving Formulas

ERNEST T. MOSLEY, Raymond International, Inc.; and
TONIS RAAMOT, Raamot Associates

The basis for the fundamental dynamic pile-driving formulas is presented first. Then modifications to account for energy losses are described. Some of the more popular pile-driving formulas are discussed with emphasis on their inherent assumptions. Finally, a solution is described that is based on the motions experienced by all parts of the hammer-cap block-pile system after hammer impact occurs. This solution is commonly known as the wave equation. The assumptions on which this solution is based are discussed, and some of its advantages are described.

•A GREAT DEAL HAS BEEN WRITTEN about dynamic pile-driving formulas. One can find numerous formulas listed in soil mechanics and foundation engineering texts. Furthermore, one frequently sees reports in technical journals of new investigations by researchers who have found none of the existing formulas to their liking and have developed a new one that better suits the results of their personal experience. Yet most of the texts on this subject caution the engineer to be wary of the use of dynamic pile-driving formulas inasmuch as they cannot be relied on to predict a pile's ultimate capacity with a reasonable degree of accuracy.

Why, then, is there this continued interest in these formulas. The reason is, of course, that there is a practical need for a dynamic formula. The majority of pile foundations are installed in soil profiles of such a variable nature that the only convenient method for determining whether or not a pile has reached sufficiently dense material is by observing the ease or difficulty with which the pile penetrates the ground. Another reason for this continued interest is that no single dynamic formula has been found that will consistently predict ultimate pile capacities that agree with load tests. This second consideration is based on the often invalid assumption that no change occurs in the soil's ability to resist further pile penetration from the time of driving to time of load testing.

In August 1960, Smith (1) presented a practical means for calculating the response of a pile to the impact of a hammer by means of finite-difference equations making use of electronic digital computers. The method of solution is such that one can conveniently account for all of the significant factors that influence pile penetration resulting from a hammer blow. During the past few years a considerable amount of research related to wave-equation solutions has been done at the Texas Transportation Institute (2, 3, 4) and has included comparisons of wave-equation solutions with data obtained from instrumented piles during driving. These investigations have confirmed a good correlation between experimental and wave-equation solution results. It is the writers' opinion that, of solutions currently available, this one is the only reliable method for calculating the response of any particular cap block-cushion, pile, and soil system resulting from the impact of a pile hammer. However, at the present time a variety of dynamic pile-driving formulas are being used for this purpose in building codes and specifications. The intent of this paper is first to review basic forms of the dynamic pile-driving formulas and the assumptions on which they are based. Then, a parameter study will be presented comparing 3 typical pile-driving formula solutions with corresponding wave-equation solutions.

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The main requirement of any dynamic pile-driving formula is to correlate driving resistance, usually recorded in blows per inch, with soil resistance encountered at the time of driving. The main limitation is that a dynamic formula can only calculate the soil resistance at the time of driving. Any changes in the soil's ability to resist further pile penetration some period of time after driving must be evaluated by static methods and added to, or subtracted from, the resistance encountered at the time of driving.

Most of the dynamic pile-driving formulas are based on the fundamental energy equation wherein the kinetic energy of the hammer's ram at impact is equated to the work done on the pile, that is, the product of the distance the pile moves and the soil force resisting this movement. However, several complicating factors must be accounted for; these include the following:

1. The resisting soil force is not constant during the period of time the pile is penetrating because the soil has some elasticity and damping characteristics.
2. The temporary elastic compressions of the cap block, cushion, pile, and soil absorb energy that does not contribute to making the pile penetrate.
3. An impact energy loss occurs because the cap block and cushion have coefficients of restitution less than unity.
4. A pile is a long slender object, and each incremental part at any instant of time will experience a different motion from that of the other parts.

The dynamic pile-driving formula most commonly used in the United States is the Engineering-News formula.

$$R = \frac{2E}{S + 0.1}$$

where

R = safe pile working load, lb;
E = hammer energy, ft-lb; and
S = pile set, in./blow.

This is one of the simplest formulas because it does not attempt to account for the first, third, and fourth complicating factors listed earlier, and furthermore it presumes that the elastic compression losses are the same for any cap block, cushion, pile, and soil. This formula is written in such a way that it has a theoretical factor of safety of six.

The second dynamic formula used for comparison is the Eytelwein formula.

$$R = \frac{2E}{S + 0.1(P/W)}$$

where

R = safe pile working load, lb;
E = hammer energy, ft-lb;
S = pile set, in./blow;
P = pile weight, lb; and
W = ram weight, lb.

It is slightly more complicated than the Engineering-News formula inasmuch as it considers the ratio of the pile's total weight to the hammer ram's weight in such a way that the first, second, and fourth complicating factors are not accounted for, but the combined coefficient of restitution of the cap block and cushion is assumed to be zero. It also has a theoretical factor of safety of six.

The third dynamic formula used for comparison is the Hiley formula.

$$R_u = \frac{12eE}{S + (1/2)(C_1 + C_2 + C_3)} \left(\frac{W + \mu^2 P}{W + P}\right)$$

where

R_u = ultimate pile capacity, lb;
E = hammer energy, ft-lb;
e = hammer efficiency;
S = pile set, in./blow;

C_1 = compression of cap block and cushion, in.;
C_2 = compression of pile, in.;
C_3 = soil quake, in.;
W = ram weight, lb;
P = pile weight, lb; and
μ = coefficient of restitution of cap block and cushion.

C_1, C_2, and C_3 are based on Chellis (6) as follows:

Steel	Concrete
$C_1 = 0.16\ (R_u/A)$	$C_1 = 0.5\ (R_u/A)$
$C_2 = 0.0008\ L\ (R_u/A)$	$C_2 = 0.008\ L\ (R_u/A)$
$C_3 = 0.1$	$C_3 = 0.1$

where

A = pile area, in.^{2}; and
L = distance from head of pile to center of soil resistance, ft.

This formula is more complicated than the other two because it attempts to account for the second and third complicating factors in a rational manner based on the physical properties of the materials used. However, it does not attempt to account for the first and fourth complicating factors. A more complete description of pile-driving formulas has been given by Cummings (5) and Chellis (6).

The mathematical model for wave-equation solutions can be readily designed to account for all 4 complicating factors. The basis is described by Smith (1).

Solutions for the 3 dynamic pile-driving formulas and for the wave equation are shown in Figures 1, 2, 3, and 4. Symbols used in Figure 4 are defined as follows.

μ = coefficient of restitution of cap block or cushion;
M/AL = Micarta-aluminum;
Q-point = soil quake at tip of pile, in.;
Q-side = soil quake at side of pile, in.;
J-point = soil damping constant at point of pile, sec/ft; and
J-side = soil damping constant at side of pile, sec/ft.

When comparing the 3 dynamic formulas with the wave-equation solutions, one should note that both the Engineering-News and Eytelwein formulas have built-in theoretical factors of safety of six. This means their intention is to correlate a safe pile working load with driving resistance. On the other hand, the Hiley formula and wave-equation solutions both correlate the ultimate soil resistance with the driving resistance.

A parameter study intended to bracket the range of commonly used steel and concrete piles driven for support of land structures has been selected as a means of comparing the 3 dynamic formulas with the wave-equation solution. The parameters used are as follows:

Pile material	Steel, concrete
Pile size, lb-ft	
Steel	20, 160
Concrete	
12 in.^{2}	150
18 in.^{2}	338
Pile length, ft	30, 120
Mode of soil resistance, percent	
Point	100
Friction	100
Hammer size, ft-lb	
No. 1 Vulcan	15,000
No. 2/0 Raymond	32,500

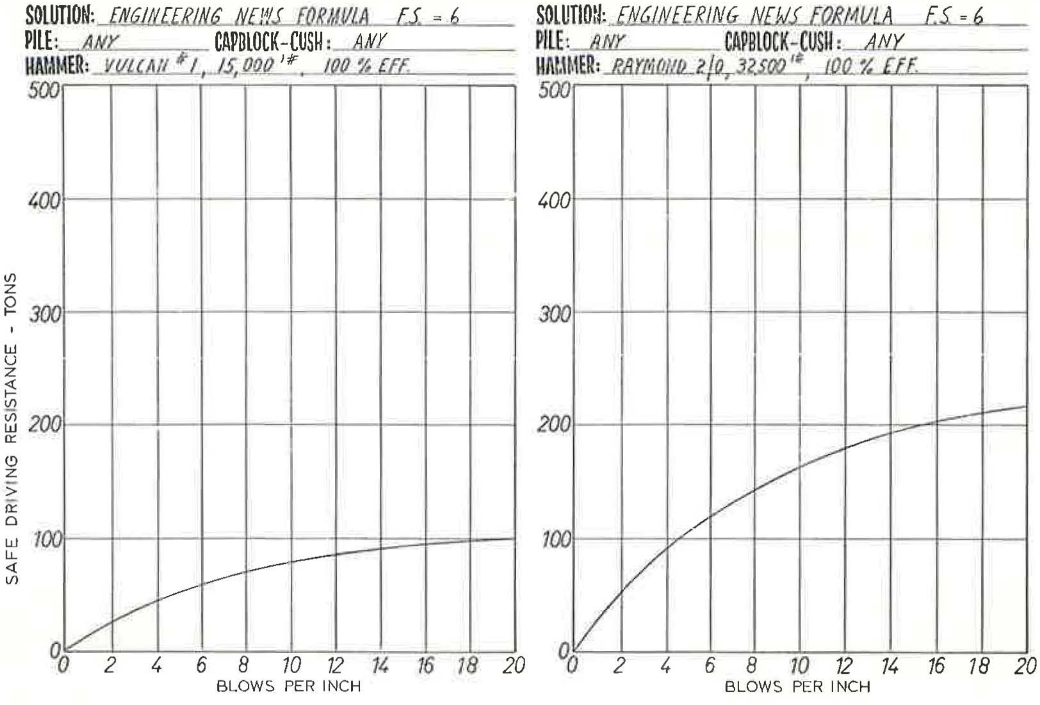

Figure 1. Solutions for the Engineering-News formula.

TABLE 1

RATIO OF WAVE EQUATION ULTIMATE RESISTANCE TO OTHER FORMULAS

Pile Type	Pile Size (lb-ft)	Pile Length (ft)	Vulcan 1				Raymond 2-0			
			Point		Friction		Point		Friction	
			5 Blows per In.	20 Blows per In.	5 Blows per In.	20 Blows per In.	5 Blows per In.	20 Blows per In.	5 Blows per In.	20 Blows per In.
Engineering-News Formula Safe Resistance										
Steel	20	30	1.9	1.4	2.4	1.7	1.5	1.0	2.1	1.2
Steel	20	120	1.4	1.0	1.8	1.1	1.0	0.7	1.3	0.7
Steel	160	30	2.6	3.0	3.8	3.7	2.7	2.3	3.9	2.7
Steel	160	120	2.6	3.0	4.1	4.1	2.5	2.3	3.9	2.7
Concrete	150	30	1.8	1.5	2.2	1.7	1.5	1.0	1.7	1.1
Concrete	150	120	1.9	1.7	2.4	1.8	1.6	1.2	1.9	1.2
Concrete	338	30	1.8	2.2	2.4	2.5	1.9	1.7	2.2	1.7
Concrete	338	120	1.9	2.3	2.4	2.9	2.0	1.8	2.6	2.0
Eytelwein Formula Safe Resistance										
Steel	20	30	1.4	0.6	1.8	0.7	1.0	0.4	1.4	0.4
Steel	20	120	1.3	0.7	1.5	0.7	0.8	0.3	0.9	0.3
Steel	160	30	2.7	2.9	3.9	3.6	2.1	1.4	3.0	1.8
Steel	160	120	5.4	9.0	8.5	11.8	3.1	3.5	4.9	4.4
Concrete	150	30	1.8	1.4	2.2	1.6	1.3	0.7	1.0	0.6
Concrete	150	120	3.6	4.7	4.3	4.9	2.1	1.8	2.4	1.8
Concrete	338	30	2.6	3.5	3.2	4.1	1.9	1.7	2.2	1.7
Concrete	338	120	7.1	12.7	8.6	16.0	4.2	5.3	5.4	5.9
Hiley Formula Ultimate Resistance										
Steel	20	30	0.8	1.0	0.9	1.1	0.8	0.9	0.9	0.9
Steel	20	120	1.1	1.3	1.0	1.0	1.1	1.2	0.9	0.9
Steel	160	30	0.9	1.6	1.3	1.9	1.0	1.3	1.3	1.5
Steel	160	120	1.2	2.3	1.8	2.6	1.3	1.9	1.7	2.0
Concrete	150	30	1.1	1.5	1.3	1.6	1.0	1.2	0.8	1.1
Concrete	150	120	1.9	2.7	2.1	2.5	1.7	2.2	1.7	1.8
Concrete	338	30	1.3	2.5	1.6	2.6	1.3	1.9	1.4	1.8
Concrete	338	120	2.0	3.6	2.2	4.3	2.1	3.1	2.5	3.1

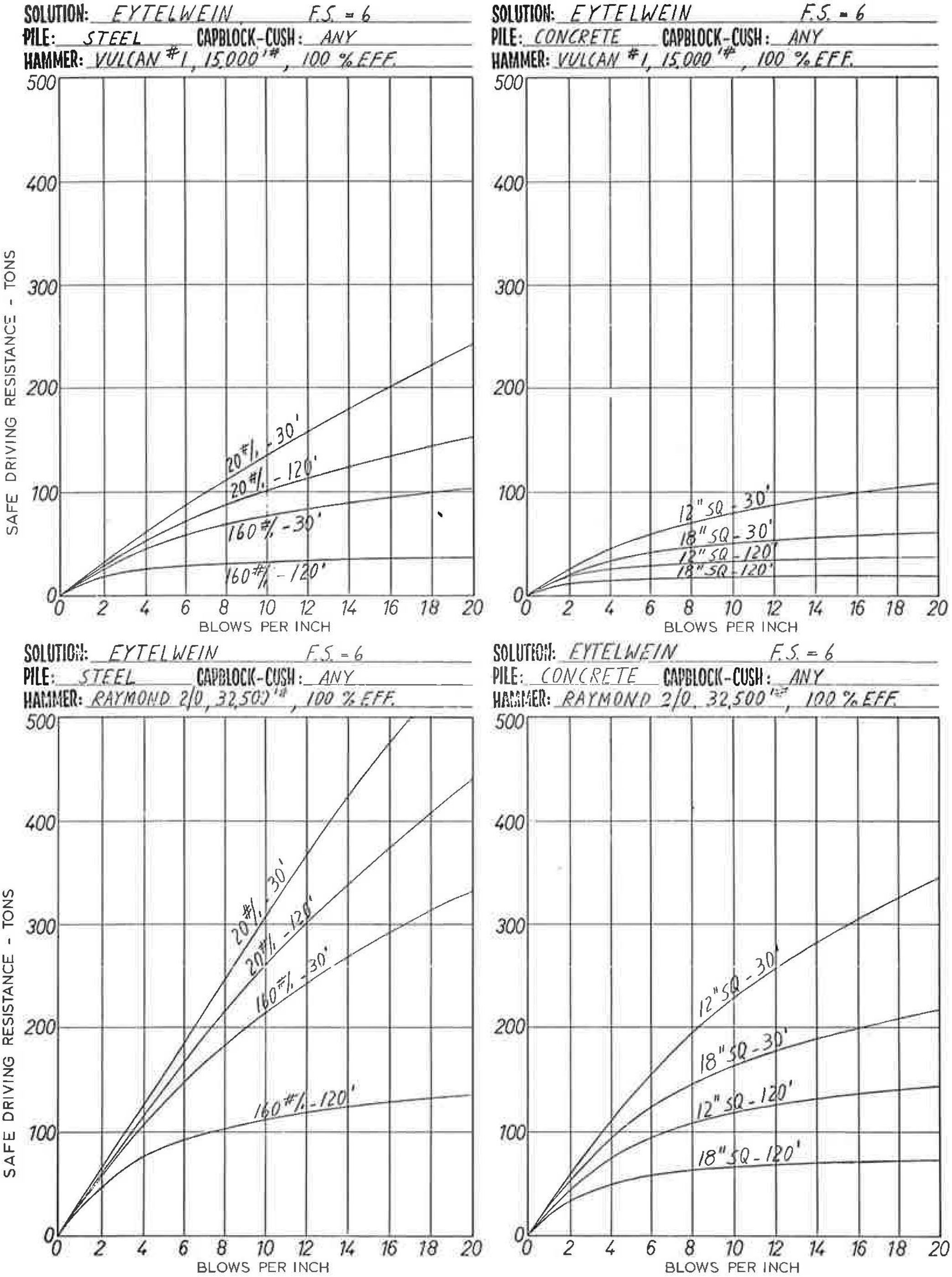

Figure 2. Solutions for the Eytelwein formula.

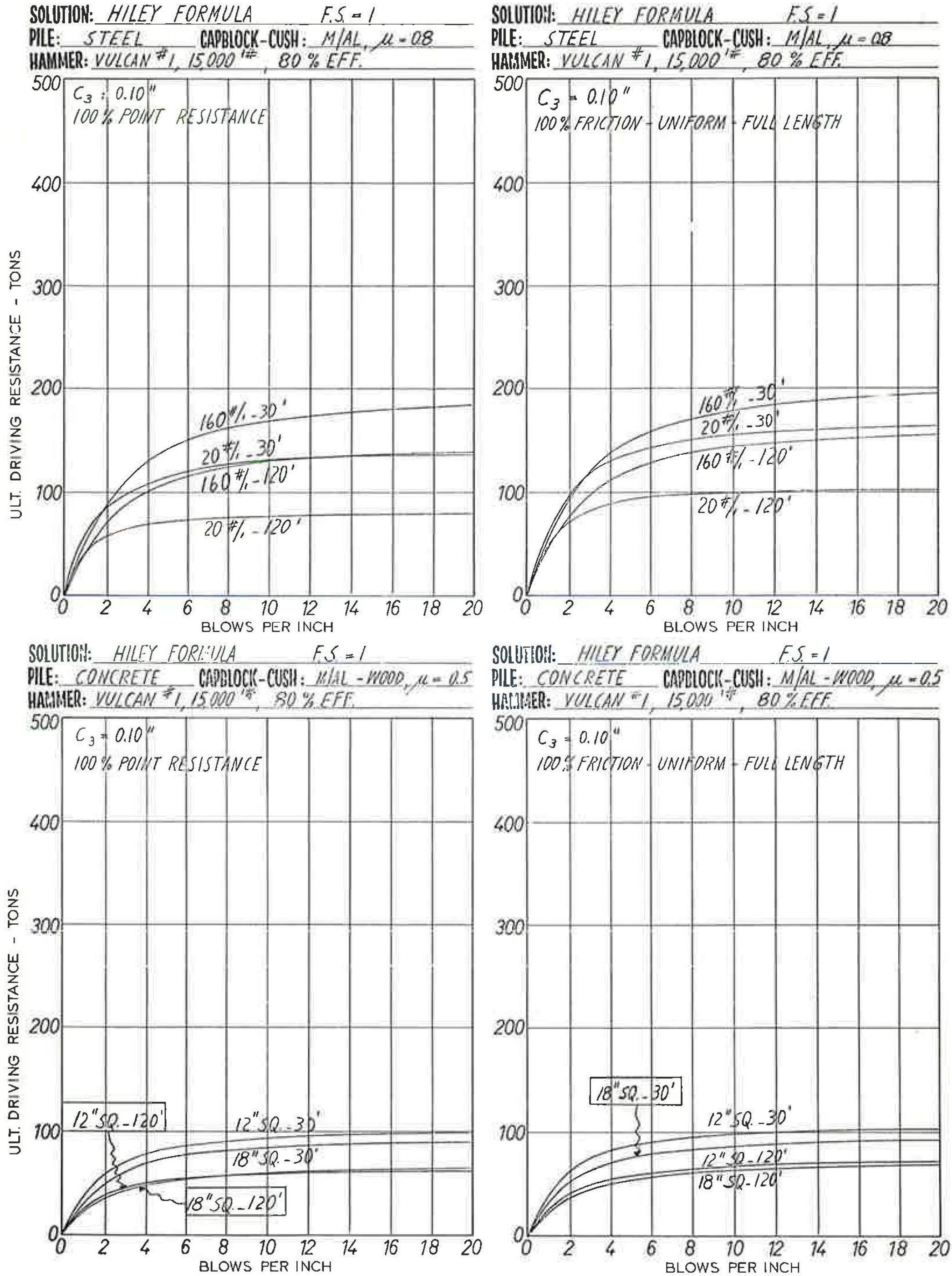

Figure 3. Solutions for the Hiley formula.

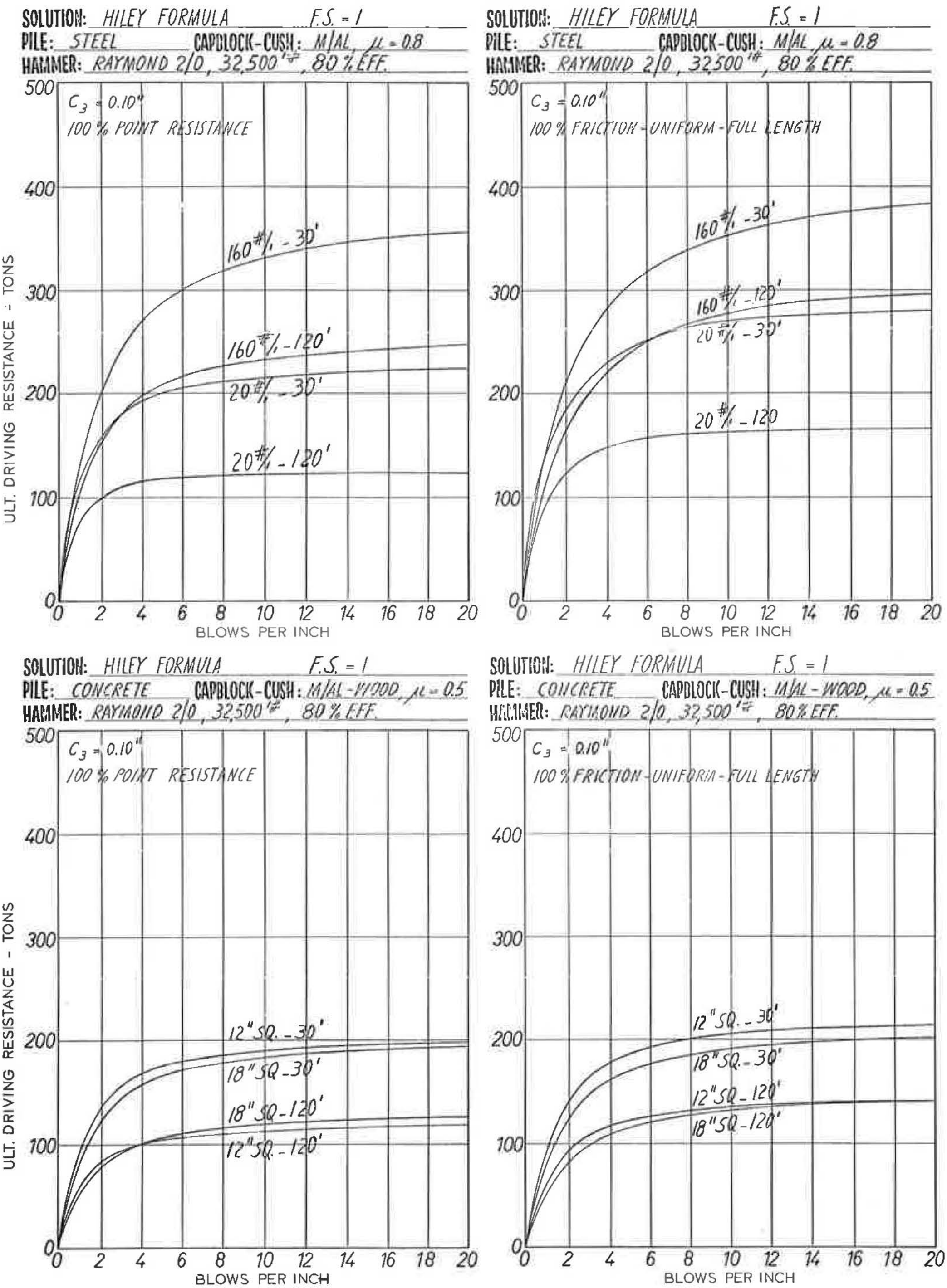

Figure 3. Continued.

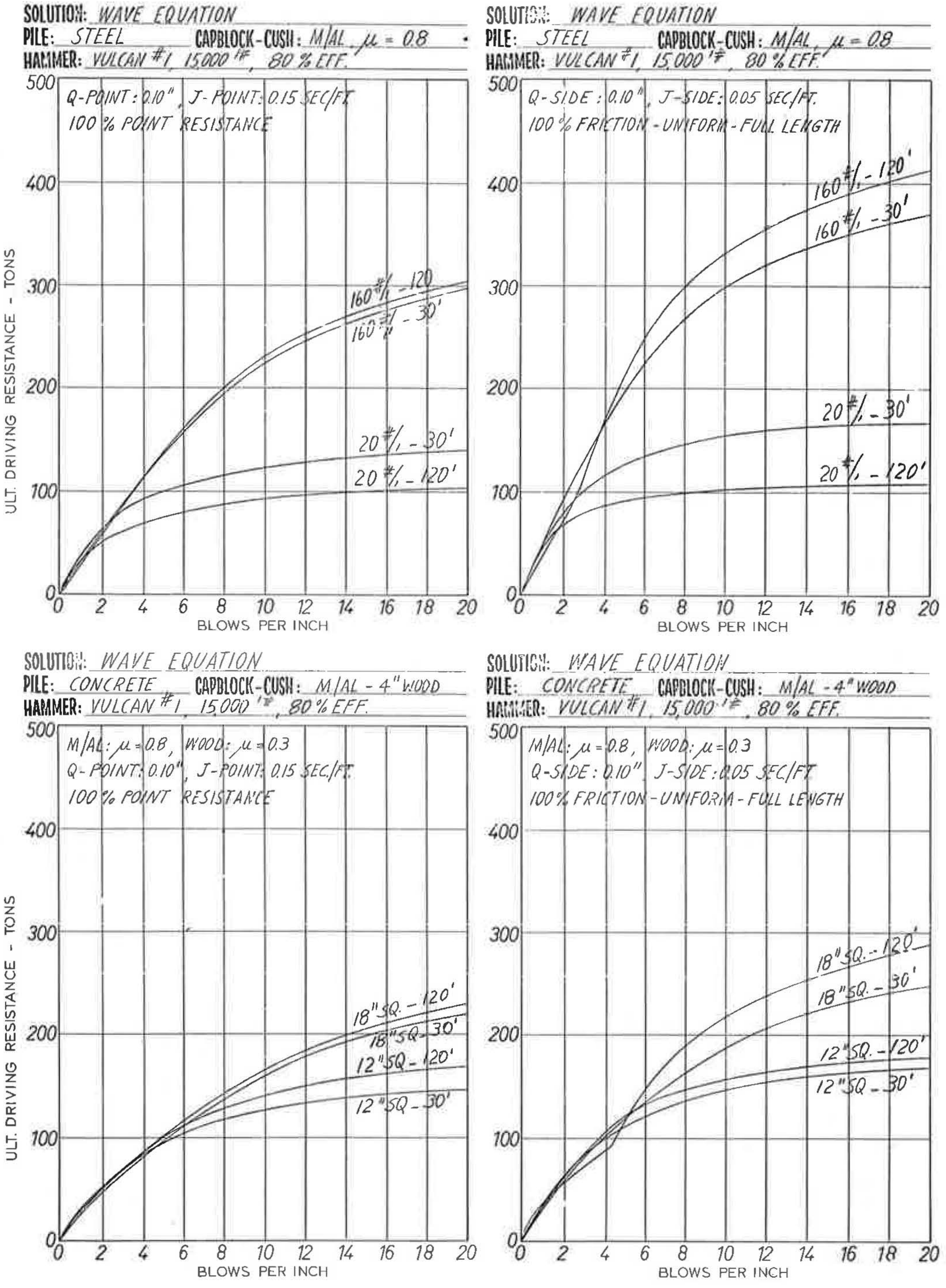

Figure 4. Solutions for the wave equation.

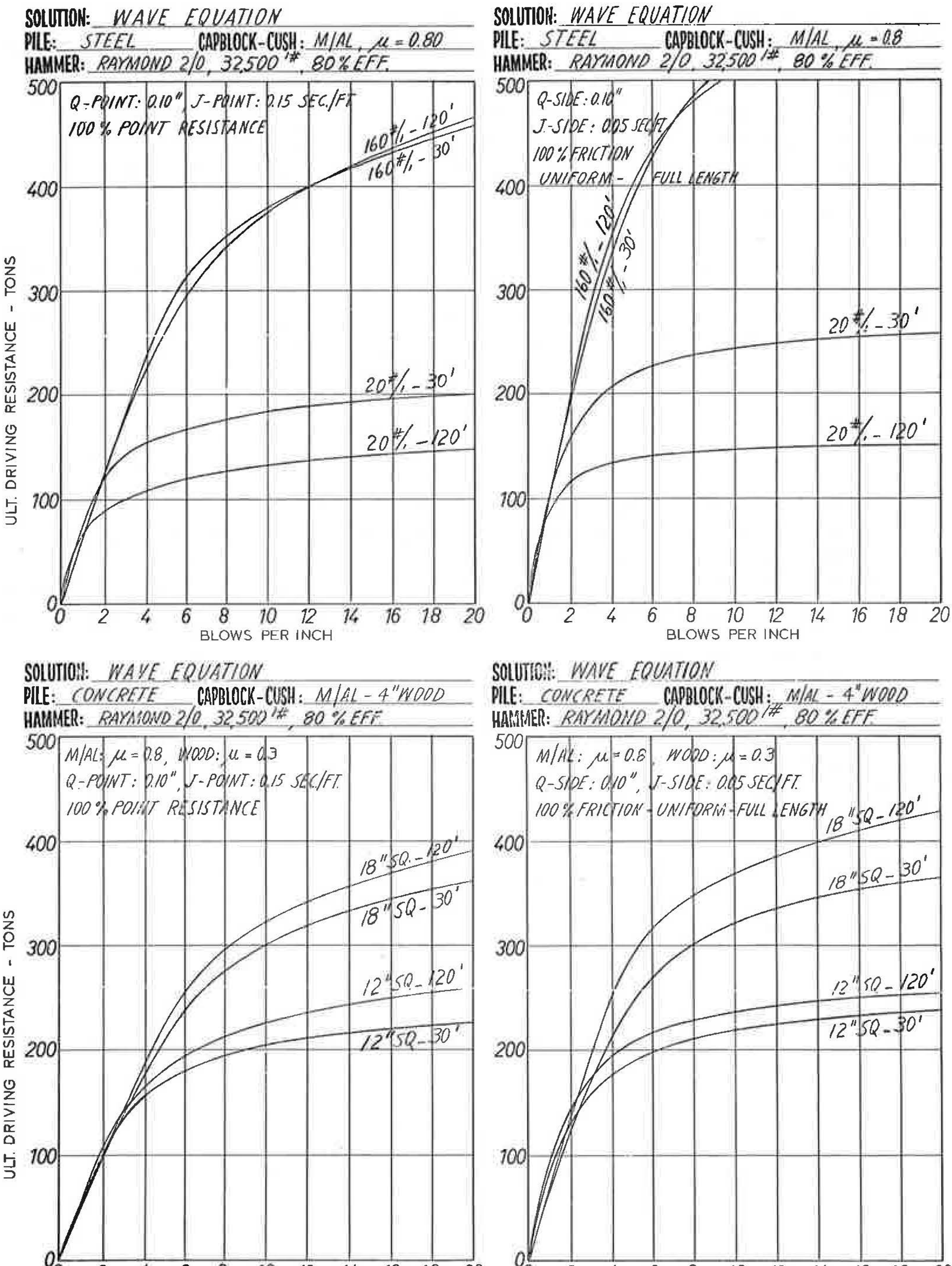

Figure 4. Continued.

The apparent factors of safety at the time of driving for the 3 dynamic formulas, based on the assumption that the wave-equation solutions are correct, are given in Table 1. In addition to the 5 parameters, Table 1 includes a sixth, that is, moderate driving resistance, considered to be 5 blows/in., and hard driving resistance, considered to be 20 blows/in. One can readily see that there are certain combinations of parameters that cause the dynamic formulas to yield a desirable factor of safety, such as two. For other combinations of parameters, these formulas can be either unsafe or else extremely conservative resulting in uneconomical solutions.

It is believed that practicing engineers will find the graphic results of these solutions a convenient reference, especially those for the wave-equation solution. Because the parameters cover the range of piling systems commonly used, an engineer may readily obtain an approximate wave-equation solution for any particular system by interpolation from these graphs. This will enable him to estimate the optimum combination of hammer, cap block, cushion, and pile for his application. If he desires a more exact solution including maximum tension and compression stresses in the pile, there are several places where he can obtain wave-equation solutions if inconvenient to set up his own program.

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Pile-Driving Analysis by One-Dimensional Wave Theory: State of the Art

T. J. HIRSCH, L. L. LOWERY, H. M. COYLE, and C. H. SAMSON, JR.,
College of Engineering, Texas A&M University

The numerical computer solution of the one-dimensional wave equation can be used with reasonable confidence for the analysis of pile-driving problems. The wave equation can be used to predict impact stresses in a pile during driving and to estimate the static soil resistance on a pile at the time of driving from driving records. If this method of analysis is used, the effects of significant parameters can be evaluated during the foundation design stage. These parameters include type and size of pile-driving hammer; driving assemblies such as cap block, helmet, and cushion block; type and size of pile; and soil condition. From such an analysis appropriate piles and driving equipment can be selected to correct or avoid expensive and time-consuming construction problems, such as excessive driving stresses or pile breakage and inadequate equipment, to achieve desired penetration or bearing capacity. Wave-equation evaluation of data from the Michigan pile study indicated that a relatively simple formula can be used to determine the energy output for both steam and diesel pile-driving hammers.

To date, the wave equation has been compared with the results of 43 actual field tests performed throughout the country, and the results are encouraging. The driving accessories significantly affect the piling behavior, and, therefore, their selection should be carefully considered and analyzed whenever possible. The effect of pile dimensions on ability to drive the pile varied greatly; generally, stiffer piles can overcome greater soil resistance to penetration. The wave equation can be used to estimate soil resistance on a pile at the time of driving. Before long-term bearing capacity can be extrapolated from this resistance, however, engineers must consider the effect of soil setup or relaxation, and other time effects that might be important.

•THE TREMENDOUS INCREASE in the use of piles in both landbased and offshore foundation structures and the appearance of new pile-driving methods have created great engineering interest in finding more reliable methods for the analysis and design of piles. Since Isaacs' paper (1), it has been recognized that the behavior of piling during driving does not follow the simple Newtonian impact as assumed by many simplified pile-driving formulas but rather is governed by the one-dimensional wave equation. Unfortunately, an exact mathematical solution to the wave equation was not possible for most practical pile-driving problems.

In 1950, Smith (2) developed a tractable solution to the wave equation that could be used to solve extremely complex pile-driving problems. The solution was based on a discrete element idealization of the actual hammer-pile-soil system coupled with the use of a high-speed digital computer. In a paper published in 1960 (3), he dealt exclusively with the application of wave theory to the investigation of the dynamic behavior of piling during driving.

Paper sponsored by Committee on Substructures, Retaining Walls and Foundations and presented at the 49th Annual Meeting.

SOLUTION TO THE WAVE EQUATION

Smith's Numerical Solution

This solution is based on dividing the distributed mass of the pile into a number of concentrated weights, W(1) through W(p), which are connected by weightless springs, K(1) through K(p - 1), and adding soil resistance that acts on the masses, as shown in Figure 1. Time is also divided into small increments. For the idealized system, Smith set up a series of equations of motion in the form of finite difference equations that were easily solved by using high-speed digital computers. He extended his original method of analysis to include various nonlinear parameters such as elastoplastic soil resistance including velocity damping and others.

Figure 1 shows the idealization of the pile system suggested by Smith. In general, the system is considered to be composed of the following:

1. A ram, to which an initial velocity is imparted by the pile driver;
2. A cap block (cushioning material);
3. A pile cap;
4. A cushion block (cushioning material);
5. A pile; and
6. The supporting medium, or soil.

In Figure 1 the ram, cap block, pile cap, cushion block, and pile are shown as appropriate discrete weights and springs. The frictional soil resistance on the side of the pile is represented by a series of side springs; the point resistance is accounted for by a single spring at the point of the pile. The characteristics of these various components will be discussed in greater detail later in this report.

Actual situations may deviate from the one shown in Figure 1. For example, a cushion block may not be used or an anvil may be placed between the ram and cap block. However, such cases are readily accommodated.

External Springs—The resistance to dynamic loading afforded by the soil in shear along the outer surface of the pile and in bearing at the point of the pile is extremely complex. Figure 2 shows the load-deformation characteristics that Smith assumed the soil to have, exclusive of damping effects. The path OABC DEFG represents loading and unloading in side friction. For the point, only compressive loading may take place and the loading and unloading path would be along OABCF.

The characteristics shown in Figure 2 are defined essentially by the quantities

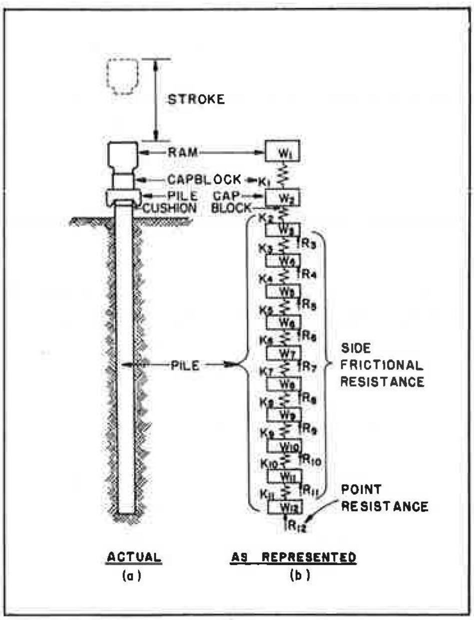

Figure 1. Method of representing pile for purpose of analysis (after Smith).

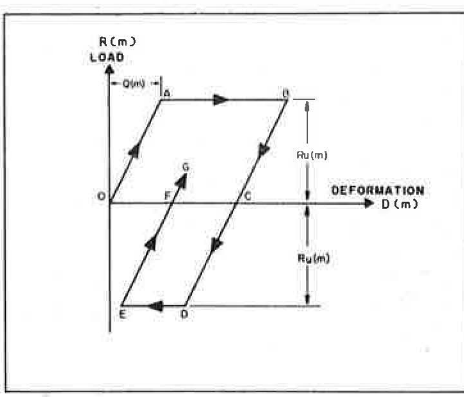

Figure 2. Load-deformation characteristics assumed for soil spring m.

Q and Ru. Q is the soil quake and represents the maximum deformation that may occur elastically. Ru is the ultimate ground resistance, or the load at which the soil spring behaves purely plastically.

A load-deformation diagram of the type shown in Figure 2 may be established separately for each spring. Thus, K′(m) equals Ru(m) divided by Q(m), where K′(m) is the spring constant (during elastic deformation) for external spring m.

Basic Equations—The following equations were developed by Smith (2):

$$D(m,t) = D(m,t-1) + 12\Delta t \, V(m,t-1) \tag{1}$$

$$C(m,t) = D(m,t) - D(m+1,t) \tag{2}$$

$$F(m,t) = C(m,t)K(m) \tag{3}$$

$$R(m,t) = [D(m,t) - D'(m,t)]\, K'(m)[1 + J(m)\, V(m,t-1)] \tag{4}$$

$$V(m,t) = V(m,t-1) + [F(m-1,t) - F(m,t) - R(m,t)]\frac{g\Delta t}{W(m)} \tag{5}$$

where

( ) = functional designation;
m = element number;
t = number of time interval;
Δt = size of time interval, sec;
C(m, t) = compression of internal spring m in time interval t, in.;
D(m, t) = displacement of element m in time interval t, in.;
D′(m, t) = plastic displacement of external soil spring m in time interval t, in.;
F(m, t) = force in internal spring m in time interval t, lb;
g = acceleration due to gravity, ft/sec^2;
J(m) = damping constant of soil at element m, sec/ft;
K(m) = spring constant associated with internal spring m, lb/in.;
K′(m) = spring constant associated with external soil spring m, lb/in.;
R(m, t) = force exerted by external spring m on element m in time interval t, lb;
V(m, t) = velocity of element m in time interval t, ft/sec; and
W(m) = weight of element m, lb.

This notation differs slightly from that used by Smith. Also, Smith restricts the soil damping constant J to 2 values, one for the point of the pile in bearing and one for the side of the pile in friction. Although the present knowledge of damping behavior of soils perhaps does not justify greater refinement, it is reasonable to use this notation as a function of m for the sake of generality.

The computations proceed as follows:

1. The initial velocity of the ram is determined from the properties of the pile driver. Other time-dependent quantities are initialized at zero or to satisfy static equilibrium conditions.
2. Displacements D(m, 1) are calculated by Eq. 1. It is to be noted that V(1, 0) is the initial velocity of the ram.
3. Compressions C(m, 1) are calculated by Eq. 2.
4. Internal spring forces F(m, 1) are calculated by Eq. 3.
5. External spring forces R(m, 1) are calculated by Eq. 4.
6. Velocities V(m, 1) are calculated by Eq. 5.
7. The cycle is repeated for successive time intervals.

Critical Time Interval

The accuracy of the discrete-element solution is also related to the size of the time increment Δt. Heising (4), in his discussion of the equation of motion for free longitudinal vibrations in a continuous elastic bar, points out that the discrete-element solution is an exact solution of the partial differential equation when

$$\Delta t = \frac{\Delta L}{\sqrt{E/\rho}}$$

where ΔL is the segment length. Smith (3) draws a similar conclusion and has expressed the critical time interval as follows:

$$\Delta t = \frac{1}{19.648}\sqrt{\frac{W(m+1)}{K(m)}} \tag{6}$$

or

$$\Delta t = \frac{1}{19.648}\sqrt{\frac{W(m)}{K_m}} \tag{7}$$

If a time increment larger than that given by Eq. 6 is used, the discrete-element solution will diverge and no valid results can be obtained. As pointed out by Smith, in this case the numerical calculation of the discrete-element stress wave does not progress as rapidly as the actual stress wave. Consequently, the value of Δt given by Eq. 6 is called the "critical" value.

Effect of Gravity

The procedure as originally presented by Smith did not account for the static weight of the pile. In other words, at $t = 0$ all springs, both internal and external, exert zero force. Stated symbolically,

$$F(m,0) = R(m,0) = 0$$

If the effect of gravity is to be included, these forces must be given initial values to produce equilibrium of the system. A relatively simple scheme has been developed as a means of getting the gravity effect into the computations (27).

PILE-DRIVING HAMMERS

Energy Output of Impact Hammer

One of the most significant parameters involved in pile driving is the energy output of the hammer. This energy output must be known or assumed before the wave equation or dynamic formula can be applied. Although most manufacturers of pile driving equipment furnish maximum energy ratings for their hammers, these are usually downgraded by foundation experts for various reasons. A number of conditions such as poor hammer condition, lack of lubrication, and wear are known to seriously reduce energy output of a hammer. In addition, the energy output of many hammers can be controlled by regulating the steam pressure or quantity of diesel fuel supplied to the hammer. Therefore, a method was needed to determine a simple and uniform method that would accurately predict the energy output of a variety of hammers in general use.

Determination of Hammer Energy Output

Diesel Hammers—At present the manufacturers of diesel hammers arrive at the energy delivered per blow by 2 different methods. One manufacturer (5) feels that "Since the amount of (diesel) fuel injected per blow is constant, the compression pressure is constant, and the temperature constant, the energy delivered to the piling is also constant." The energy output per blow is thus computed as the kinetic energy of the falling ram plus the explosive energy found by thermodynamics. Other manufacturers simply give the energy output per blow as the product of the weight of the ram-piston W_R and the length of the stroke h, or the equivalent stroke in the case of closed-end hammers.

The energy ratings given by these 2 methods differ considerably because the ram stroke h varies greatly. There is much controversy, therefore, as to which, if either, method is correct and what energy output should be used in dynamic pile analysis.

In conventional single-acting steam hammers, the steam pressure or energy is used to raise the ram for each blow. The magnitude of the steam force is too small to force the pile downward, and consequently it works only on the ram to restore its potential energy, $W_R \times h$, for the next blow. In a diesel hammer, on the other hand, the diesel explosive pressure used to raise the ram is, for a short time at least, relatively large (Fig. 3).

Although this explosive force works on the ram to restore its potential energy, $W_R \times h$, the initially large explosive pressure also does some useful work on the pile. Because the total energy output is the sum of the kinetic energy at impact plus the work done by the explosive force,

$$E_{total} = E_k + E_e \tag{8}$$

where

E_{total} = total energy output per blow;
E_k = kinetic energy of the ram at the instant of impact; and
E_e = the diesel explosive energy that does useful work on the pile.

It has been noted that, after the ram passes the exhaust ports, the energy required to comprss the air-fuel mixture is nearly identical to that gained by the remaining fall, d, of the ram (5). Therefore, the velocity of the ram at the exhaust ports is essentially the same as at impact, and the kinetic energy at impact can be closely approximated by

$$E_k = W_R(h - d) \tag{9}$$

where

W_R = ram weight;
h = total observed stroke of the ram; and
d = distance the ram moves after closing the exhaust ports and impacts with the anvil.

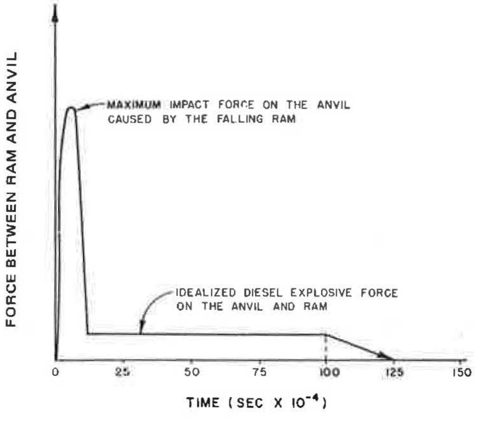

Figure 3. Typical force versus time curve for a diesel hammer.

The total amount of explosive energy $E_{e(total)}$ is dependent on the amount of diesel fuel injected, compression pressure, and temperature and, therefore, may vary somewhat.

Unfortunately, the wave equation must be used in each case to determine the exact magnitude of E_e because it depends not only on the hammer characteristics but also on the characteristics of the anvil, helmet, cushion, pile, and soil resistance. However, values of E_e determined by the wave equation for several typical pile problems indicate that it is usually small in proportion to the total explosive energy output per blow and, furthermore, that it is on the same order of magnitude as $W_R \times d$. Thus, assuming that

$$E_e = W_R \times d \tag{10}$$

and substituting Eqs. 9 and 10 into Eq. 8 give

$$E_{total} = E_k + E_e = W_R(h - d) + W_R d \tag{11}$$

so that

$$E_{total} = W_R h \tag{12}$$

The results given by this equation were compared with experimental values and the average efficiency was found to be 100 percent.

Steam Hammers—Using the same equation for comparison with experimental values indicated an efficiency rating of 60 percent for the single-acting steam hammers and 87 percent for the double-acting hammer, based on an energy output given by

$$E_{total} = W_R h \tag{13}$$

In order to determine an equivalent ram stroke for the double-acting hammers, the internal steam pressure above the ram that is forcing it down must be taken into consideration. The manufacturers of such hammers state that the maximum steam pressure or force should not exceed the weight of the housing or casing, or the housing may be lifted off the pile. Thus the maximum downward force on the ram is limited to the total weight of the ram and housing.

Because these forces both act on the ram as it falls through the actual ram stroke h, they add kinetic energy to the ram, which is given by

$$E_{total} = W_R h + F_R h \tag{14}$$

where

W_R = ram weight;
F_R = steam force not exceeding the weight of the hammer housing; and
h = observed or actual ram stroke.

Because the actual steam pressure is not always applied at the rated maximum, the actual steam force can be expressed as

$$F_R = \left(\frac{p}{P_{rated}}\right) W_H \tag{15}$$

where

W_H = hammer housing weight;
p = operating pressure; and
P_{rated} = maximum rated steam pressure.

The total energy output is then given by

$$E_{total} = W_R h + \left(\frac{p}{P_{rated}}\right) W_H h \tag{16}$$

This can be reduced in terms of Eq. 13 by using an equivalent stroke h_e that will give the same energy output as Eq. 16.

Thus,

$$E_{total} = W_R h_e \tag{17}$$

Setting Eqs. 16 and 17 equal yields

$$W_R h_e = W_R h + \left(\frac{p}{P_{rated}} W_H\right) h = h\left(W_R + \frac{p}{P_{rated}} W_H\right)$$

or solving for the equivalent stroke yields

$$h_e = h\left(1 + \frac{p}{P_{rated}} \times \frac{W_H}{W_R}\right) \tag{18}$$

Conclusions—The preceding discussion has shown that it is possible to determine reasonable values of hammer-energy output simply by taking the product of the ram weight and its observed or equivalent stroke and applying an efficiency factor. This method of energy rating can be applied to all types of impact pile drivers with reasonable accuracy.

Significance of Driving Accessories

In 1965 the Michigan Department of State Highways completed an extensive research program designed to obtain a better understanding of the complex problem of pile driving. Though a number of specific objectives were given, one was of primary importance. As noted by Housel (6), "Hammer energy actually delivered to the pile, as compared with the manufacturer's rated energy, was the focal point of a major portion of this investigation of pile-driving hammers." In other words, the researchers hoped to determine the energy delivered to the pile and to compare these values with the manufacturer's ratings.

The energy transmitted to the pile was termed ENTHRU by the investigators and was determined by the summation

$$\text{ENTHRU} = \Sigma F \Delta S$$

where F, the average force on the top of the pile during a short interval of time, was measured by a specially designed load cell, and ΔS, the incremental movement of the head of the pile during this time interval, was found by using displacement transducers or was reduced from accelerometer data or both. It should be pointed out that ENTHRU is not the total energy output of the hammer blow, but only a measure of that portion of the energy delivered below the load-cell assembly.

Many variables influence the value of ENTHRU. As was noted in the Michigan report: "Hammer type and operating conditions; pile type, mass, rigidity, and length; and the type and condition of cap blocks were all factors that affect ENTHRU, but when, how, and how much could not be ascertained with any degree of certainty." However, the wave equation can account for each of these factors so that their effects can be determined.

The maximum displacement of the head of the pile was also reported and was designated LIMSET. Oscillographic records of force versus time measured in the load cell were also reported. Because force was measured only at the load cell, the single maximum observed values for each case was called FMAX.

ENTHRU is greatly influenced by several parameters, especially the type, condition, and coefficient of restitution of the cushion, and the weight of extra driving caps.

The wave equation was therefore used to analyze certain Michigan problems to determine the influence of cushion stiffness, e, additional driving cap weights, and driving resistance encountered.

Table 1 gives data that show how ENTHRU and SET increase when the load cell assembly is removed from Michigan piles.

The data given in Table 2 show that ENTHRU does not always increase with increasing cushion stiffness. Furthermore, the maximum increase in ENTHRU noted here is relatively small—only about 10 percent.

TABLE 1

EFFECT OF REMOVING LOAD CELL ON ENTHRU, LIMSET, AND PERMANENT SET OF PILE

Case	Ram Velocity (ft/sec)	ENTHRU (kip/ft)		LIMSET (in.)		Permanent Set (in.)	
		With Load Cell	Without Load Cell	With Load Cell	Without Load Cell	With Load Cell	Without Load Cell
DTP-15, 80.5	8	1.5	1.6	0.27	0.34	0.23	0.25
	12	3.3	3.6	0.53	0.67	0.57	0.57
	16	5.8	6.5	1.02	1.03	0.94	0.97
	20	9.1	10.1	1.54	1.54	1.43	1.47
DLTP-8, 80.2	8	3.1	3.8	0.62	0.71	0.51	0.62
	12	7.1	8.5	1.15	1.32	1.06	1.29
	16	12.5	15.1	1.91	2.10	1.82	2.15
	20	19.5	23.6	2.70	3.08	2.65	3.13

TABLE 2

EFFECT OF CUSHION STIFFNESS ON ENERGY TRANSMITTED TO THE PILE (ENTHRU)

Ram Velocity (ft/sec)	RUT (kip)	ENTHRU (kip/ft) by Cushion Stiffness			
		540 kip/in.	1,080 kip/in.	2,700 kip/in.	27,000 kip/in.
8	30	3.0	3.0	3.0	2.9
	90	3.1	3.2	3.3	2.9
	150	3.0	3.2	3.3	3.0
12	30	6.6	6.4	7.1	6.4
	90	7.0	7.1	7.2	6.4
	150	6.9	7.2	7.4	6.7
16	30	11.8	11.9	12.2	11.3
	90	12.3	12.6	12.8	11.5
	150	12.4	12.9	13.2	11.4

When different cushions are used, the coefficient of restitution will probably change. Because the coefficient of restitution of the cushion may affect ENTHRU, a number of cases was solved with e ranging from 0.2 to 0.6. The data given in Tables 3 and 4 show that an increase in e from 0.2 to 0.6 normally increases ENTHRU from 18 to 20 percent, while increasing the permanent set from 6 to 11 percent. Thus, for the case shown, the coefficient of restitution of the cushion has a greater influence on rate of penetration and ENTHRU than does its stiffness. This same effect was noted in the other solutions, and results of the cases for which data are given in Tables 3 and 5 are typical of the results found in other cases.

The data given in Table 5 show that any increase in cushion stiffness also increases the driving stress. Thus, according to the wave equation, increasing the cushion stiffness to increase the rate of penetration (for example by not replacing the cushion until it has been beaten to a fraction of its original height or by omitting the cushion entirely) is both inefficient and poor practice because of the high stresses induced in the pile. It would be better to use a cushion having a high coefficient of restitution and a low cushion stiffness in order to increase ENTHRU and to limit the driving stress.

TABLE 3

EFFECT OF COEFFICIENT OF RESTITUTION ON MAXIMUM POINT DISPLACEMENT

Pile	RUT (kip)	Ram Velocity (ft/sec)	Maximum Point Displacement (in.)			Maximum Change (percent)
			e = 0.2	e = 0.4	e = 0.6	
BLTP-6, 10.0	30	12	2.13	2.14	2.36	10
		16	3.38	3.47	3.58	6
		20	4.73	4.93	5.17	8
BLTP-6, 57.9	150	12	0.46	0.48	0.50	8
		16	0.73	0.76	0.81	10
		20	1.05	1.10	1.18	11

TABLE 4

EFFECT OF COEFFICIENT OF RESTITUTION ON ENTHRU

Pile	RUT (kip)	Ram Velocity (ft/sec)	ENTHRU (kip/ft)			Maximum Change (percent)
			e = 0.2	e = 0.4	e = 0.6	
BLTP-6, 10.0	30	12	6.0	6.5	7.3	18
		16	10.5	11.8	12.8	18
		20	16.5	17.4	20.0	17
BLTP-6, 57.9	150	12	6.7	7.2	8.2	18
		16	11.6	12.7	14.5	20
		20	18.2	19.7	22.4	19

Unfortunately, the tremendous variety of driving accessories precludes general conclusions to be drawn from wave equation analyses in all but the most general of terms.

Although the effect of driving accessories is quite variable, it was generally noted that the inclusion of additional elements between the driving hammer and the pile or the inclusion of heavier driving accessories or both consistently decreased both the energy transmitted to the head of the pile and the permanent set per blow of the hammer. Increasing cushion stiffness will increase compressive and tensile stresses induced in a pile during driving. Table 6 gives data on the effect of cushion stiffness on the maximum displacement of the head of the pile.

CAP BLOCK AND CUSHIONS

Methods Used to Determine Cap Block and Cushion Properties

As used here, cap block refers to the material placed between the pile-driving hammer and the steel helmet, and cushion refers to the material placed between the steel helmet and pile (usually used only when concrete piles are driven). Although a cap block and cushion serve several purposes, their primary function is to limit impact stresses in both the pile and hammer. In general, it has been found that a wooden cap block is quite effective in reducing driving stresses, more so than a relatively stiff cap block material such as Micarta. However, the stiffer Micarta is usually more durable and transmits a greater percentage of the hammer's energy to the pile because of its higher coefficient of restitution.

For example, when 14 different cases in the Michigan study were solved by the wave equation, the Micarta assemblies averaged 14 percent more efficient than cap block assemblies of wood. However, the increased cushion stiffness in some of these cases increased the impact stresses. The increase in stress was particularly important when concrete or prestressed concrete piles were driven. When concrete piles are

TABLE 5

EFFECT OF CUSHION STIFFNESS ON MAXIMUM FORCE MEASURED AT THE LOAD CELL (FMAX)

Ram Velocity (ft/sec)	RUT (kip)	FMAX (kip) by Cushion Stiffness			
		540 kip/in.	1,080 kip/in.	2,700 kip/in.	27,000 kip/in.
8	30	132	185	261	779
	90	137	185	261	779
	150	143	186	261	779
12	30	198	278	391	1,169
	90	205	278	391	1,169
	150	215	279	391	1,169
16	30	264	371	522	1,558
	90	275	371	522	1,558
	150	288	371	522	1,558

TABLE 6

EFFECT OF CUSHION STIFFNESS ON MAXIMUM DISPLACEMENT OF THE HEAD OF THE PILE (LIMSET)

Ram Velocity (ft/sec)	RUT (kip)	LIMSET (in.) by Cushion Stiffness			
		540 kip/in.	1,080 kip/in.	2,700 kip/in.	27,000 kip/in.
8	30	1.09	1.08	1.08	1.13
	90	0.44	0.44	0.45	0.45
	150	0.32	0.33	0.33	0.33
	30	2.21	2.14	2.19	2.25
12	90	0.80	0.82	0.84	0.84
	150	0.55	0.57	0.58	0.58
	30	3.62	3.59	3.63	3.68
16	90	1.30	1.31	1.32	1.34
	150	0.85	0.87	0.88	0.90

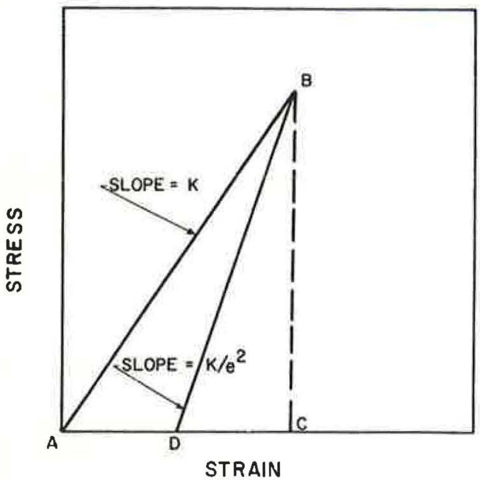

Figure 4. Stress-strain curve for a cushion block.

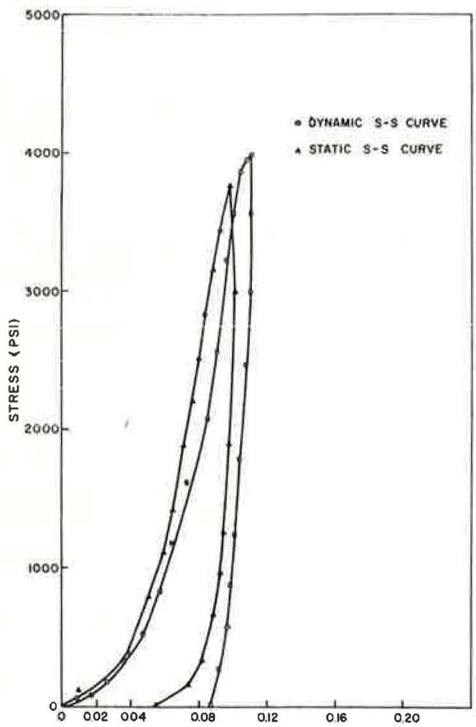

Figure 5. Dynamic and static stress-strain curves for a fir cushion.

TABLE 7

TYPICAL SECANT MODULI OF ELASTICITY E AND COEFFICIENTS OF RESTITUTION e OF VARIOUS PILE-CUSHIONING MATERIAL

Material	E (psi)	e
Micarta plastic	450,000	0.80
Oak (green)	45,000[a]	0.50
Asbestos disks	45,000	0.50
Fir plywood	35,000[a]	0.40
Pine plywood	25,000[a]	0.30
Gum	30,000[a]	0.25

[a]Properties of wood with load applied perpendicular to wood grain.

driven, it is also frequently necessary to include cushioning material between the helmet and the head of the pile to distribute the impact load uniformly over the surface of the pile head and prevent spalling.

To apply the wave equation to pile driving, Smith assumed that the cushion's stress-strain curve was a series of straight lines as shown in Figure 4.

Static and dynamic stress-strain properties were measured for several types of cushions. It was determined that the stress-strain curves were not linear as was assumed by Smith, but rather appeared as shown in Figure 5.

Surprisingly, the static and dynamic stress-strain curves for wooden cushions agreed remarkably well. A typical example of this agreement is shown in Figure 5. The stress-strain curves for a number of other materials commonly used as pile cushions and cap blocks, namely oak, Micarta, and asbestos, were also measured.

Idealized Load-Deformation Properties

The major difficulty encountered in trying to use the dynamic curves determined for the various cushion materials was that it was extremely difficult to input the information required by the wave equation. Although the initial portion of the curve was nearly parabolic, the top segment and unloading portion were extremely complex. This prevented the curve from being input in equation form and required numerous points on the curve to be specified.

Fortunately, it was found that the wave equation accurately predicted both the shape and the magnitude of the stress wave induced in the pile, even if a linear force-deformation curve was assumed for the cushion, so long as the loading portion was based on the secant modulus of elasticity for the material (as opposed to the initial, final, or average modulus of elasticity) and the unloading portion was based on the actual dynamic coefficient of restitution. Typical secant moduli of elasticity values for various materials are given in Table 7.

Coefficient of Restitution

Although the cushion is needed to limit the driving stresses in both hammer and pile, its internal damping reduces the available driving energy transmitted to the head of the pile. Figure 4 shows this energy loss, with the input energy being given by the area ABC while the energy output is given by area BCD. This energy loss is commonly termed coefficient of restitution of the cushion e, in which

$$e = \sqrt{\frac{\text{area BCD}}{\text{area ABD}}}$$

Once the coefficient of restitution for the material is known, the slope of the unloading curve can be determined as shown in Figure 4.

For practical pile-driving problems, secant moduli of elasticity and coefficient of restitution values for well-consolidated cushions should be used. Table 7 also gives the coefficient of resitution for the materials that are recommended when the problem is analyzed by the wave equation.

SOIL PROPERTIES

A limited amount of work has been done on soil properties and their effects on the wave equation solution of the piling behavior problem (7, 8, 9). A brief summary of the results of these tests is given in this section.

Equations to Describe Soil Behavior

Examination of Eq. 19 show that Smith's equation describes a type of Kelvin rheological model as shown in Figure 6.

$$R(m,t) = [D(m,t) - D'(m,t)]\,K'(m)\,[1 + J(m)\,V(m,t-1)] \quad (19)$$

The soil spring behaves elastically until the deformation D(m,t) equals Q, and then it yields plastically with a load-deformation property as shown in Figure 7a. The dashpot J develops a resisting force proportional to the velocity of loading V. Smith has

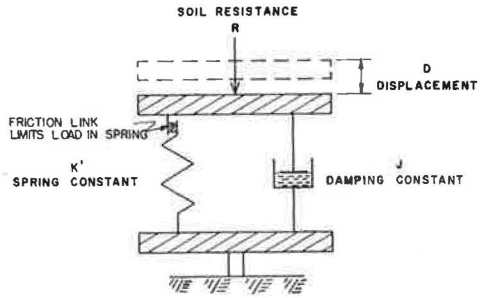

Figure 6. Model used by Smith to describe soil resistance on pile.

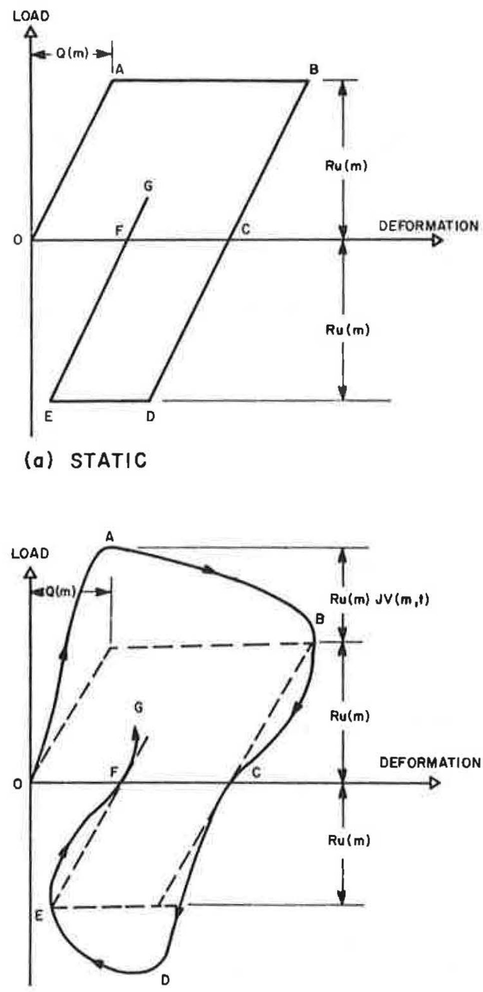

Figure 7. Load-deformation characteristics of soil.

modified the true Kelvin model slightly as shown by Eq. 20. This equation will produce a dynamic load-deformation behavior shown by path OABCDEF in Figure 7b. If terms in Eq. 19 are examined, it can be seen that Smith's dashpot force is given by

$$[D(m,t) - D'(m,t)]\, K'(m)\, [J(m)\, V(m,t-1)] \tag{20}$$

The dimensions of J are sec/ft, and it is assumed to be independent of the total soil resistance or size of the pile. It is also assumed to be constant for a given soil under given conditions as is the static shear strength of the soil from which Ru on a pile segment is determined. Ru is defined as the maximum soil resistance on a pile segment.

Care must be used to satisfy conditions at the point of the pile. Consider Eq. 10 when $m = p$, where p is the number of the last element of the pile. K(p) is used as the point soil spring and J(p) as the point soil damping constant. Also at the point of the pile, the soil spring must be prevented from exerting tension on the pile point. The point soil resistance will follow the path OABCFG in Figure 7b. It should be kept in mind that at the pile point the soil is loaded in compression or bearing. The damping constant J(p) in bearing is believed to be larger than the damping constant J(m) in friction along the side of the pile.

Soil Parameters to Describe Dynamic Soil Resistance During Pile Driving

The soil parameters used to describe the soil resistance in the wave equation are Ru, Q, and J.

Soil resistance Ru—For the side or friction soil resistance, Ru is determined by the maximum static soil adhesion or friction against the side of a given pile segment by

$$Ru(m) = fs\, \Sigma_0\, \Delta L \tag{21}$$

where

fs = maximum soil adhesion or friction, lb/ft^2;
Σ_0 = perimeter of pile segment, ft; and
ΔL = length of pile segment, ft.

In cohesionless materials (sands and gravels)

$$fs = \bar{\sigma} \tan \phi' \tag{22}$$

where

$\bar{\sigma}$ = effective normal stress against the side of the pile, lb/ft^2; and
ϕ' = angle of friction between soil and pile, deg.

In cohesive soils (clays), fs during driving is the remolded adhesion strength between the soil and pile.

At the point of the pile, Ru is determined by the maximum static bearing strength of the soil and is found by

$$Ru = (Qu)(Ap) \tag{23}$$

where

Qu = ultimate bearing strength of soil, lb/ft^2; and
Ap = area of pile point, ft^2.

In cohesive soils (clays), it is believed that the undisturbed strength of the soil may be used conservatively to determine Qu, because the material at the pile point is in the process of being compacted and may even have a higher bearing value.

Quake Q—The value of Q, the elastic deformation of the soil, is difficult to determine for various types of soil conditions. Various sources of data indicate that values of Q in both friction and point bearing probably range from 0.05 to 0.15 in.

Chellis (10) indicates that the most typical value for average pile-driving conditions is Q = 0.10 in. If the soil strata immediately underlying the pile tip is very soft, it is possible for Q to go as high as 0.2 in. or more. At the present state of the art of pile-driving technology, it is recommended that a value of Q = 0.10 in. be used for computer simulation of friction and point soil resistance. However, in particular situations where more precise values of Q are known, they should be used.

Damping constant J—The Texas Transportation Institute has conducted static and dynamic tests on cohesionless soil samples to determine if Smith's rheological model adequately describes the load-deformation properties of these soils. Triaxial soil tests were conducted on Ottawa sand at different loading velocities. Figure 8 shows typical results from a series of such tests.

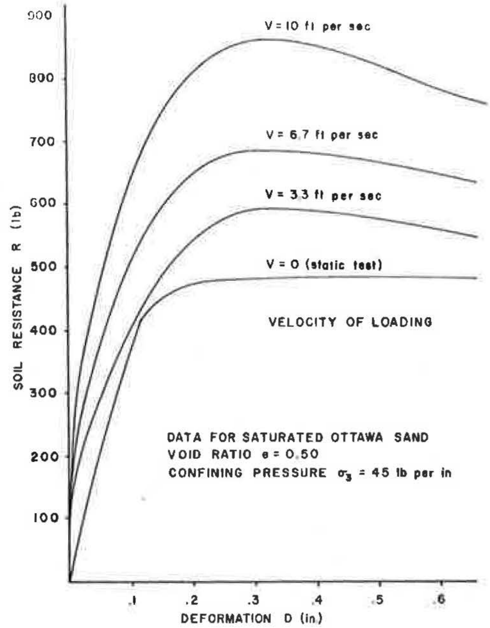

Figure 8. Load-deformation properties of Ottawa sand determined by triaxial tests (specimens 3 in. wide by 6.5 in. high).

Figure 9 shows additional data concerning the increase in soil strength as the rate of loading is increased. Because these tests were confined compression tests, it is believed that they simulate to some extent the soil behavior at the pile point. The J-value increases as the sand density increases (void ratio e decreases), and it increases as the effective confining stress $\bar{\sigma}_3$ increases.

$$\bar{\sigma}_3 = \sigma_3 - u$$

where

σ_3 = total confining pressure; and
u = pore water pressure.

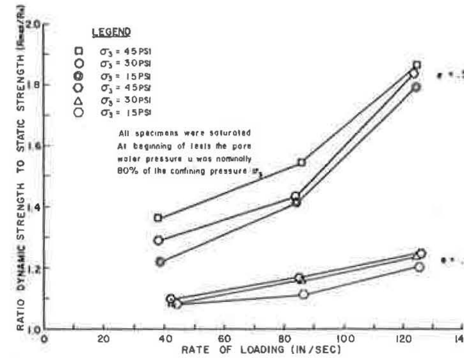

Figure 9. Increase in strength versus rate of loading for Ottawa sand.

For saturated Ottawa sand specimens, J(p) varied from about 0.01 to 0.12. When the sand was dry J(p) was nominally equal to zero. These values of J(p) for sand are in reasonable agreement with those recommended by Smith (11) and Forehand and Reese (12)—0.1 to 0.4.

The value of J(p) for cohesive soils (clays) is not presently known. The very limited data available indicate it is at least equal to that for sand. Forehand and Reese believe it ranges from 0.5 to 1.0.

There are no data now available to indicate the value of J(m) in friction along the side of the pile. Smith believes it is smaller than J(p) and recommends J(m) values in friction of about ⅓ those at the point. Research is under way at Texas A&M University that should indicate the value of J in friction. At the present time J(m) in friction or adhesion is assumed to be ⅓ of J(p).

Static Soil Resistance After Pile Driving (Time Effect)

Immediately after the pile is driven, the total static soil resistance or bearing capacity of the pile equals the sum of the Ru values discussed previously. Thus, Ru(total) is the bearing capacity immediately after driving.

$$Ru(total) = \sum_{m=1}^{m=p} Ru(m)$$

where

Ru(m) = soil adhesion or friction on segments m = 1 to m = p - 1, lb (note that this is the strength of the disturbed or remolded soil along the side of the pile); and

Ru(p) = bearing or compressive strength of soil at the pile point m = p, lb (note that this is taken as the strength of the soil in an undisturbed condition, which should be conservative).

As time elapses after the pile is driven, Ru(m) for m = 1 to p - 1 may increase as the disturbed or remolded soil along the side of the pile reconsolidates and the excess pore water pressure dissipates back to an equilibrium condition. In cohesive soils (clays) the increase in strength upon reconsolidation (sometimes referred to as setup) is often considerable.

The bearing capacity of the pile will increase as the remolded or disturbed clay along the side of the pile reconsolidates and gains strength, because the adhesion or friction strength of clay is generally restored with the passage of time. Loading tests at increasing intervals of time show that ultimate adhesion is approximately equal to the undisturbed cohesion. Therefore, the amount of increase in bearing capacity with time is related to the sensitivity and reconsolidation of the clay; sensitivity of clay = (undisturbed strength/remolded strength).

Figure 10 shows the time effect or setup of a pile driven in a cohesive soil. In cohesionless soils (sands and gravels) the friction strength of the soil will usually change very little. Normally, the value of Ru(p) at the pile point changes very little.

USE OF THE WAVE EQUATION TO PREDICT PILE LOAD-BEARING CAPACITY AT TIME OF DRIVING

In general, engineers are interested in the static load-carrying capacity of the driven pile. In the past the engineer has often had to rely on judgment based on simplified dynamic pile equations such as the Hiley or Engineering-News formulas. By the wave-equation method of analysis, a much more realistic engineering estimate can be made by using information generated by the program.

Previous sections have shown how the hammer-pile-soil system can be simulated and analyzed by the wave equation to determine the dynamic behavior of piling during driving. With this simulation, the driving stresses and penetration of the pile can be computed.

Wave Equation Method

In the field the pile penetration or permanent set per blow (in./blow) is observed, and this can be translated into the static soil resistance through the use of the wave equation.

Consider the example for soil that is a soft marine deposit of fine sand, silt, and muck, with the pile point founded on a dense layer of sand and gravel.

Steel step taper pile, ft	75
No. 00 Raymond hammer	
Efficiency, percent	80
Ram weight, lb	10,000
Energy, ft-lb	32,500
Micarta cap block	
K, lb/in.	6,600,000
e	0.8

Soil parameters assumed

J(p) point, sec/ft	0.15
J(m) side, sec/ft	0.05
Q(p) point, in.	0.10
Q(m) side, in.	0.10

Soil distribution assumed

Curve I	
Side friction (triangular distribution), percent	25
Point bearing, percent	75
Curve II	
Side friction (triangular distribution), percent	10
Point bearing, percent	90

Data shown in Figure 11 were developed by using J(point) = 0.3 for clay and J(point) = 0.1 for sand. The accuracy of the correlation, as shown in Figure 11, was approximately ±25 percent. Moseley (15) has found a similar correlation with 12 piles driven in sand. Figure 12 shows a summary of the data for the piles tested and shows that all resistances on these piles fall within ±20 percent of that predicted by the wave equation.

This information is used to simulate the system to be analyzed by the wave equation. A total soil resistance Ru(total) is assumed by the computer for analysis in the work. It then computes the pile penetration or permanent set when driven against this Ru(total). The reciprocal of permanent set is usually computed to convert this to blows per inch.

The computer program then selects a larger Ru(total) and computes the corresponding blows per inch. This is done several times until enough points are generated to develop a

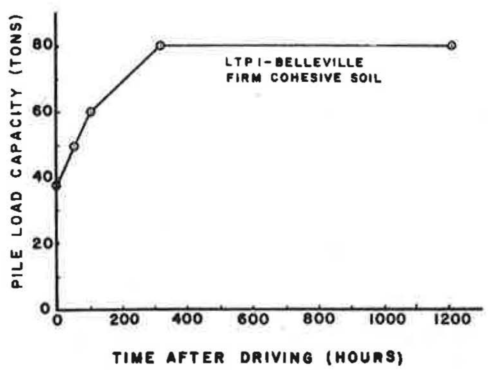

Figure 10. Setup or recovery of strength after driving in cohesive soil (13).

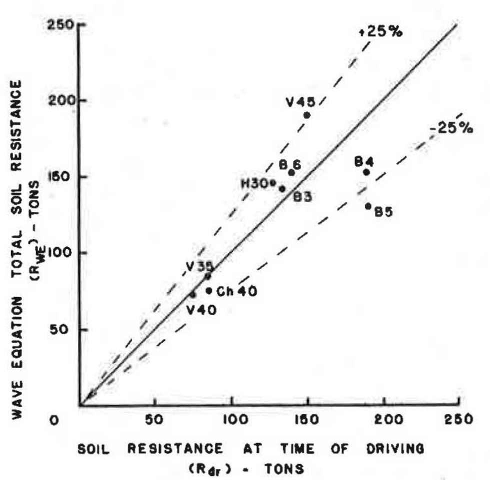

Figure 11. Comparison of wave-equation predicted soil resistance to soil resistance determined by load tests for piles driven in both sand and clay (14).

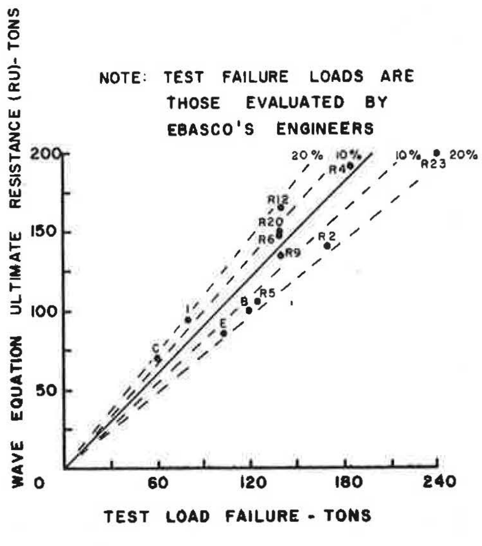

Figure 12. Wave-equation ultimate resistance versus test load failure (15).

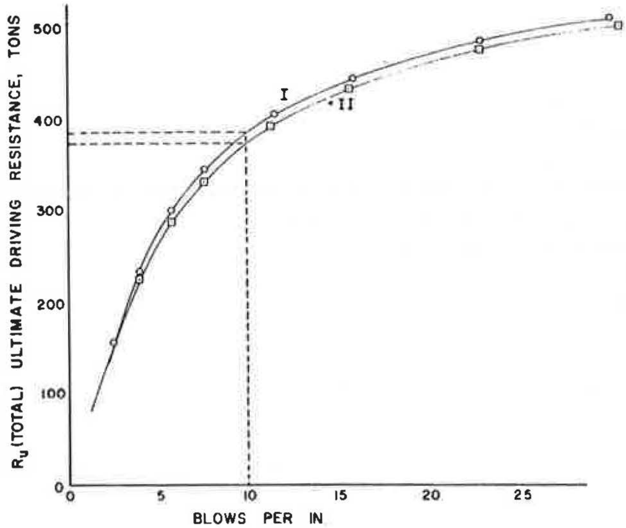

Figure 13. Ultimate driving resistance versus blows per inch for the example problem.

curve relating blows per inch to Ru(total) as shown in Figure 13 (2 curves for the 2 different assumed distributions of soil resistance are shown).

In the field if driving had ceased when the resistance to penetration was 10 blows/in. (a permanent set equal to 0.1 in./blow), then the ultimate pile load bearing capacity immediately after driving should have been approximately 370 to 380 tons as shown in Figure 13. It is again emphasized that this Ru(total) is the total static soil resistance encountered during driving, because the increased dynamic resistance was considered in the analysis by use of J. If the soil resistance is predominantly due to cohesionless materials such as sands and gravels, the time effect or soil setup that tends to increase the pile bearing capacity will be small or negligible. If the soil is a cohesive clay, the time effect or soil setup might increase the bearing capacity as discussed earlier. The magnitude of this setup can be estimated if the sensitivity and reconsolidation of the clay is known. It can also be conservatively disregarded because the setup bearing capacity is usually greater than that predicted by a curve similar to the one shown in Figure 13.

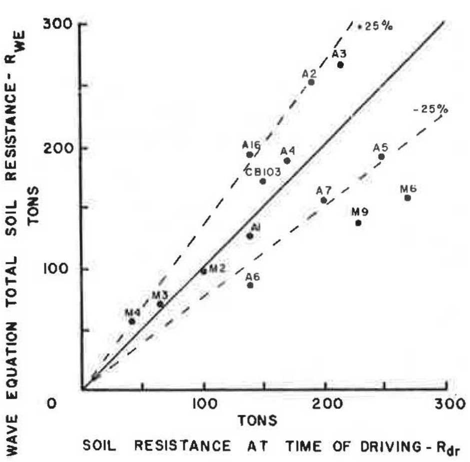

Figure 14. Comparison of wave-equation predicted soil resistance to soil resistance determined by load tests for piles driven in sands (14).

In developing the curves shown in Figure 13, it was necessary to assume that the soil parameters are distribution of soil resistance, soil quake Q, and soil damping constant J.

As shown by curves I and II in Figure 13, small variations in the distribution of soil resistance between side friction and point bearing will not affect the wave-equation results significantly. All that is required is a reasonable estimate of the situation. For most conditions an assumption of soil quake Q = 0.1 in. is satisfactory. The value of J(m) is assumed to be ⅓ of J(p).

Comparison of Predictions With Field Tests

Correlations of wave-equation solutions with full-scale load tests to failure have provided a degree of confidence in the previously described method of predicting static bearing capacity.

For the sand-supported piles, damping constants of J(point) = 0.1 and J′(side) = J(point)/3 were found to give the best correlation. Figure 14 shows the accuracy of the correlation to be approximately ±25 percent.

For clay-supported piles, the damping constants J(point) = 0.3 and J′(side) = J(point)/3 gave the best correlation. The accuracy of the correlation is shown in Figure 15 to be approximately ±50 percent. If more than one soil was involved, the damping constant used was a weighted average.

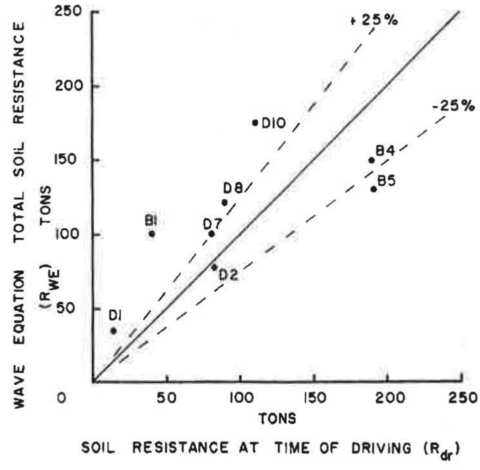

Figure 15. Comparison of wave-equation predicted soil resistance to soil resistance determined by load tests for piles driven in clay (14).

USE OF THE WAVE EQUATION FOR PARAMETER STUDIES

The wave equation can be used effectively to evaluate the effects of the numerous parameters that affect the behavior of a pile during driving. These include, for example, the determination of the optimum pile driver to drive a given pile to a specified soil resistance, the determination of the pile stiffness that will yield the most efficient use of a specified pile hammer and cushion assembly, the determination of the optimum cushion stiffness to make the most efficient utilization of a specified pile hammer and and driving assembly to drive a specific pile, and the determination of the effects of various distributions of soil side and point resistance on the pile bearing capacity, driving stresses, and penetration per blow.

Significant Parameters

The parameters that are known to significantly affect the behavior of a pile during driving are as follows:

1. The pile-driving hammer—(a) stiffness and weight of the pile-driver's ram; (b) energy of the falling ram that is dependent on the ram weight, the effective drop, and the mechanical efficiency of the hammer; (c) in the case of a diesel hammer, weight of the anvil and impulse of the explosive force; (d) stiffness of the cap block, which is dependent on its mechanical properties, thickness, cross-sectional area, and mechanical conditioning effects caused by repeated blows of the hammer; (e) weight of the pile helmet and the stiffness of the cushion between the helmet and the pile (in the case of steel piles the cushion is usually omitted); and (f) coefficient of restitution of the cap block and cushion that influences the shape of the wave induced in the pile and hence affects the magnitude of the stresses that are generated.
2. The pile—(a) length of the pile; (b) stiffness of the pile that is a function of its cross-sectional area and the modulus of elasticity of the pile material; (c) weight of the pile, specifically the distribution of the weight; and (d) existence of physical joints in the pile that cannot transmit tension.
3. The soil—(a) soil quake at the point; (b) soil quake in side friction; (c) damping constant of the soil at the point; (d) damping constant of the soil in friction; and (e) distribution of point and side frictional resistance.

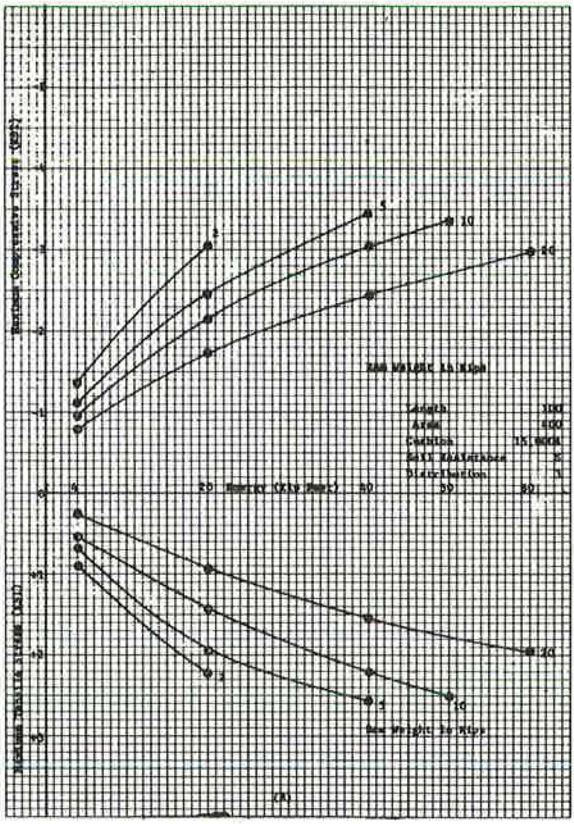

Figure 16. Effect of cushion stiffness, ram weight, and driving energy on stresses for square pile with uniformly distributed soil resistance of 107 tons.

Examples of Parameter Studies

The most notable parameter study that has been reported to date is that presented by Hirsch (16). In that report, the results of 2,106 problems are presented graphically. This study was oriented toward providing information on the effects of ram weight and energy, stiffness of cushion blocks, length of pile, soil resistance, and distribution of soil resistance on the driving behavior of representative square concrete piles. Figures 16 and 17 show representative curves from this study. The results of this study have played a very significant part in formulating recommended driving practices for prestressed concrete piles (17).

Parameter studies of this type have been used by others. McClelland, Focht, and Emrich (18) have used the wave equation to investigate the characteristics of available pile hammers for obtaining pile penetrations sufficient to support the heavy loads required in offshore construction. The parameters varied in this study were the pile length above the mud line, pile penetration, and the ratio of the soil resistance at the pile point to the total soil resistance (Fig. 18A). The results of this study enabled the authors to determine the pile-driving limit versus the design-load capacity as shown in Figure 19. Figure 18B shows the results of one study to determine the effects of varying the unembedded portion of a pile whose total length was held constant. Figure 18C shows the results for the same pile, but with the unembedded length held constant and the embedded length varied. Figure 18D shows the results when the ratio of point soil resistance to total resistance is varied.

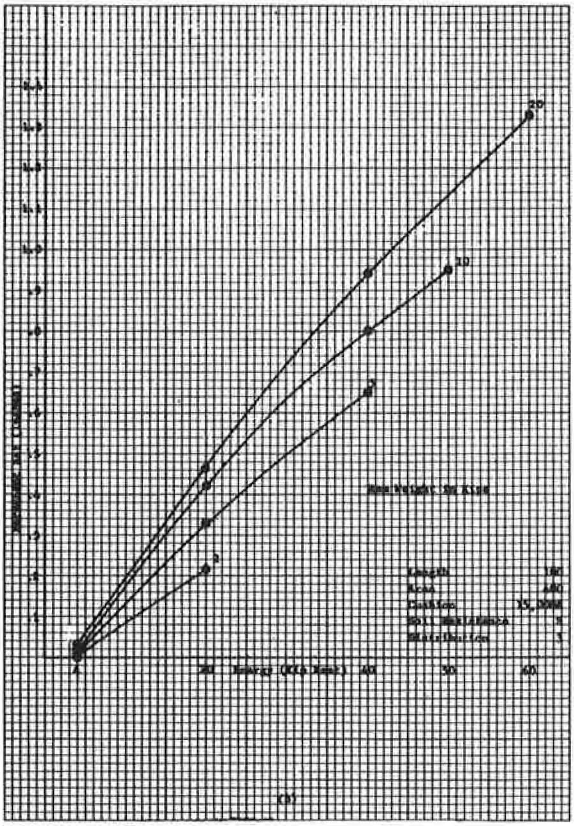

Figure 17. Effect of cushion stiffness, ram weight, and driving energy on permanent set for square pile with uniformly distributed soil resistance of 107 tons.

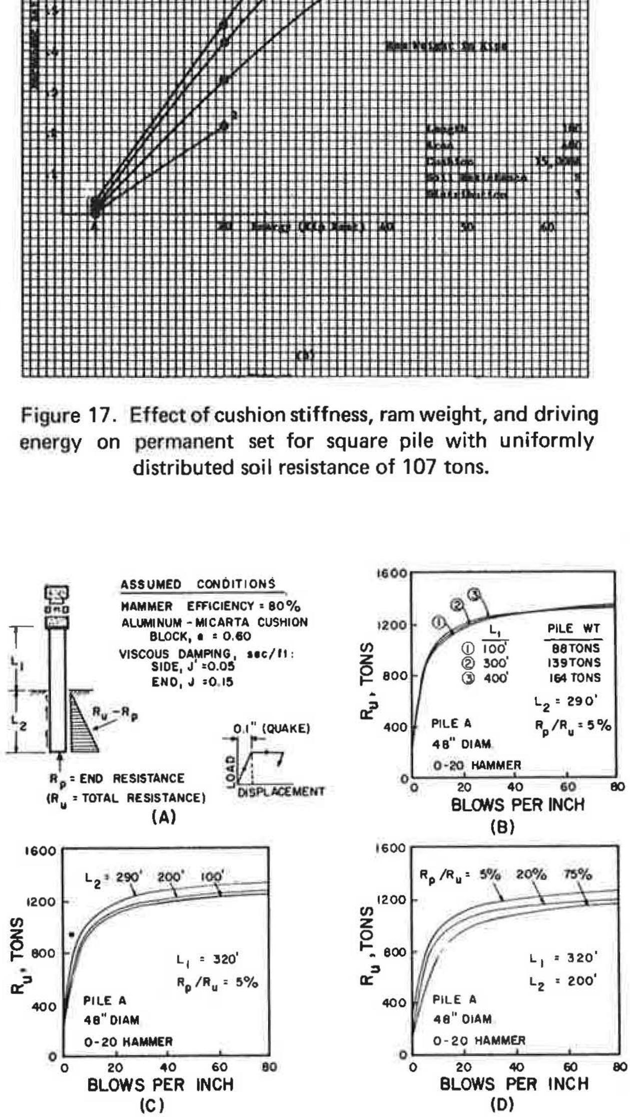

Figure 18. Computer analysis of pile-hammer effectiveness in overcoming soil resistance, Ru, when driving pile under varying conditions: (A) computer input representing conditions of problem; (B) variations in pile length above ground; (C) variations in pile penetration; (D) variations in distribution of soil resistance, Ru (18).

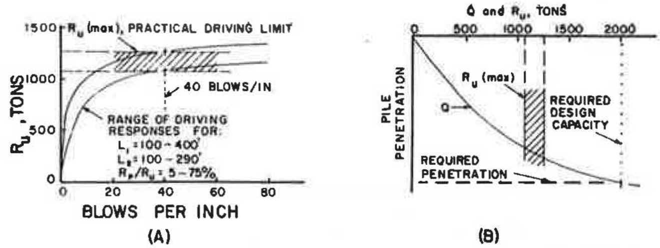

Figure 19. Evaluation of 0-20 hammer (60,000 ft-lb) for driving pile to develop ultimate capacity of 2,000 tons: (A) summary of wave equation analysis (Fig. 18) establishing approximate pile driving limit, Ru (max); (B) comparison of Ru (max) with required design capacity (18).

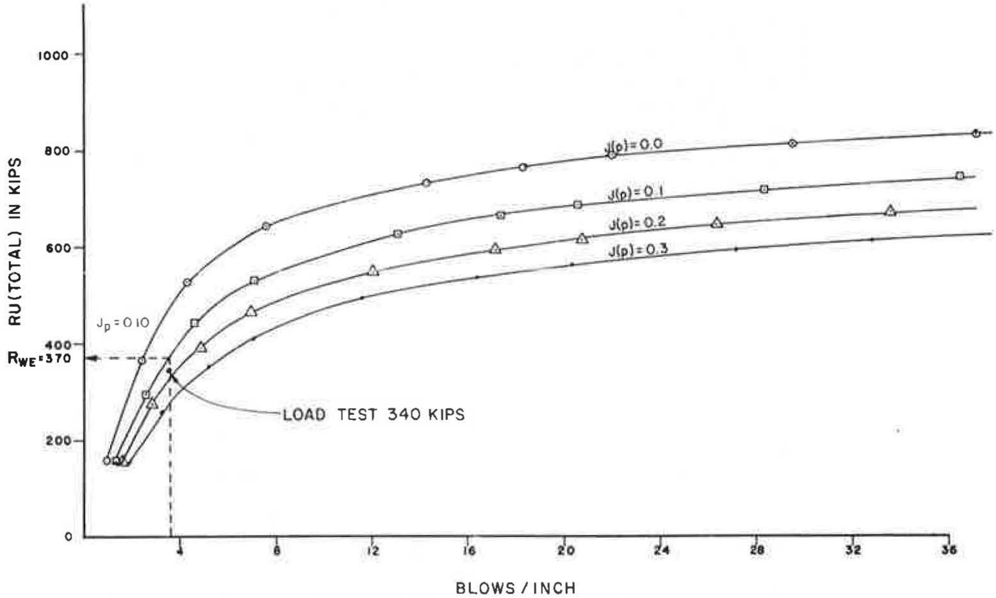

Figure 20. Blows/inch versus Ru (total) for Arkansas load test pile 4.

In an earlier report (14), the writers used the wave equation to determine the soil damping values for various soils encountered in field tests. In this particular parameter study, the pile-hammer-soil system was held constant and soil damping values were varied. By generating an ultimate soil resistance Ru(total) versus blows per inch curve, the appropriate soil damping properties could be determined by comparing the computer-generated solution with the measured data taken from a full-scale field test pile (Fig. 20). This study yielded representative values of the soil damping constants for the soil at the point of the pile and the soil in side friction.

It is not necessary that all parameters for a particular pile installation be known. For example, several problems can be solved in which the unknown parameter is varied between the upper and lower limits. These limits can usually be established with a reasonable amount of engineering judgment.

SUMMARY AND CONCLUSIONS

The numerical computer solution of the one-dimensional wave equation can be used with reasonable confidence for the analysis of pile-driving problems. The wave equation can be used to predict impact stresses in a pile during driving and can also be used to estimate the static soil resistance on a pile at the time of driving from driving records.

By using this method of analysis, the effects of significant parameters such as type and size of pile driving hammer, driving assemblies (cap block, helmet, and cushion block), type and size of pile, and soil condition can be evaluated during the foundation design stage. From such an analysis appropriate piles and driving equipment can be selected to correct or avoid expensive and time-consuming construction problems such as excessive driving stresses or pile breakage and inadequate equipment to achieve desired penetration or bearing capacity.

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Summary and Review of Part II of the Symposium on Pile Foundations

G. A. LEONARDS, Purdue University

•PILE FOUNDATIONS share with cellular cofferdams the distinction of yielding most reluctantly to the advantageous use of theory as a basis for design practice. Papers presented during the second part of the Symposium summarize much of the progress made to date; they offer little comfort to those who view foundation engineering as an applied science. The papers hardly overlap in coverage of subject matter, and it is convenient to consider them separately.

Coyle and Sulaiman review the state of the art concerning the bearing capacity of driven vertical piles subject to static vertical loads. The scope of this paper is indeed comprehensive.

In the discussion of static formulas to predict point-bearing resistance in cohesionless soils, the authors point out that values of the bearing capacity factor N_q' determined theoretically by different investigators vary widely (Fig. 3 in the paper). N_q' is also very sensitive to the angle of shearing resistance ϕ'; a variation in ϕ' of a few degrees results in N_q' being changed by a factor of two or more, regardless of which theory may be selected for the analysis. Yet, natural variations in state of compaction and subsequent changes due to pile driving preclude an accurate estimate of ϕ' in situ. Coupling this with the uncertainty in N_q' even if ϕ' were known precisely leads the writer to the conclusion that predicting point-bearing from static formulas is largely an academic exercise. Correlations with static cone penetration resistances in specific granular deposits can narrow the margin of error.

Concerning shaft friction in cohesionless soils the authors state, "In general, the (unit) frictional resistance decreases with depth, and is independent of initial overburden pressures." The distribution of unit shaft friction due to butt loads added subsequent to driving can be deduced from measurements of the changes in pile load versus depth. In reality, residual stresses due to driving can be very significant, and little is known of their magnitude and distribution; thus, any generalization regarding the actual distribution of unit shaft friction is tenuous at best. There is some justification, however, for stating that (at ultimate load) the fraction of butt load carried by shaft friction, per unit surface area of embedded pile, increases (more or less) linearly with pile penetration and then approaches a constant value (in a manner similar to that shown for point bearing in Figure 4 of the paper). Present concepts regarding load transfer in piles (point-bearing and distribution of shaft friction) are based almost exclusively on short-term loading tests. What happens in the long run is a matter for speculation.

The authors draw attention to cases where the load-carrying capacity of piles increases significantly with time. Losses in driving resistance (and presumably in bearing capacity) have also been observed for piles driven into dense sands, clay tills, and other hard strata. Negative pore pressures that dissipate with time may be the culprit. On the other hand, relaxation of residual stresses could be a contributing factor. Redriving can increase pile penetrations substantially. The concomitant increase in static load capacity may be important if the design specifies piles of high capacity.

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Coyle and Sulaiman also treat the bearing capacity of single piles predicted from dynamic formulas. In this connection, it is useful to summarize the main findings of the Michigan pile study:

1. Depending on the pile capacity and the type of hammer, pile, pile cap, cushion, and soil conditions, the ratio of measured energy transmitted to the top of a pile to the manufacturer's rated hammer energy was 0.25 to 0.65 and averaged around 0.50. Field control of the pile-driving operations was such that little of the energy lost could be attributed to malfunction of the hammer, friction in the leads, and similar extraneous factors.
2. The inconsistencies of dynamic formulas (those based on total energy balance) in predicting ultimate pile capacity as measured by load tests are sufficiently large to make their use for this purpose undesirable, even if the measured transmitted energy is used in the calculation.

As the authors point out, much attention is now being focused on the wave equation as a possible means of predicting pile capacity. The input data for the wave equation include the properties of the hammer, pile cap, cushion, and pile; the static point-bearing and shaft friction resistance offered by the soil, and the deformations at which these resistances are mobilized; and the additional point-bearing and frictional resistance of the soil due to dynamic penetration. The output is the penetration per blow and the stresses in the pile. The inverse procedure—prediction of the static resistances from one output, the penetration per blow—is a much more tenuous matter that, in the writer's opinion, is presently beyond the range of routine application. However, the wave equation has proved its worth for the purpose of matching the pile and driving equipment to achieve desired penetrations efficiently and for guarding against overstressing in the driving process.

Concerning the bearing capacity of pile groups, the efficiency factor E for piles in granular soils is of small consequence (E may exceed two for closely spaced, relatively long piles driven into loose cohesionless soils) because the design of such pile groups is governed by tolerable movements. The prediction of group displacement is often based on the expected movement of a single pile at working load and an estimate of the factor by which this movement is multiplied to obtain the settlement of the group (Fig. 13 in the paper). The movement of a single pile at working load can be estimated from load tests; however, it would be useful if this movement could be predicted by less costly means. Skempton et al. (51 in the paper) suggested that the displacement of a pile in granular soil is (approximately) a unique function of the ratio of applied load to the ultimate capacity of the pile, although this is admittedly an oversimplification. The authors mention the development of an in situ testing device at Texas A&M University that measures skin friction and point-bearing as a function of pile movement. Such a device could prove to be very useful, and it is hoped that the results of this research will soon be published.

It is not widely recognized that the curve proposed by Skempton et al. (Fig. 13 in the paper) for estimating the settlement of pile groups in granular soils is only tentative and was based on 2 case records presented at the Second International Conference on Soil Mechanics and Foundation Engineering: One was by Feagin and involved Mississippi River silty sands, and the other was by Vargas of Brazil where the granular soils contained significant amounts of clay. The writer is aware of 4 additional case studies where the ratio of the measured settlement of the pile group to that predicted from Skempton's curve ranges from 0.2 to 2.0—hardly a situation that calls for complacency. It appears that the settlement ratio (group to single pile) depends greatly on whether the loads are transmitted in point-bearing or by shaft friction, on the length of the piles, and on the degree to which driving the pile group further densifies or loosens the soil. This is an area where additional research is badly needed.

Concerning the bearing capacity of friction pile groups in soft to medium clays, there is little to add to the recommendations summarized in the paper except to point out that only model tests are available to confirm the proposed procedures and that heaving is likely to be a problem if the pile spacing is 3 diameters or less. To calculate the settlement of such pile groups due to consolidation, Terzaghi's suggestion of transferring the load to an elevation two-thirds the pile length from the top of the piles is clearly a first approximation. More rational methods for predicting these settlements are still in the research stage.

Fuller and Hoy review qualitatively the procedures commonly used for performing pile load tests and provide a useful checklist of factors to be considered in planning a testing program. A feature of the paper is the description of the quick test method used by the Texas Highway Department, which is a simpler version of the constant rate of penetration test developed at the Building Research Station, England. Correlations are given for the maximum proven design load as interpreted from the quick load test and from the standard AASHO test. For effective pile penetrations of 20 to 60 ft in sand and clay and silty, sandy clay the correlations are extremely good. (It is interesting to note that the K-factor that multiplies the safe load given by the Engineering-News formula to yield the maximum proven design load ranges from 0.6 to 4.9—another example of the unreliability of dynamic formulas for predicting pile capacity.) If the correlations prove to be valid for a wide range of soil types and pile penetrations, the quick test method can replace conventional load test procedures at substantial savings in time and costs.

Although the defects of dynamic formulas are often cited, the limitations of conventional load test results are seldom mentioned. Such tests usually record the butt load and the corresponding butt deflection on the loading and unloading cycle. In the writer's opinion the specifications should provide a load capacity sufficient to establish the ultimate resistance of the pile. The ultimate resistance is clearly defined if the pile "plunges" into the ground (the deflection per unit time, at constant load, remains constant or begins to increase with time). If this does not happen an interpretation such as that given in Figure 6 can be made. In the latter case the test should not be stopped unless the pile deflection exceeds 20 percent of the tip diameter (or the structural strength of the pile is being approached) to ensure that full point resistance has been mobilized. Such criteria as "the load at 0.25 in. net deflection" are merely empirical procedures intended to estimate the pile load at which the settlement of the pile group is likely to be tolerable. They may bear no relation to the ultimate pile capacity.

A conventional load test can only establish the capacity of a pile at the time of testing under conditions where the ultimate point-bearing and ultimate shaft friction are mobilized simultaneously. The following factors obscure the relation between such test results and the subsequent performance of a pile foundation:

1. The net settlement curve obtained from rebound data gives only an indication of tip penetrations because the stresses in the pile and in the surrounding soil during unloading can be very different from those during loading. The discrepancy may be reduced, but probably not eliminated, by cycling the load several times.
2. The settlement of the pile at a given butt load can change drastically with time if a transfer of load from shaft friction to point-bearing takes place. Such load transfers can occur because of (a) creep of the surrounding soil, (b) consolidation of clay layers caused by placement of fill, groundwater lowering, or remolding or excess pore pressures or both caused by pile driving, and (c) minor vibrations associated with normal occupancy of the structure.
3. There is no consistent relationship between the settlement of a single pile and the settlement of the pile group at the same load per pile. Therefore, selecting a design load on the basis of the load at a given gross or net deflection, or at a given fraction of the ultimate pile capacity, is equivalent to accepting an unknown factor of safety with respect to satisfactory performance of the foundation.

If properly conducted, load tests can provide important information regarding the performance of a single pile for load transfer and soil conditions extant at the time of the test. Although this information is superior to that which can be deduced from static or dynamic formulas, it is not sufficient for rational design of pile foundations. In situations where experience with given types of pile foundations is lacking or very limited, the designer is compelled either to be very conservative or to take an "uncalculated risk" with respect to satisfactory performance.

Davisson presents an exceptionally lucid summary of the nondimensional procedure for estimating the shears, moments, and deflections in laterally loaded vertical piles based on the subgrade modulus k model. As this model is clearly an approximation to the actual soil response, the selection of appropriate magnitudes and distributions of k-values is crucial to the potential success of the analysis. In this connection, Davisson's suggestions for selecting k-values (Table 1 in the paper) are welcome because they represent a synthesis of considerable experience.

Recent studies indicate that k decreases significantly with increasing deflection especially near the ground surface. As the deflections are maximum near the surface, and this is the zone where k has its greatest influence on the stresses in the pile, the practical importance of the nonlinearity in k deserves further study. The author suggests that a load test could be conducted, which is strongly endorsed by the writer. In fact, there is at least as much to be gained from a lateral load test as from the corresponding vertical load test; however, the writer would prefer to conduct the test on a prototype pile. (It is often assumed that the lateral resistance of the ground is minimum when the pile supports no vertical loads. Is there any field evidence to justify this assumption?) Not enough is known about the effect of pile width and nonlinearity of k with deflection; elimination of these uncertainties should more than compensate for any additional cost of testing the prototype pile.

The author correctly emphasizes the importance of the effects on k of repeated loading and pile spacing if the performance of a pile foundation is to be assessed, and states that their combined effect can result in $k_{effective}$ being as low as 10 percent of that applicable to initial loading of a single pile. This statement is based on small-scale model studies of the group effect while the repeated load effect stems largely from the behavior of a single pile. The suggestion that $k_{eff} = 0.1k$ may be unduly conservative, as indicated by the following example. Lateral load tests were performed on 2 groups of 150 piles. The lower portions of the piles were embedded in soft rock and the upper portions extended through submerged, relatively loose, uniform sand. Pile spacing in the direction of the load was 2½ to 3 diameters. The design load was applied and cycled 5 times. Because of cycling, k_{eff} reduced to 55 and 80 percent of k respectively (30 percent was indicated in the paper). Actually, maintaining the lateral load for 24 hours decreased k_{eff} almost as much as load cycling. Unfortunately, a load test on a single pile was not conducted, hence no definitive statement can be made regarding group action. Judging from the observed deflection of the pile cap and the loose nature of the sand, however, it is unlikely that k_{eff} was 0.25k as indicated in the paper. More field measurements on laterally loaded pile groups are badly needed.

York presents an interesting survey of the structural behavior of driven piles, including factors such as pile stresses during and after driving, the stresses in and load capacity of bent and damaged piles, the potential magnitude of negative friction (dragdown), and the likelihood that many pile foundations may never be subjected to their design loads. Attention is also directed to the wide variation of accepted values for allowable stresses in concrete, steel pipe and steel H-piles. To the writer's knowledge this paper is unique in the literature.

Although the examples cited are most instructive and a valuable guide to good judgment, it is difficult to draw general conclusions from the survey. For example, it is true that steel pipes filled with concrete, when tested in the laboratory, have ultimate strengths that suggest that working stresses allowed by most codes are too conservative. On the other hand, definitive methods for assessing the corrosion that may occur during the intended life of the piling are difficult to come by. In this connection, the study by the National Bureau of Standards on the corrosion of H-piling is encouraging but not entirely comforting. Moreover, the corrosive activity of the ground can change with time often because of factors that are difficult to anticipate at the time of construction. In another vein, bent and damaged piles have consistently shown surprisingly high load capacities under short-term loading. If it is argued that their capacity may not deteriorate much with time on the grounds that many existing foundations supported by such bent or damaged piles or both have performed satisfactorily, consideration should be given to the high probability that in most instances the pile stresses were correspondingly low. Would they perform equally well in the long run at higher working stresses? Foundation performance may be likened to a chain with many links; changing any one link (such as using higher working stresses) may lead to unexpected failures in other links. The "good field data" that the author recommends be collected must be comprehensive in scope and long term in nature. Such data are difficult to generate, but they are a prerequisite to more rational design methods. Unless (a) the loads supported by the piles are clearly defined, (b) the piles can be inspected and only straight, undamaged piles are accepted, and (c) pile deterioration can be evaluated accurately, conservatism in structural design of piles is fully justified.

Structural Behavior of Driven Piling

DONALD L. YORK, The Port of New York Authority

A review of the structural behavior of driven piles is made, and it is shown that, except for piles that fail because of improper construction or piles that deteriorate in service, there are very few reports of structural failure. The various reasons for this excellent record are examined, and the conclusion is that one of the principal reasons is the remarkable load-carrying capacity of damaged piling. Several case histories are described involving the behavior of damaged piling under load and load tests on pile groups. The implications of these findings are discussed in relation to the allowable stresses used in practice, and a few situations are cited where it appears that the allowable loading on driven piles could be safely increased.

•VERY FEW FAILURES of pile foundations can be attributed to structural failure of the piling. Except for piles that fail because of improper construction or piles that deteriorate in service, the few reports that have been published involve piles that failed because the actual pile loading greatly exceeded the design loading. This paper examines the reasons for this excellent record. It investigates the various types of damage that occur during pile installation and the effects of damage on pile capacity. Finally, the allowable stresses used in practice are briefly surveyed. The discussion is limited to driven piles.

There are a few reports of structural pile failure due to dragdown loading resulting from settlement of the surrounding soils. One case involves 85-ft-long timber piles driven through 40 ft of fill and 28 ft of soft bay mud to end-bearing on decomposed rock. The tops of the piles settled several inches and, because the tips were restrained, it is presumed that the piles broke. The potential dragdown load was estimated to be almost 200 tons per pile (1). Chellis (2) reports a similar case involving 100-ft-long steel H-piles driven end-bearing to rock through a deep bed of plastic clay. The area was brought to grade with 15 ft of slag fill, which also supported a floor slab carrying very heavy loads of armor plate. Within a year settlements of 1 ft occurred, and the H-piles jackknifed. The total loads in this case were estimated to be 350 tons per pile.

Dragdown loads of this magnitude are quite possible. In Norway, Bjerrum and his colleagues (3) have measured dragdown loads of 120 metric tons on a 12-in. steel pipe and 300 metric tons on a 20-in. pipe. The piles were driven through 100 ft of fill and soft clay to bearing on rock.

Another type of structural pile failure is reported to be the cause for large settlements of an ore-storage dock (4). Step-taper piles, used to support a rigid concrete slab, were driven to end-bearing through 80 ft of fill and soft clay. Some of the piles stopped in an underlying layer of hardpan while neighboring piles were able to penetrate through the hardpan and reach bedrock. This condition resulted in piles of unequal stiffness. When the slab was loaded a gradual, progressive failure took place, presumably because the stiffer piles became overloaded and failed. The piles had an 8-ft-long 10.75-in. O. D. pipe section at the tip and were designed for a 150-ton capacity. This is a very heavy loading for this pile in an end-bearing situation and, no doubt, this was a contributing factor to the failure. Failure probably occurred in shear immediately above the composite connection. Settlements of almost 2 ft were observed.

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Perhaps a more extensive study would uncover other reports of failures and, of course, some failures go unreported; but judging by the scarcity of published reports, there are remarkably few structural failures of driven piles.

PILE BEHAVIOR

The first serious loading a pile is subjected to occurs when it is driven into the ground. In most cases the stresses that a pile experiences during driving exceed the design stresses and are the highest stresses that the pile ever experiences. If the pile can withstand these forces without damage, it has, in effect, been pretested. A few measurements of driving stresses in piles are reported in the following.

A second reason for the excellent record of the structural behavior of driven piling is that piles seldom receive their full design loading. The principal reason for this is that the usual design provisions for live loading are quite conservative and are seldom, if ever, realized in practice. However, in addition to this there is also a small but growing body of evidence to indicate that the actual column loads may never reach the piles. It seems that when piles are driven in groups and capped a significant proportion of the column load is transferred directly from the pile cap into the soil.

Evidence of this phenomenon is demonstrated in tests by Vesic (5) on large-scale models of pile groups in sand. The model piles were 4-in. diameter aluminum tubes that were jacked into the sand to a depth of 60 in. A concrete cap was then cast directly on the sand surface. The test loads were regulated with an electronic proving

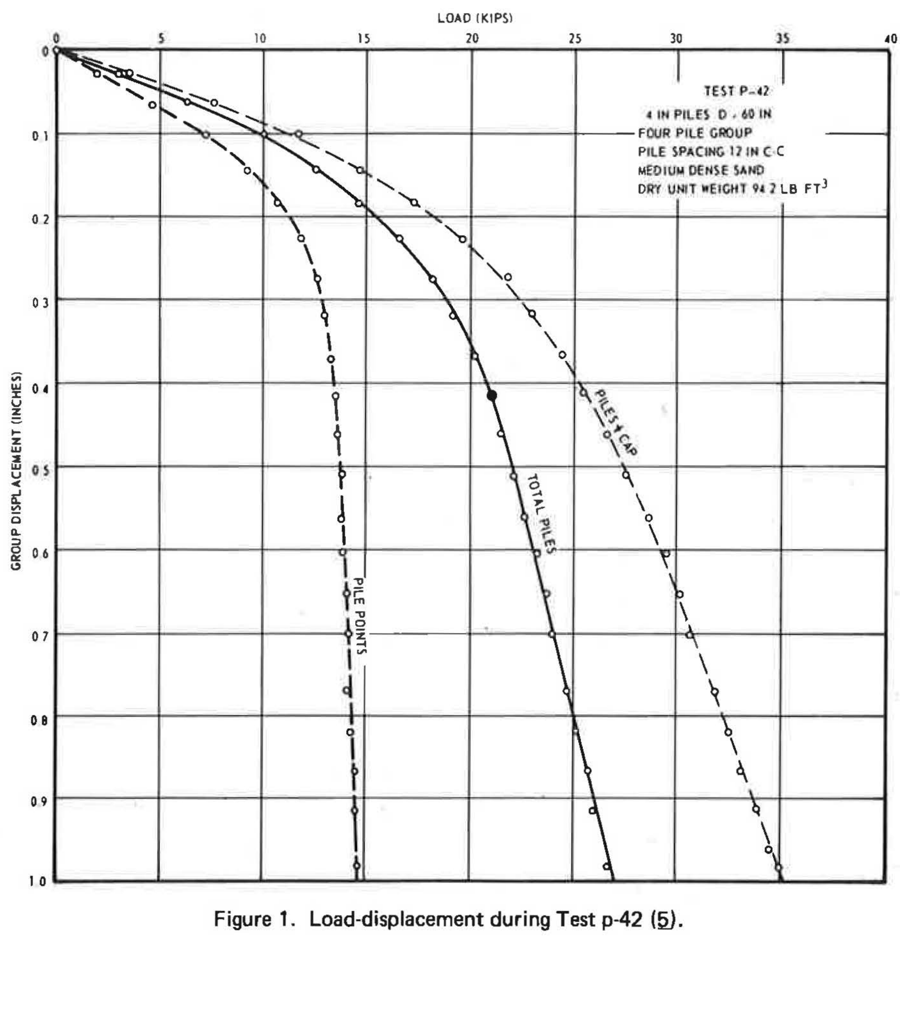


Figure 1. Load-displacement during Test p-42 (5).

ring, and the pile loads were measured with strain gages. Test results of a 4-pile group in medium-dense sand are shown in Figure 1. The proportion of the total load carried by the cap increased from about 19 percent at a displacement of 0.1 in. to about 23 percent at a displacement of 1.0 in. This test is fairly typical.

Very few field tests have been performed on pile groups. One of the few published reports (6) concerns a 9-pile group of 12-in. pipe piles spaced at 3-ft centers. The piles were pushed 19 ft into a deep deposit of soft organic silt. A shallow excavation was made in order to place the cap directly on the organic silt layer. Test loads were applied with a hydraulic jack, and the soil reactions at the bottom of the pile cap were measured with 3 earth pressure cells. As shown in Figure 2, at loads less than 140 metric tons about 10 percent of the group load was carried by the base reaction on the cap. At this point the cap had settled about 0.6 in. As the load was increased further, yield occurred and the base reaction increased rapidly. At failure, slightly more than 20 percent of the total load was carried by the bearing pressure at the base of the pile cap.

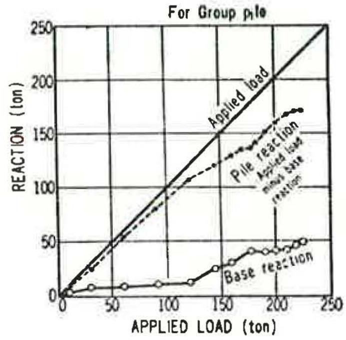


Figure 2. Load carried by piles and pile cap (6).

These data show that, even for very soft soils, the load reaching the piles is less than the full applied load.

Another favorable circumstance that contributes to the excellent record of the structural behavior of driven piling is that piles are frequently stronger than is recognized by our present design procedures. The concrete core in a concrete-filled pipe pile, for example, benefits from the confinement provided by the steel pipe. Recent experiments have shown that where concrete-filled steel tubes act as short columns ($KL/r < 35$), the ultimate load-carrying capacity is 20 to 30 percent greater than that predicted by theory (7). This may be of particular interest, because some building codes now permit piles to be designed on the basis of ultimate strength. Also, the materials used to construct piles are usually stronger than we credit them with being. Design stresses are based on specified minimum strengths, and it is obvious that the average strength must be greater than the specified minimum. It is not unusual, for example, to find that the average yield strength of steel pipe exceeds the specified minimum by as much as 10,000 psi.

PILE DAMAGE

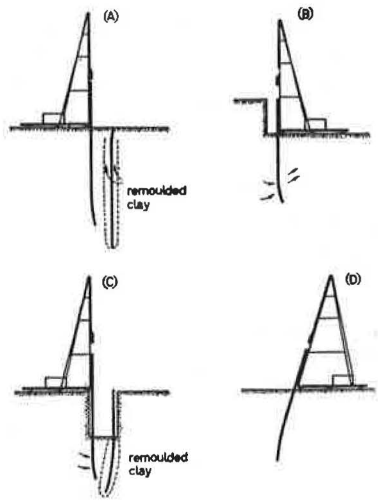
All of these factors act to minimize the possibility of structural failure. However, driving piles into the ground is a brutal process that frequently causes damage to piling, and this increases the danger of structural failure. Bent piling and tip damage are the more prevalent and serious types of damage. Damage to the tips of piles usually occurs because of overdriving, but it may also occur if the pile hits an obstruction. For concrete piles easy driving can be as harmful as hard driving. When concrete piles are driven into a soft stratum, tension cracking sometimes occurs because a tensile stress wave is reflected up from the pile tip (8).

A bent pile usually results from hitting an obstruction, but it appears that bending can also be due to a variety of other causes. The driving of slender piles into soft soils has been studied at the Norwegian Geotechnic Institute (9), and some of the factors that may contribute to pile bending are shown in Figure 3. Bending will obviously increase the danger of buckling, and this can be a serious problem in soft clays because of the limited ability of the soil to provide lateral support.

Hanna (10), based on a study of bending of long steel H-piles, suggests that bending results from a mechanism that is inherent in all pile-driving work (Fig. 4). As a pile punches through the overburden, an asymmetric failure pattern develops at the pile tip, and this results in an eccentric tip reaction. This causes a bending moment at the pile tip that may be sufficient to initiate bending even under light driving conditions in uniform soils.

The pile will have a tendency to bend in the direction of newly driven neighboring piles around which the shear strength of the clay is previously reduced.

Along the walls of an excavation, the displacement will take place mainly in the direction of the excavation; and the pile will tend to bend away from the wall.



In trenches and shafts the displaced clay will primarily be squeezed up in the bottom of the excavation, and the piles will consequently have a tendency of a curvature against the centerline of the excavation.

Because of their weight, batter piles will tend to bend in a downward direction.

Figure 3. Causes of pile bending (9).

Another factor that may cause bending is the misalignment of spliced sections. When the leads of the driving rig are spliced in, it is particularly difficult to control alignment. Finally, there are certain pile joints and connectors that either permit limited rotation or have reduced moment restraint. Bending sometimes occurs at these points of inherent weakness.

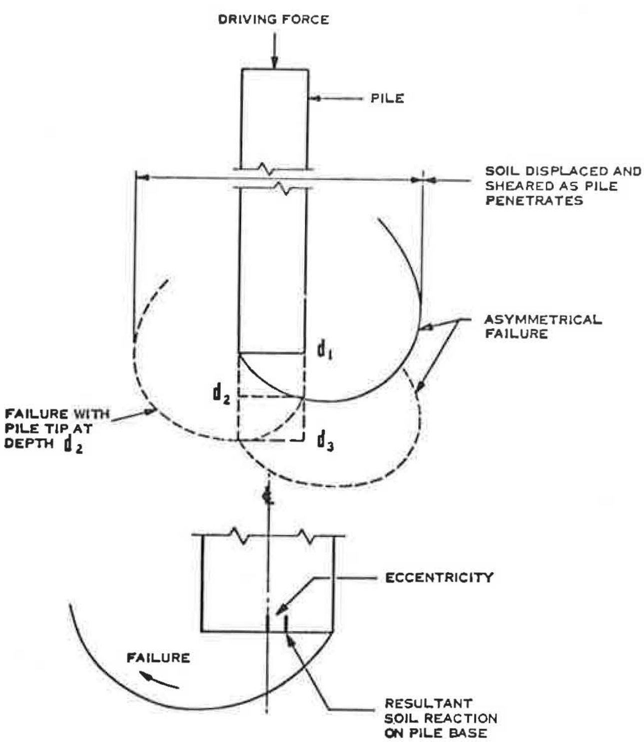


Figure 4. Idealized mode of soil displacement during driving (10).

The behavior of damaged piling under loading is difficult to determine because surprisingly few test data are available. Load tests are hardly ever performed on piles that are known to be damaged and, unfortunately, piles that cannot be inspected are seldom extracted after load testing to investigate for possible damage. However, information that is available indicates that damaged piles have remarkable load-carrying capacity.

A search of the literature has uncovered only a few reports of load tests on piles with tip damage (11, 12, 13). All of these involve steel H-piles that were overdriven into very dense material. The following example is typical of these test results.

Several steel H-piles, including one pile instrumented with strain gages, were driven through the Chicago blue clays and into a very dense hardpan (11). The driving records for 3 piles are shown in Figure 5 together with a typical soil profile. The piles drove easily to a depth of 60 ft, where the driving resistance began to increase steadily as the piles penetrated into the firmer clays. The hardpan occurs at a depth of 80 ft, and very hard driving was required to penetrate into this stratum. Pile 02 was driven to a resistance of 125 tons by the Engineering-News formula and penetrated less than 2 ft into the hardpan. The other 2 piles were purposely overdriven and penetrated deeper into the hardpan. A maximum driving stress of 17,000 psi is reported for the instrumented pile (Pile B4). The piles were driven with a single-acting hammer with a rated energy of 30,225 ft-lb and equipped with a helmet and hardwood block cushion.

The load test results are shown in Figures 6 and 7. There are 2 significant points. The first is that the piles that were overdriven experienced less settlement and had greater load-carrying capacity than did Pile 02. The second is that the strain-gage measurements show a high rate of load transfer in the hardpan stratum.

At the conclusion of the load tests, the 2 piles that had been overdriven were extracted. The condition of the instrumented pile is shown in Figure 8. The manner in which the flange was folded at the pile tip would indicate that the pile hit a boulder, although no boulders were encountered in either of the 2 borings at the site. There is also a bend in the flanges about 10 ft from the pile tip. As shown in Figure 9, the bottom of Pile B2 is completely mangled, and it is difficult to believe that this pile supported a 300-ton load. However, this pile was driven deeper than the other piles and, based

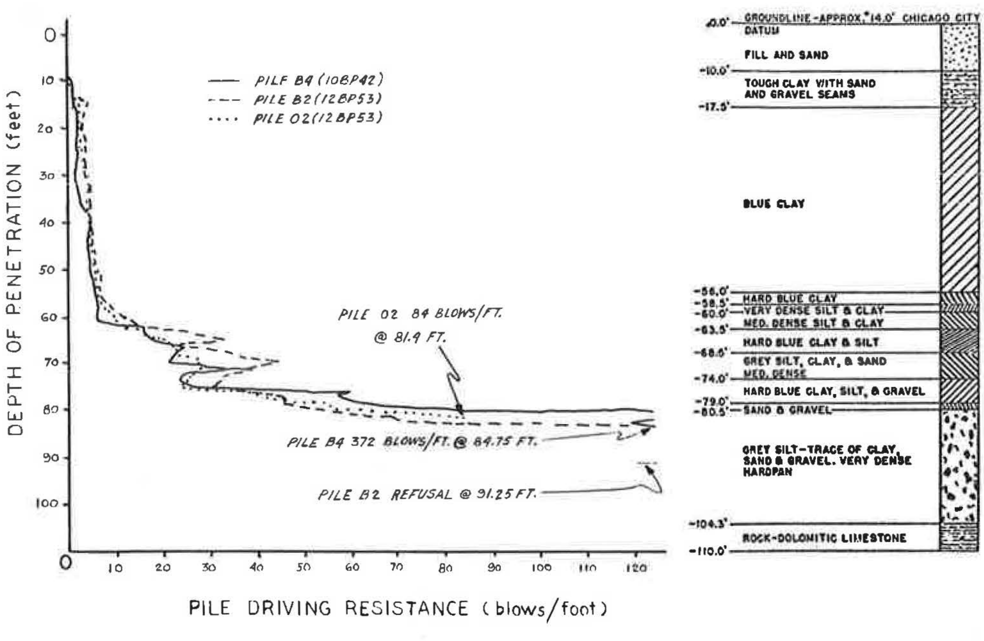


Figure 5. Pile driving records and typical soil profile for Piles B4, B2, and 02 (11).

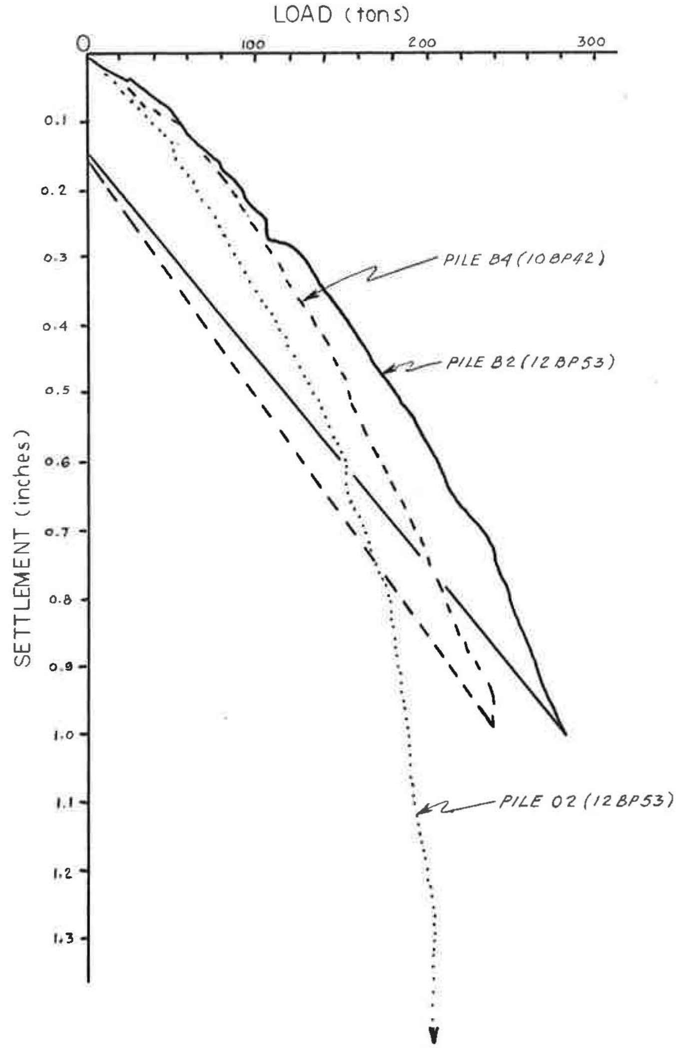


Figure 6. Load-settlement for Piles B4, B2, and 02 (11).

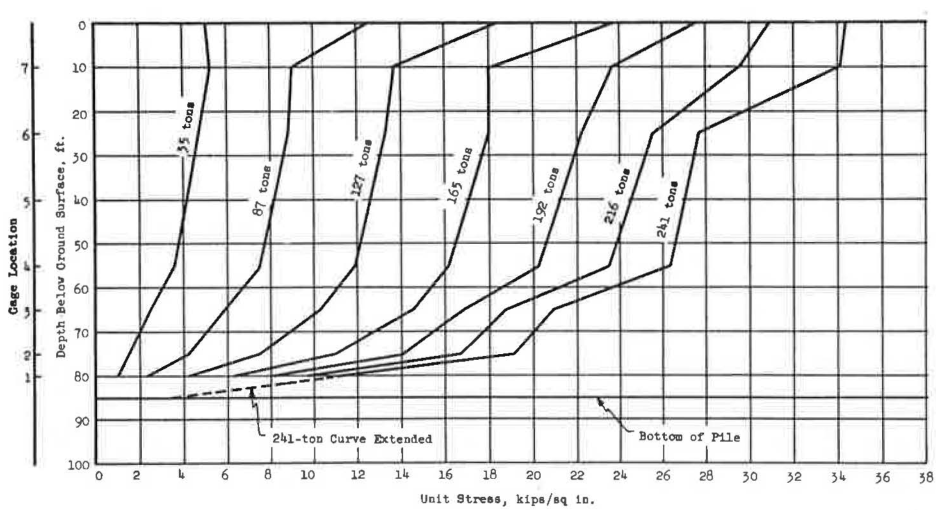


Figure 7. Distribution of stress in Pile B4 (11).

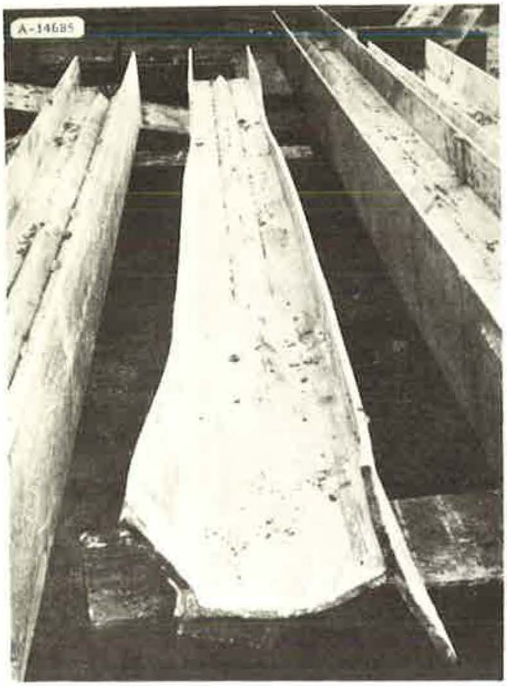


Figure 8. Lower end of Pile B4 after extraction (11).

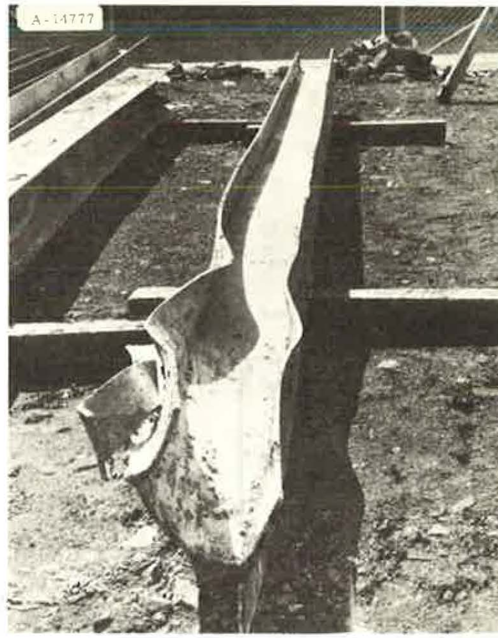


Figure 9. Lower end of Pile B2 after extraction (11).

on the strain-gage readings on Pile B4, it is likely that substantial loading did not reach the badly damaged portion of the pile.

Pile 02 was not extracted, so its condition cannot be definitely established. Because this pile was not overdriven, however, it seems likely that the pile was not damaged. As previously noted, this pile had the lowest load-carrying capacity.

On several occasions piles have been exposed during the driving of tunnels. Peck (14), for example, reports encountering groups of timber piles in Chicago that were in such a twisted configuration that it was impossible to tell what piles belonged to a particular group. These piles had previously supported a 13-story apartment house without apparent signs of distress. As Peck points out, the significant point is that these piles are not different from those under hundreds of other buildings in Chicago.

A very interesting series of load tests performed on bent steel H-piles in Ontario, Canada, is reported by Hanna (10). Three H-piles were driven through a 140-ft deposit of glacial-lake clays. Except for the upper 35 ft, which were desiccated, the soils appear to be normally consolidated. As shown in Figure 10, the soils just below the desiccated layer are quite soft, with shear strengths on the order of 500 psf. Bedrock, a shale, occurs at a depth of 150 feet. Overlying the bedrock is a thin layer of dense sandy till.

The piles were driven to a final resistance of 40 blows per inch with a diesel hammer having a rated energy of 39,700 ft-lb. The driving resistance gradually increased from about 20 blows per foot near the ground surface to about 60 blows per foot at a depth of 140 ft. The resistance then increased rapidly as the pile penetrated through the till layer to bedrock.

Deflections were measured in 2 piles with an inclinometer. One leg of a 5-in. angle was tack-welded to the pile web and the other to the inside surface of the flange to provide a duct for the inclinometer casing. The duct extended to within 10 ft of the pile tip. The inclinometer readings (Fig. 11) show both piles to be severely bent around the weak axis, with each pile having a minimum radius of curvature of less than 200 ft. This places the stresses in the flange area well above the yield point.

The load test results of one of these bent piles (14BP73) are shown in Figure 12. The test results appear to be quite satisfactory. The net settlements are appreciable for an end-bearing pile, but most building codes would consider this pile satisfactory for an allowable load of at least 100 tons.

Despite these favorable load test results, the bent piles were considered to be unacceptable for the support of the permanent structure. It was reasoned that under long-term loading, the highly stressed soil supporting the pile in the vicinity of the bent section would consolidate and that this would cause the pile to settle. For this reason the construction piles were driven into preaugered holes in an effort to minimize bending. The results of a load test on a steel H-pile (12BP74) that was driven into an 80-ft-long preaugered hole are also shown in Figure 12. This test is much more satisfactory. The gross settlements are similar to those of the bent pile, but this is because the preaugering has removed the surrounding soil to a depth of 80 ft. Note that the net settlements are very small; at 200 tons the net settlement is only 0.06 in. This pile can be accepted without qualification for a design load of at least 100 tons.

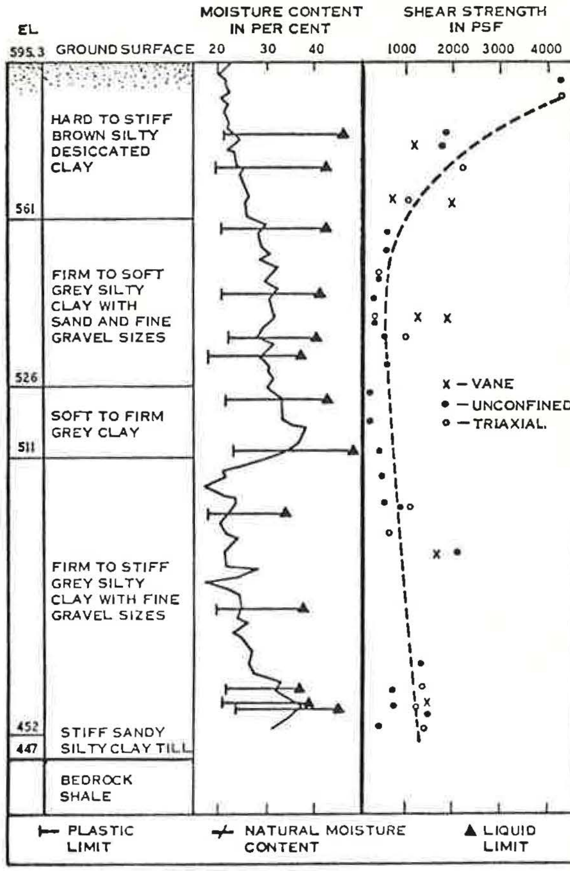


Figure 10. Geologic profile at Lambton generating station (10).

As part of the pile-testing program for the Columbia Lock and Dam on the Ouachita River in Louisiana, the Corps of Engineers drove, load-tested, and extracted a pair of 14-in. steel H-piles (14BP73) that were instrumented with strain gages (15).

The piles were driven through 17 ft of stiff fat clays and 14 ft of very dense sand into a tertiary deposit of stiff clay mixed with layers of silty sand. Spoon blows in the tertiary deposit ranged from 60 to 90. Pile 2 was driven with a differential-acting hammer with a rated energy of 36,000 ft-lb. It penetrated to a depth of 51 ft, where it met refusal. Pile 3 was driven with a single-acting hammer rated at 48,000 ft-lb per blow. It penetrated about 30 ft deeper than Pile 2, but below 48 ft the driving was very hard, with driving resistances that ranged from 100 to 700 blows per foot. A total of 479 blows was required to drive the final 7 in.

The strain gages were read during driving and at the completion of the driving. A record was thus attained of both the peak dynamic strains during driving and the residual strains after driving. For Pile 3, the measured peak dynamic strains during driving are shown in Figure 13. The corresponding peak dynamic stresses were 26,100 psi for Pile 3 and 21,500 psi for Pile 2. For Pile 2 the measured residual strains were small. For Pile 3, however, strain gages mounted on the pile web recorded a residual strain of 3,000 microinches at a point about 25 ft above the pile tip. This is well above the yield point, and it was thought that the pile was bent.

Following the completion of the load tests the piles were pulled. The results are shown in Figure 14. Pile 2 was undamaged, but Pile 3 was badly bent, the tip being 6.1 ft from the axis of the straight portion of the pile. The strain gage measurements

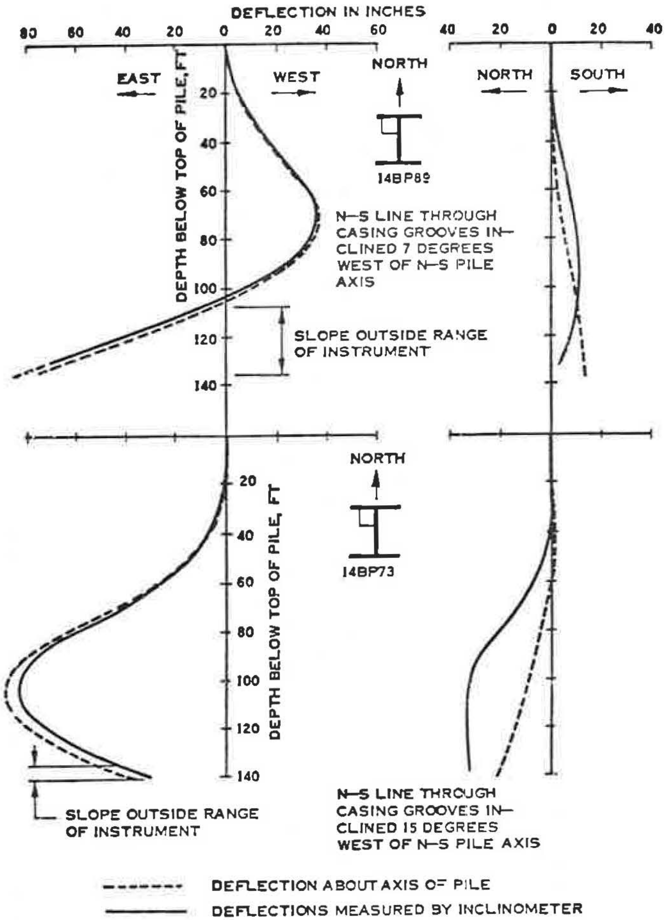


Figure 11. Measured deflections of bent H-piles (10).

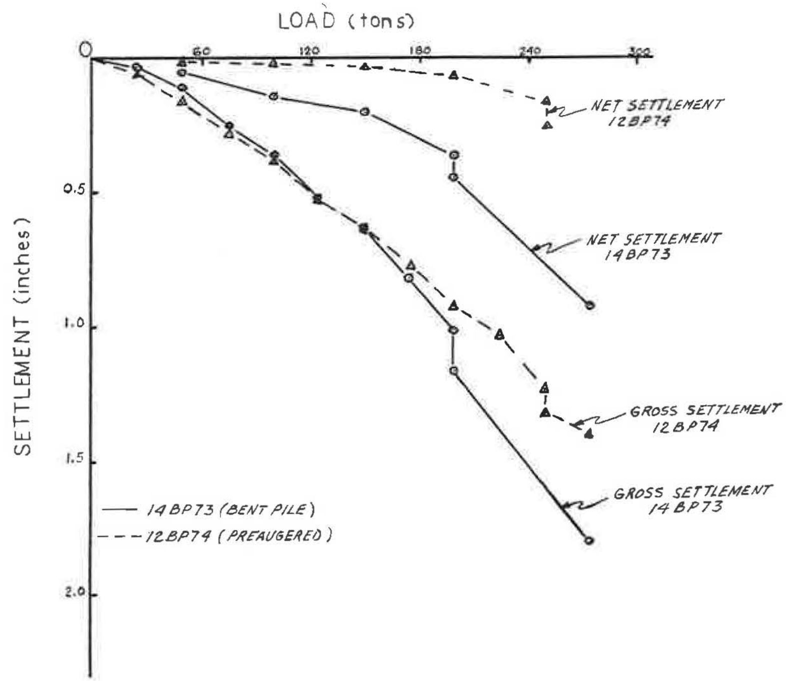


Figure 12. Load-settlement for 2 piles (10).

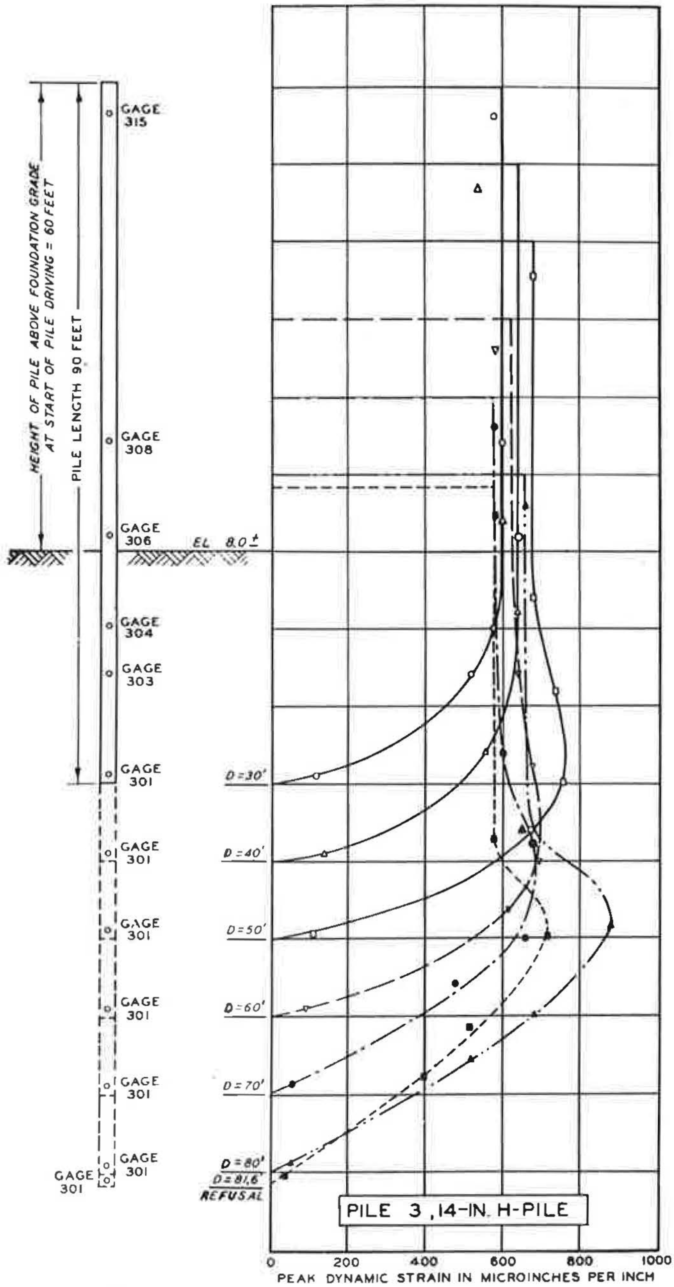


Figure 13. Peak dynamic strains during driving for Pile 3 (15).

obtained during the load test (Fig. 15), showed that the bent section of the pile carried a load of 200 tons.

The load test results are shown in Figure 16. The test results on the bent pile are quite satisfactory, but for Pile 2 the gross and tip settlements are both smaller. The load test results for the bent pile are also very similar to those for a pile that was load-tested during the construction of the lock. The construction piles were driven

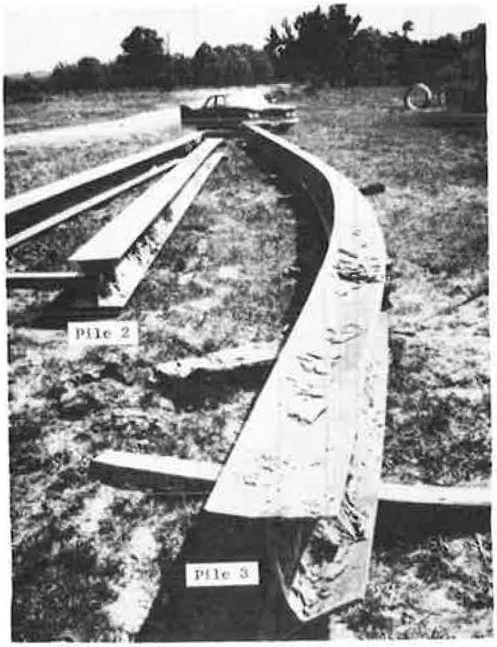


Figure 14. Piles 2 and 3 after extraction (15).

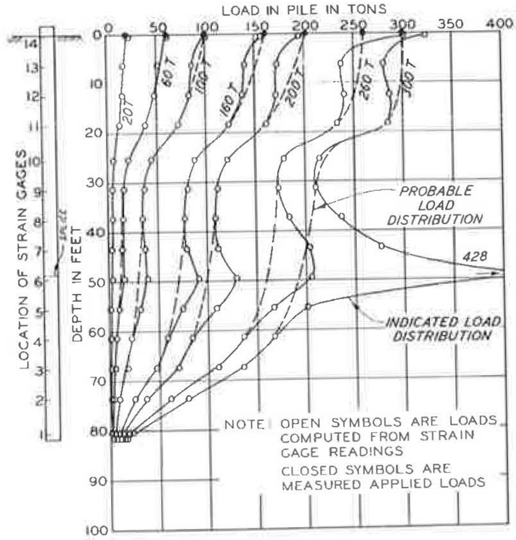
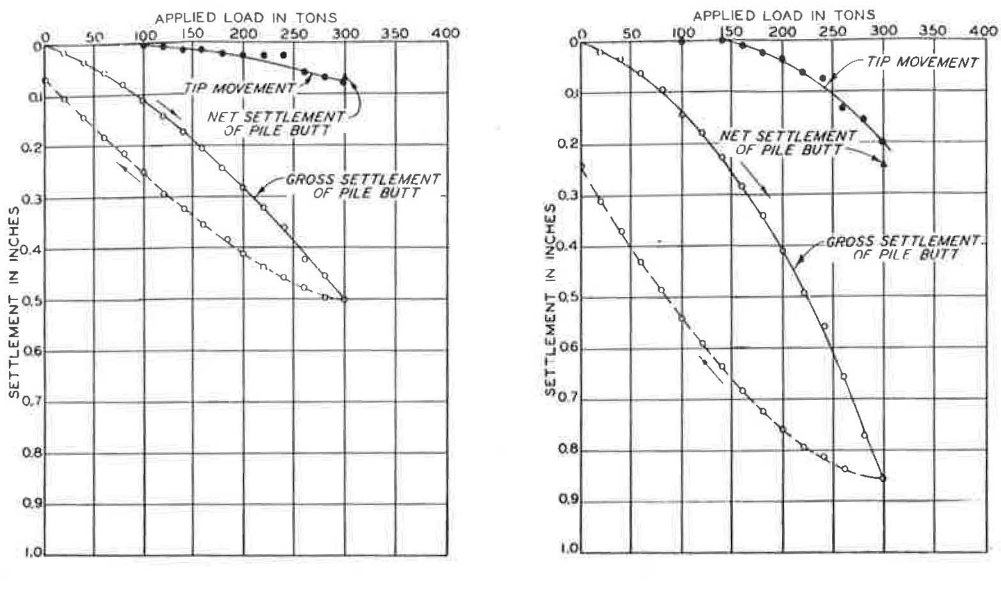


Figure 15. Distribution of load in Pile 3 (15).

to a depth of only 61 ft in an effort to avoid the severe bending that occurred with Pile 3.

Other load tests on bent piles have been performed by Parsons and Wilson (16) and Mohr (17). These tests also show that bent piles have considerable load-carrying capacity.



PILE 2

PILE 3

Figure 16. Load-settlement for Piles 2 and 3 (15).

ALLOWABLE STRESSES

The theoretical basis for determining allowable pile loads considers the pile as a column in an elastic medium. Experience has shown that elastic buckling does not occur for piles that are fully embedded in the ground, provided they are not unusually slender. The only reports of initially straight piles that have experienced elastic buckling are very slender piles in very soft soils. Bjerrum (9), for example, reports 2 train rail piles that buckled during underpinning operations on a church in Norway. The section modulus of these piles was only 8.4 $in.^4$, and the applied stress at failure was more than 40,000 psi.

On the basis of experience the normal practice is to consider piles as short columns. By this is meant a column that is sufficiently stiff to be designed on the basis of the yield stress of the pile material, rather than the elastic buckling load. The danger of inelastic buckling due to bends and other misalignments is only considered in a general way by placing tolerances on the straightness and verticality of piles that can be inspected and in the selection of allowable stresses.

The allowable stress for cast-in-place concrete piles and reinforced concrete piles is taken as 0.225 f_c' in most building codes. One notable exception is the Chicago building code, which uses 0.40 f_c'. Timber piles are not stress-graded, and the allowable stress is most often taken as 60 percent of the basic compressive stress for clear material of like species as set forth in ASTM standards. The more liberal codes, such as the BOCA code (1963) and the St. Louis building code (1961), use 100 percent of the basic compressive stress and design as short columns (18).

For some reason there is even greater variety in the allowable stresses used for steel piles. As shown by data given in Table 1, the allowable stress for a steel H-pile in Boston is only 7,500 psi, while the same pile is permitted a stress of 17,000 psi in many cities and a few building codes permit a stress of 20,000 psi. A survey of 80 governmental agencies by the National Academy of Sciences (19) shows an even greater divergence in the allowable stresses used in practice.

To some extent this disparity in the allowable stresses used in practice reflects differences in local foundation conditions and local experiences, but mainly the differences can be attributed to the lack of knowledge of the structural behavior of piles.

The data given in Table 1 show another curious inconsistency in current practice. In most codes steel H-piles are permitted the same allowable stress as steel pipe piles, despite the fact that pipe piles are inspected after driving to investigate for bending and tip damage.

TABLE 1
ALLOWABLE STRESSES IN STEEL PILES

Building Code	Pipe Piles	H-Piles	Remarks
BOCA (1963), Scranton, and Newark	0.50 F_y	20,000 psi	Designed as short column (BOCA)
St. Louis (1961)	0.50 F_y	17,000 psi	Designed as short column
Detroit, Hartford, Providence, and Camden		17,000 psi	
New York (1968)	0.35 F_y	0.35 F_y	F_y maximum = 36,000 psi 0.125-in. minimum for pipe 0.40-in. minimum for H-piles
National (1967)	0.35 F_y	0.35 F_y	0.1-in. minimum for pipe 0.375-in. minimum for H-piles
Chicago (1963) and Buffalo (1965)	12,000 psi	12,000 psi	
Atlanta, Seattle, Kansas City, Philadelphia, and San Francisco		12,000 psi	
Denver (1962)	9,000 psi	9,000 psi	
Boston (1962)	8,500 psi	7,500 psi	
Baltimore (1955)	7,500 psi	8,000 psi	

Note: Data from Johnson and Kavanagh (18).

CONCLUSIONS

In most building codes the selection of allowable stresses for piles is based on column formulas, the more liberal codes treating piles as short columns. For many situations this approach underestimates the ultimate capacity of initially straight piles because it fails to consider that the soil may provide lateral support that increases the critical buckling load (inelastic) beyond that of a short column. This contention seems to be supported by a small amount of field data that indicate that damaged piles have remarkable load-carrying capacity. It appears that in these cases the soil adjacent to the damaged section of the pile had sufficient strength both to support a portion of the pile load and to provide the lateral support necessary to prevent collapse.

Damaged piling, however, may have reduced load-carrying capacity and stiffness. In addition, there is evidence that driving records and load tests are not reliable indicators of pile damage. Long-term load tests and tests with cyclic loading will sometimes indicate pile damage when compared to load tests on undamaged piles.

The published reports of structural pile failures suggest that there are situations where current practice may be underestimating the actual pile loads. Dragdown loading, for example, is sometimes underestimated or ignored. This can be a particular problem in soft, sensitive clays where remolding due to pile driving can cause dragdown loading to develop with time. Bjerrum reports this phenomenon in the very sensitive Norwegian clays (9).

What is required to put the structural design of piles on a truly rational basis is more good field data, particularly research into the basic causes of pile damage and the load-carrying characteristics of damaged piling. With sufficient data it should be possible to develop a rationale that would permit the use of higher stresses for favorable foundation conditions, with the more conservative values being reserved for difficult situations. This type of approach is presently used in Oslo, Norway, where a stress of 14,200 psi is permitted for steel H-piles less than 40 ft long, 11,400 psi for piles 40 to 100 ft long, and only 9,200 psi for piles over 100 ft long (9).

Situations where higher stresses could be considered include piles that are inspected for damage after driving, piles embedded in firm soils that are capable of providing substantial lateral support, piles that cannot be subjected to dragdown loading, and piles whose support is primarily derived from friction. It also seems that in many cases some benefit could be taken for the load carried by the base reaction of the pile cap.

The selection of allowable pile stresses should also include consideration of the maximum stresses during driving. The use of the wave equation (20, 21) looks encouraging as a method for predicting driving stresses; however, no comparison with field measurements has yet been reported. To accurately predict pile behavior during driving will require an investigation of the effects of dynamic stress on pile materials.

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Pile Load Tests Including Quick-Load Test Method, Conventional Methods, and Interpretations

FRANK M. FULLER, Raymond International, Inc.; and
HORACE E. HOY, Texas Highway Department

This paper covers the general principles of pile load testing, including objectives of pile testing, importance of planning, various types and methods of testing, instrumentation, data to be obtained, and interpretation and use of the test results. Also included are some typical case histories together with correlative data between standard test methods and the constant rate of penetration test method.

•THE PURPOSE of pile load testing can be either to prove the adequacy of the pile-soil system for the proposed pile design load or to develop criteria to be used for the design and installation of the pile foundation. Tests in the first category are generally routine, are carried to twice the proposed working load, and are conducted at the start of the job. Tests to develop design and installation criteria involve more elaborate programs, and piles are usually tested to failure.

Pile load tests are expensive and can be quite time consuming. For small projects the cost of pile testing can represent a considerable portion of the overall foundation cost. In many cases, prior experience combined with adequate subsoil data and sound judgment can preclude the need for pile testing, especially if the pile design load is relatively low.

Routine pile load testing is often the decision of the foundation engineer, but may be required by the general specification or building code having jurisdiction over that type of construction. The decision to embark on an advance test program to develop design criteria is usually made by the owner and the foundation engineer and is based on the scope of the project and the complexities of the foundation conditions. Such test programs can often result in substantial savings in foundation costs, and these can more than offset the investment in the test program.

The prime objective of a test program is to produce data to determine the most economical and suitable pile foundation, including the pile types to be used, the most efficient or highest working load for each type of pile, the required length for each type of pile, and the installation methods necessary to achieve the desired results.

PLANNING THE TEST PROGRAM

Proper planning for any type of pile testing is necessary, but it is absolutely essential for the test program conducted to develop design criteria. Planning starts with a detailed review of the subsoil data in conjunction with the design requirements of the proposed structure. This analysis leads to the following decisions:

1. Final test data to be developed;
2. Type or types of testing to be performed;
3. Extent of the testing that will be required;

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4. Special testing procedures necessary to achieve the desired results;
5. Selection of test locations;
6. Effects of soil conditions on test results, and the need for any additional subsoil data;
7. Selection of the different types of piles to be tested;
8. Determination of approximate pile lengths;
9. Outline of possible installation methods to be used; and
10. Preparation of the technical specifications.

Although thorough planning of a test program is essential, the overall plan must be flexible enough to permit modifications that might be necessary as the driving and testing data are produced.

The technical specifications for the pile test program cover the following points:

1. Prequalification of the pile contractor—This is necessary if the contract for the pile test program is to be awarded on the basis of competitive bidding. However, the complexities and importance of a pile test program require that serious consideration be given to negotiating the contract with a carefully selected contractor.
2. Types of piles to be tested and maximum lengths to be furnished—It should be noted that where a proprietary type of pile is to be included in a test program, any special equipment or material necessary to properly install this type of pile must be made available to the test pile contractor. This problem of proprietary types of piles may be handled by subcontract or separate negotiation with the pile contractor specializing in that piling system.
3. Size and capacity of basic pile-driving equipment to be furnished—This is to eliminate the problem of starting the test program with inadequate pile-driving equipment that might preclude any extension of the test program beyond that originally contemplated or even the proper execution of the original program. Proprietary types of piles may have special equipment requirements.
4. Driving criteria and special installation methods that may be required.
5. Types of tests and maximum testing capacity to be furnished—This will permit the contractor to properly plan for the necessary equipment and to build into the test program some degree of flexibility.
6. Required testing equipment and instrumentation including calibration.
7. Testing procedures to be followed.
8. Data to be recorded and reported.
9. Payment method and schedule of bid items—The flexibility mentioned earlier should be reflected in the pricing and payment method for the test work, for example, lump sum for mobilization and demobilization, unit price for materials, and hourly rates for various types of operations such as pile driving, moving, testing, or standing by.

TEST TYPES AND METHODS

Pile load testing usually involves the application of a direct axial load to a single vertical pile. However, load testing can involve uplift or axial tension tests; lateral tests applied either horizontally or perpendicular to the pile axis (e.g., if battered); group tests; combined axial and lateral tests; any of these tests applied to batter piles; or any of these tests applied to pile groups consisting of vertical piles, batter piles, or a combination of such.

Pile load testing also generally involves the application of a static load to the pile. However, other methods of load application have been used, such as dynamic, vibratory, and explosive. Neither dynamic nor explosive testing is too reliable, and these methods are infrequently used. Vibratory testing is only used where structure loading conditions warrant. The test load, whether it be for a bearing test, uplift test, or lateral test, is usually applied statically by a force acting directly on the pile either by direct weight or by hydraulic jacks in combination with some type of reaction system.

Where the test load is applied directly to the pile by means of a loaded platform (Fig. 1), the load must be capable of being applied and removed in increments of known weight. The test beam and platform are considered part of the test load and included in the first increment of loading.

Figure 1. Test load applied directly to pile by using loaded platform and water-filled interconnected steel tanks.

The load and platform must be kept balanced at all times. Usually timber cribs are placed under the platform edges to prevent tipping of the load in case the platform becomes unstable. Wedges between the timber crib and platform edge are tightened only while the load is being added or removed. These wedges must be kept loose as the pile settles under the direct load.

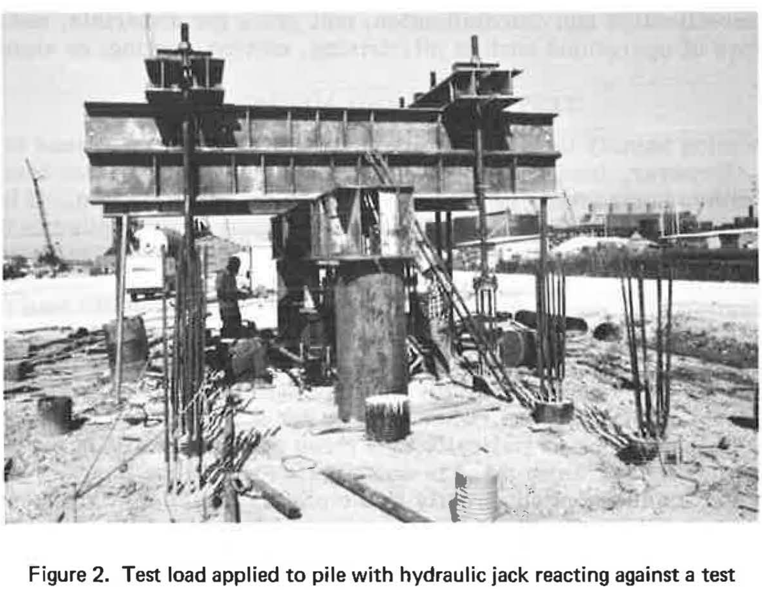

Figure 2. Test load applied to pile with hydraulic jack reacting against a test frame and anchor piles.

Most test loads are applied with hydraulic jacks reacting against either a stable loaded platform or a test frame anchored to reaction piles (Fig. 2); there may also be some other type of reaction. The use of hydraulic jacks has several advantages. For example, it is the only practical way to apply load-unload-reload cycles, and hydraulic jacks are more suitable for uplift tests, lateral tests, and tests on batter piles.

Regardless of the method of load application, the load should be kept constant under increasing pile deflection. For direct loading this presents no problem. When hydraulic jacks are used this can be accomplished by activating the jack pump with a compressed gas control system. Precautions should be taken to avoid eccentric loading by carefully centering test beams or jacks and maintaining a balanced load.

Anchor piles or the supports for a reaction load must be placed a sufficient distance from the test pile to avoid influencing its performance. This minimum distance will depend on such things as the magnitude of load to be applied and the subsoil conditions. Such influence could reflect a greater or lesser ultimate bearing capacity than actual.

It is recommended that, during a lateral load test, an axial compressive load equal to the minimum design dead load, be applied to the pile. This type of combined test loading would give a more accurate indication of the actual lateral load capacity of the pile under service conditions. When a vertical load is applied during a lateral load test, the pile butt should not be restrained from lateral movement. This can be accomplished by using a system of rollers between the vertical load and the pile. The point of application of the horizontal load should, if possible, simulate in-service conditions.

INSTRUMENTATION

The basic information to be developed from the pile load test is usually the deflection of the pile butt under the test load. Probably the fundamental method of measuring the pile butt movement is by reading a target rod (or scale fixed to the pile) with an engineer's transit referenced to a fixed bench mark. In most cases, the degree of accuracy obtained with this type of instrumentation is sufficient. Quite often, measurements with the level and rod (or scale) are used as a secondary or backup system to check other measuring systems.

Direct readings of the pile butt movement (either vertically or horizontally) can be made by using the mirror, scale, and wire method. A measuring scale is fixed to a mirror, which in turn is attached directly to the pile or the test plate. A taut wire passing in front of the scale permits direct readings of pile movement. Consistent scale readings are obtained by aligning the wire and its image in the mirror. The wire can be kept taut by a weight and pulley system or by springs.

The most common method for measuring the pile movement is with dial extensometers mounted on an independent support system, and with gage stems bearing against the top of the test plate or on angle irons attached to the sides of the pile (Fig. 3). At least 2 dial gages mounted on opposite sides of the pile should be used to compensate for possible tilting or lateral movement of the pile under load. Sometimes a gage sensitivity of 0.001 in. is specified, but usually gages reading to 0.01 in. have sufficient accuracy to meet the normal settlement criteria. With ultra-sensitive dial gages, it is often impossible to satisfy some of the specification requirements such as "until settlement stops."

When the instrumentation for a compression test is set up, it is often advisable to mount dial gages to measure lateral movements of the pile under test. Such movement could be due to eccentric loading and contribute to the apparent vertical movement of the pile butt.

The instrumentation system must be supported independently from the loading system with supports protected from extreme temperature variations, effects of the test load, and accidental disturbance by test personnel. It is advisable to have a secondary or backup instrumentation system in case of an accidental disturbance of the primary system or the necessity to reset dial gages so that continuity of data is maintained.

Data on load distribution and the elastic behavior of the pile can be obtained with displacement (or so-called "strain") rods or strain gages. This type of instrumentation can be installed in almost all types of conventional piling but more readily in cast-in-place

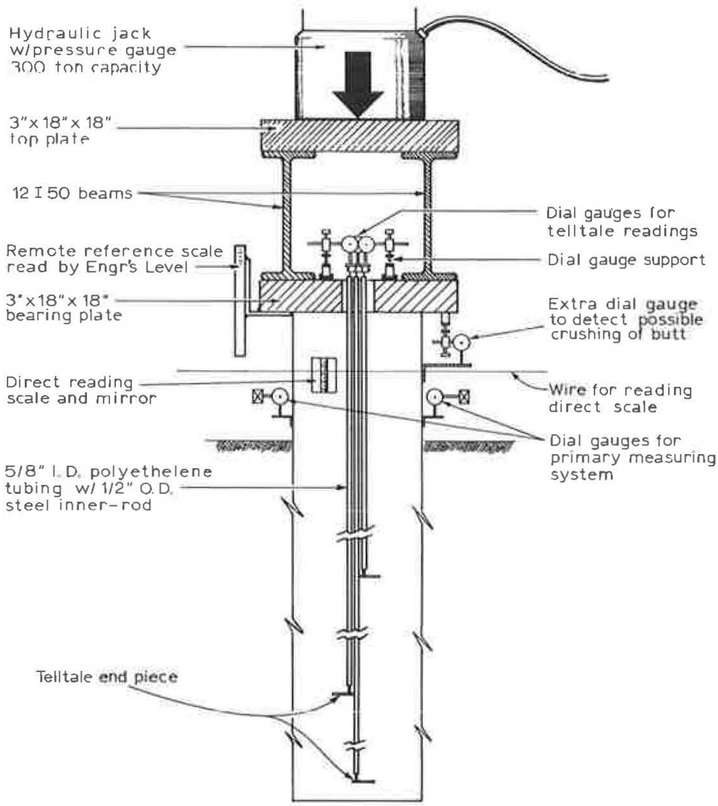

Figure 3. Instrumentation of test pile.

concrete piles. Strain gages or the terminal points of "strain" rods can be located at various positions along the pile.

In general, strain rods are less complicated, are less subject to malfunction, are more easily handled by field personnel, and produce direct elastic shortening data over a long gage length between the terminal point and the pile butt. The proper installation of strain gages, so as to avoid malfunction and produce reliable data, is an extremely sensitive operation.

The installation of strain rods or gages results in a physical change in the cross section of the pile and thus its elastic properties. Although data at frequent intervals along the pile shaft are desirable, it is sometimes advisable to sacrifice some data in the interest of practicality. Often a single strain rod to the pile tip is sufficient to provide the essential information on the elastic behavior of the pile and the basic load distribution.

TEST PROCEDURES

Most routine tests are carried to one and one-half or twice the proposed design load for a single pile, or one and one-half times the design load for a pile group. Carrying the test load any higher merely wastes job time and money. Rarely can such additional data be used advantageously, such as for redesign, without seriously affecting the job schedule.

Most test programs that are specifically executed to produce design data should include testing piles to failure in order to develop the most efficient design. However, this is not always essential, and definite design decisions can be reached if sufficient routine testing is done on piles of different types, sizes, shapes, and lengths.

The time interval between pile driving and testing depends on the type of pile and subsoil conditions. For example, sufficient time should be permitted for the proper curing of cast-in-place concrete piles before they are tested. Where test piles are driven into cohesive soils, it is advisable to wait several days for the soil to regain its shear strength, which in all probability was reduced because of the remolding effects of pile driving.

The test load can be applied in various increments and time intervals. In general, the load should be applied over an extended period of time, with increments equal to about 25 percent of the proposed or assumed design load. However, in the interest of saving time, the increments can be larger during the early stages of the test and, in the interest of obtaining accuracy, they should be smaller as the total load is increased.

A normal time interval between load increments is from 1 to 2 hours. Frequently, specifications will require that the load be held until the rate of settlement is less than some fixed value such as 0.01 in. per hour, but in most cases a maximum time interval between increments will also be specified. Providing that the pile-soil system has not failed, the full test load should be held for some period of time, such as 24 or 48 hours. Specifications will often establish a maximum rate of settlement under full load that cannot be exceeded over a certain period of time in order for the test to be considered satisfactory. Where specifications use language such as "until settlement has stopped," the impracticality of using highly sensitive dial gages is obvious.

Instrumentation readings should be taken before and after each increment of load and at sufficient intermediate intervals in order to define the load-time-deflection curves. When piles are not tested to failure, and after the full test load has been applied, readings are taken at least every 30 minutes for the first 12 hours and every hour thereafter. During removal of the test load, readings should also be taken before and after each load decrement, and a final rebound reading should be taken about 12 hours after the full load has been removed.

Among the several special testing techniques available are cycle loading and the constant rate of penetration (CRP) method. When piles are tested to establish the design load, cycle loading can help determine more accurately the load that satisfies the allowable deflection criteria. Also, cycle loading can provide some indication as to the distribution of load between friction and end-bearing. Van Weele (1) has suggested a method by which a plot of the elastic recovery at each unloading cycle versus load applied at that cycle is used to separate friction from point-bearing. The curve usually becomes a straight line soon after the early load increments (Fig. 4). The distance between the plotted curve and a line drawn through the origin and parallel to the straight part of the curve represents the portion of the load carried by friction. At best, this is only an approximation.

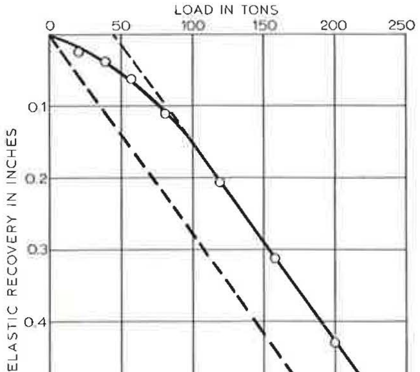

Figure 4. Approximate distribution of test load between point-bearing and friction (1).

Cycle loading should not be mandatory for routine testing because it could add unnecessary expense without contributing significant additional data. Such special procedures should be included at the engineer's option.

The constant rate of penetration method was first experimented with in 1957 by Whitaker but did not receive wide publicity until after 1961 (2). Under favorable conditions, this method has shown reasonably good correlation with standard test methods. For the CRP test, a force or load of sufficient magnitude is applied to the pile to maintain a constant penetration rate into the ground. This means that the applied load might have to be adjusted as the test proceeds. In general, the recommended penetration rate is about 0.03 in. per minute for cohesive soils and about 0.06 in. per minute for granular soils. However, the penetration rate could vary over a rather wide range and still produce satisfactory results.

The CRP method is applicable to friction types of piles, and sufficient testing capacity of the pile-soil system.

Special testing procedures can be used to produce specific data. For example, the distribution of applied load between friction and point-bearing can be approximated by driving and testing piles of different lengths. Some would be driven just short of the end-bearing stratum, while others would be driven to full embedment. An uplift test might also produce approximate data on the amount of load carried by friction.

Another special test procedure would be the casing off of that portion of the test pile that extends through soils offering temporary support so as to determine the capacity of the pile-soil system within the permanent bearing strata.

INTERPRETATION OF RESULTS

The basic purpose of the pile test is to determine or verify the safe working load for the pile-soil system. In most cases, tests are not carried to failure, and some arbitrary criterion is applied to determine if the test results are satisfactory. Some of these criteria are rather vague, such as "where the settlement is disproportionate to the load" or "where the load-settlement curve breaks." Others are based on a maximum allowable gross or net settlement that can either be a fixed number, such as 1 in. or related to the amount of test load applied, such as 0.01 in. per ton.

When definite failure does not occur, such as plunging of the pile into the ground, some arbitrary definition of "failure" must be used. Such criteria should be realistic—neither too conservative nor too liberal. The important factors to be considered are the permissible differential settlement under the design load and safety.

Settlement usually governs and requires consideration of the elastic shortening of the pile under the design load. Assuming that all the piles are of the same material, of approximately equal length, and driven into substantially similar soils, the elastic shortening will be approximately equal for all piles and thus will not contribute to differential settlement.

Many methods have been suggested for determining the safe allowable pile load or for defining the "failure" of the pile-soil system. The application of these various criteria can produce a wide range of "safe" pile loads from the same test data.

Unless failure actually occurs, it would appear reasonable to define the point of "failure" by a maximum slope of the load-settlement curve. For example, the failure load could be defined as the load that results in a slope greater than 0.05 in. per ton on the gross load-settlement curve or a slope greater than 0.03 in. per ton on the plastic load-settlement curve, whichever is smaller (Fig. 5). This is still an arbitrary definition of failure, but is a more generalized approach. The total criterion would include a maximum allowable gross settlement under the design load, with consideration given to elastic shortening of the pile and to safety.

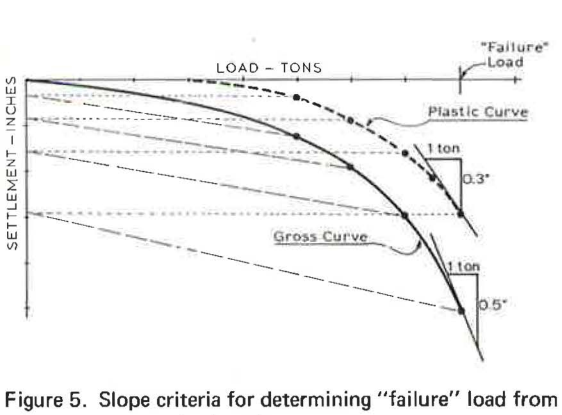

Figure 5. Slope criteria for determining "failure" load from load-settlement curves.

Where failure results from some arbitrary criterion, the factor of safety could range from one and one-half to two. Where actual failure of the pile-soil system is determined by a plunging of the pile into the ground, this factor of safety could range from two to two and one-half.

The complete analysis of the test results should include consideration of all factors, such as the elastic behavior of the pile (from instrumentation or cycle loading) and an evaluation of the long-term performance. This could involve an analysis and evaluation of the subsoil data in conjunction with the test results.

It should be noted that observed settlements made at the top of the pile may not necessarily indicate downward movement of the pile into the ground. Where high load tests are performed, the possibility of local failure of the pile above ground surface, or crushing of the grout under the test plate, should be recognized as possible factors contributing toward observed "settlements."

APPLICATION OF RESULTS

Because it is impractical to test every pile on a project, the results of the testing must be applicable to other piles to be driven. This is a reasonable and accepted procedure, providing that the following conditions exist:

1. The other piles are of the same type, material, and size as the test piles;
2. Subsoil conditions are comparable to those at the test pile locations;
3. Installation methods and equipment used are the same as or comparable to those used for the test piles; and
4. Piles are driven to the same penetration depth or resistance or both as the test piles to compensate for variations in the vertical position and density of the bearing strata.

The results of tests on single piles can usually be applied to pile groups, especially in granular soils. The group effect, if any, depends a great deal on the subsoil profile to some depth below the pile tips. Unless the bearing stratum is relatively thin and underlain by deep deposits of soft compressible soils, there should be no detrimental effects from group loading. However, where the piles receive their principal support in cohesive soil, group action should be analyzed.

The application of the results of the advance test program to the foundation design and specification can often produce substantial savings in foundation costs. Although, as a practical measure, the test results would lead to the selection of a single design load, the requirements for various types of piles as to size, length, shape, weight per foot (stiffness), installation methods, and driving requirements could vary over a rather wide range. These differences should be reflected in the specifications and, in turn, will be reflected in the alternative costs to produce the most economical foundation for the conditions involved.

LOAD TESTING BY THE TEXAS HIGHWAY DEPARTMENT

In 1963, correlative pile load test field studies were initiated by the Texas Highway Department between the standard 48-24 hour test method (3) versus a quick test method (4) that was modified after the constant rate of penetration method described by Whitaker and Cooke (2).

Purpose of Pile Load Test as Used by the Texas Highway Department

Design values, construction procedures, and anticipated performance of a piling or drilled shaft foundation should be substantiated by load tests in certain cases.

Load testing of piling is especially recommended when it has been established by soil studies that static resistance (design load as indicated on the plans) will be obtained at specified plan tip elevation but dynamic resistance by hammer formula will not be reached. If it is apparent that considerable savings may be attained by load testing, this procedure should be used and specified on the plans.

For design purposes, a static loading test is performed for the following reasons:

1. To prove the piling adequate for the proposed design load at the selected pile tip elevation; and
2. To determine the true relationship between static and dynamic capacity of the piling in a particular soil condition and thereby to obtain a K-factor that is applied to the dynamic formula. The K-factor is determined by the following formula:

$$K = \frac{L}{P}$$

where

L = maximum proven design load as indicated by pile load test, and
P = "safe" load capacity determined by the dynamic hammer formula used on the test-loaded piling before the K-factor is applied.

The specification hammer formula is then modified, to conform to the maximum proven design load as determined by the load test. For the Engineering-News formula, this modification would be

$$P = K\frac{2WH}{s + 0.1}$$

Standard 48-24 Hour Test Method

Prior to January 1963, the Texas Highway Department (THD) used the basic AASHO 48-24 hour test method as modified by the THD specifications (3). This method consists of first loading the pile to approximately the design load with successive load increments (in multiples of 5 tons) equal to about one-third the design load (Fig. 6). Gross settlement readings, loads, and other data are recorded immediately before and after the application of each increment of load and at 15-minute intervals between load application. Load increments are not added until 2 hours have elapsed without measurable settlement, which is considered to be 0.005 in. or more.

If the estimated net settlement exceeds 0.25 in. before the application of twice the proposed design load, the pile is unloaded and the rebound recorded. If the actual net settlement is more than 0.25 in., the pile is driven to a greater resistance.

When the estimated net settlement (gross settlement minus estimated elastic rebound) has reached a value of 0.25 in. (Fig. 7), or when a minimum of two times the design load is on the pile and an estimated net settlement does not exceed 0.25 in. the addition of load is discontinued and a standard AASHO 48-24 hour test is run.

The 48-24 hour test consists of holding the load on the pile constant for a minimum of 48 hours and for 24 hours of no measurable settlement. Readings for gross settlement are made every 15 minutes during this period. If the gross settlement at the end of a successful 48-24 hour test is less than 0.3 in., additional load is applied until the estimated net settlement is about 0.25 in., and the standard 48-24 hour test is run again. At the conclusion of the standard 48-24 hour test, all load is removed and rebound readings are taken every 15 minutes for 4 hours. If the recorded net settlement is less than 0.25 in., the pile is reloaded to twice the proposed design load, and this load is held for 4 hours without measurable settlement. Following this the load is increased until the anticipated net settlement is 0.25 in., and the standard 48-24 hour test is repeated. Such testing is continued until the actual net settlement equals or exceeds 0.25 in. or until the testing capacity is reached.

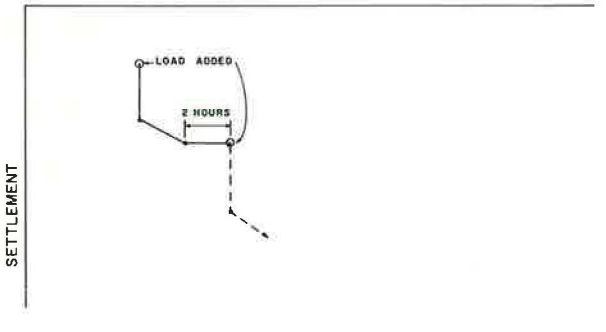

Figure 6. Load-settlement-time relationship for one load increment (48-24 hour test method).

The theoretical "failure" load is considered to be the load that results in a net settlement (gross minus rebound) of 0.25 in. The maximum proven design load is interpreted to be 50 percent of the load that, after a minimum of 48 hours, causes a permanent net settlement of not more than 0.25 in., measured at the top of the pile. If the test load causes a permanent net settlement of more than 0.25 in., then the allowable design load is 50 percent of the load obtained by interpolation from the computed net settlement line value of 0.25 in. This line is obtained by calculation based on the actual recorded recovery.

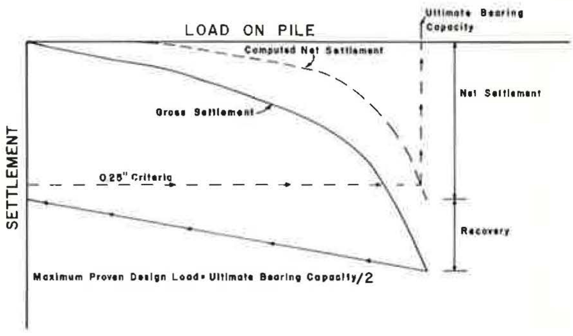

Figure 7. Interpretation of results (48-24 hour test method).

Constant Rate of Penetration Method

In January 1963, Engineering News-Record published an article by Esrig (5) in which he discussed a new pile-testing procedure developed by Whitaker and Cooke (2). Whitaker termed the new testing procedure the constant rate of penetration (CRP) test. This method requires that the test-loaded pile be forced into the ground at a constant rate with the loads corresponding to specific penetrations being measured.

Quick Test Method

After publication of Esrig's article, plans were immediately made to perform load tests not only in accordance with the Texas Highway Department standard specifications but also by a modification of the constant rate of penetration (CRP) test.

The CRP test calls for records of time and jacking force to be made at equal intervals of movements of the pile head with the rate of jacking being adjusted so that readings occur at equal intervals of time. For convenience and simplicity, the CRP test was modified by the Texas Highway Department to produce the quick test method. Essentially, it requires that loads be added in increments of 5 or 10 tons with gross settlement readings, loads, and other data recorded immediately before and after the application of each increment of load. Each increment is held for 2½ minutes, and the next increment is then applied.

When the load-settlement curve obtained from these test data (Fig. 8) shows that the pile is definitely being failed (i.e., the load on the pile can be held only by constant pumping of the hydraulic jack and the pile is being driven into the ground), pumping is stopped. Gross settlement readings, loads, and other data are recorded immediately after pumping has ceased and again after intervals of 2½ minutes and 5 minutes. The load on the pile in the case of constant pumping is called plunging failure load. Then all load is removed, and the pile is allowed to recover. Net settlement readings are made immediately after all load has been removed and at intervals of 2½ minutes for a total period of 5 minutes.

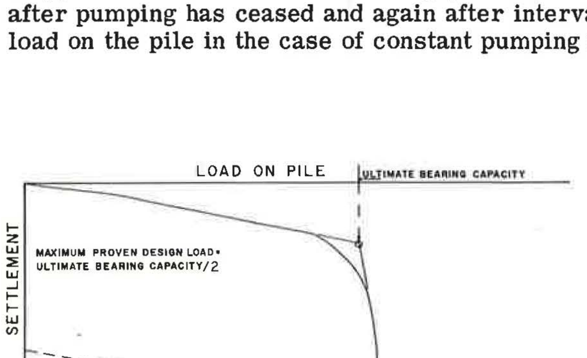

Figure 8. Interpretation of results (Texas Highway Department quick test method).

All test loads are carried to plunging failure or to the capacity of the equipment. The maximum proven design load is considered to be 50 percent of the ultimate bearing capacity, which is indicated by the intersection of lines drawn tangent to the 2 basic portions of the load-settlement curve as shown in Figure 8.

For this method of interpretation, the scale to be used for plotting the load-settlement curve should be 1 in. for each 10-ton load increment and

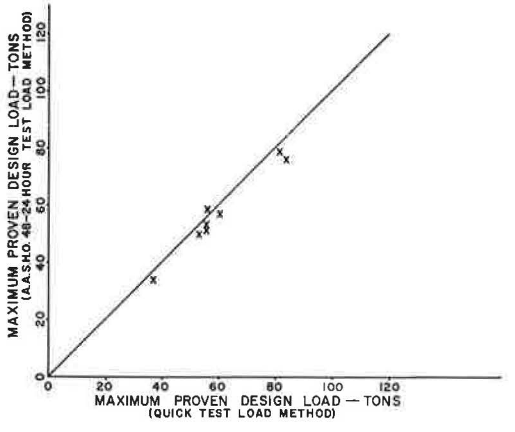

Figure 9. Correlation of proven design load between standard 48-24 hour and quick test methods.

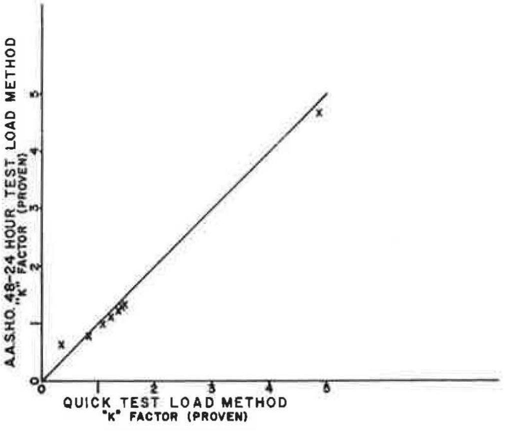

Figure 10. Correlation of proven K-factor between standard 48-24 hour and quick test methods.

0.10-in. settlement respectively. The maximum proven design load is the value used to establish a K-factor for use with the dynamic hammer formula as stated earlier.

Correlation Studies

From January 1963 until March 1965, 11 pile load tests were performed by the Texas Highway Department by using both the standard 48-24 hour and the quick test methods. Out of this number of tests, eight were test-loaded to theoretical failure by the 48-24 hour test method. All of the tests were taken to plunging failure with the quick test method. A summary of the data for these tests is given in Tables 1, 2, 3, and 4. The maximum proven design load obtained by the quick test method and the 48-24 hour test method are shown in Figure 9. The average deviation of maximum proven design load values obtained from the quick test method versus the standard 48-24 hour test method was about 4 percent.

The K-factors given in Tables 3 and 4 for both load test methods are shown in Figure 10. Agreement is considered to be very good in all ranges of value.

TABLE 1

LOCATION OF PILES TESTED FROM FEBRUARY 1963 TO NOVEMBER 1964

Test Number	County	Structure	Bent and Test Pile
1	Brazoria	Chocolate Bayou	Bent 6, C
2	Wharton	Colorado River	Bent 4
3	Arkansas	Copano Bay	Pile 1
4	Victoria	MP RR OP	Pile 1
5	Harris	Ramp F	Bent 9
6	Jefferson	US-69 and FM-365	Pile 1
7	Harris	Pierce	Bent 94
8	Harris	183	Bent 3
9	Harris	SP RR UP	Bent 2
10	Galveston	GC and SR RR UP	Bent 24
11	Harris	SP RR UP	Bent 2

CONCLUSIONS

Based on the results of these tests, the Texas Highway Department began using the quick test method as the standard testing method in April 1965. A special provision to Item 405, Test Loading Piling, was prepared for this test method and has been in use by the department since that time (4).

From the 1963-1965 study as well as from the department's experience to date, the following observations and conclusions have been reached relative to the quick test method:

1. A pile load test can be expeditiously performed in about 1 hour with resultant savings in money and time;
2. Construction delay to the project caused by load testing is greatly reduced;

TABLE 2

DESCRIPTION OF PILES, SOIL, AND HAMMER

Test Number	Pile Type	Total Pile Length (ft - in.)		Effective Pile Length (ft - in.)		General Soil Type	Pile Design Load (tons)	Type of Hammer	ENF Bearing Value (tons)	Final Penetration (in./blow)
1	16 in. sq PC/PS	40	0	36	0	Clay, sand, silty	47	Link-Belt 520	40.2	0.429
2	12BP53	51	0	49	0	Sand, clay	46	McK-T DE-30	42.8	0.260
3	18 in. sq PC/PS	103	0	83	6	Sand, clay	40	Vulcan 014 Vulcan 014	83.7	0.385
4	12BP53	32	4	25	0	Sand, clay	36.4	Vulcan 1	15.0	0.90
5	14 in. sq PC	28	8	21	0	Clay, silty, sandy	53	Vulcan 1	65.2	0.130
6	16 in. pipe	63	8	60	0	—	44	Delmag D-12	25.3	0.444
7	14 by 11 in. step-taper	31	0	31	0	Silt, clay	60	Raymond 1-S	39.0	0.400
8	12BP53	29	0	24	0	Clay, sand, silty	60	Delmag D-12	79.8	0.150
9	12BP53	32	0	31	0	Clay, sand, silty	60	Delmag D-12	97.1	—
10	16 by 11 in. step-taper	47	10	44	0	Clay, silty, sand	31	Raymond 1	11.22	1.20
11	12BP53	21	1	21	0	Sand, clay, silty	52	Delmag D-12	63.7	0.170

TABLE 3

AASHO 48-24 HOUR TEST METHOD

Test Number	Duration of Test (hr)	Maximum Load on Pile (tons)	Maximum Load Held 48 Hours (tons)	Gross Settlement (in.)	Net Settlement (in.)	Proven Design Load (tons)	K-Factor
1	102.25	110.0	105.0	0.313	0.251	52.5 [a]	1.31
2	55.67	110	110	0.420	0.167	55 [b]	1.28
3	57	95	90	0.156	0.049	45 [b]	0.54
4	140.25	80	80	0.324	0.161	40 [b]	2.67
5	258	155	155	0.379	0.259	77 [a]	1.18
6	83.25	75	75	0.412	0.349	35 [a]	1.38
7	114.5	120	115	0.562	0.447	56.9 [a]	1.46
8	132	160	160	0.501	0.281	79 [a]	0.99
9	192	115	115	0.496	0.362	57.52[a]	0.59
10	140.25	115	110	0.566	0.448	52.5 [a]	4.67
11	111	105	105	0.390	0.262	50 [a]	0.784

Note: Piles loaded by hydraulic jack and reaction beam supported by anchor piling. Settlement was obtained by extensometers.

[a]In those cases where the standard 48-24 hour test load caused a permanent net settlement of more than 0.25 in. and other criteria were met, then the maximum proven design load is taken to be 50 percent of that load obtained by interpolation from the computed net settlement line value of 0.25 in. This line was obtained by calculations based on actual recorded recovery.

[b]Not failed.

TABLE 4

QUICK TEST METHOD

Test Number	Duration of Test (min)	Plunging Failure Load (tons)	Ultimate Bearing Capacity (tons)	Gross Settlement (in.)	Net Settlement (in.)	Proven Design Load (tons)	K-Factor
1	65	120	109	0.185	0.087	54.5	1.36
2	45	145	125	1.151	—	62.5	1.46
3	55	180	150	0.651	0.379	75	0.89
4	65	140	96	0.818	—	48	3.2
5	—	190	166	0.397	0.256	83	1.27
6	50	85	74	0.666	0.596	37	1.46
7	—	134	121.5	0.476	0.356	60.7	1.56
8	75	170	162	0.597	0.371	81	1.01
9	—	120	113.5	0.301	0.168	56.75	0.58
10	105	120	109	0.403	0.284	54.5	4.86
11	67	115	103	0.337	0.226	51.5	0.81

3. Substantial decrease in bid price of load test setup ensures feasibility of testing on small projects;

4. Simplicity of the testing procedure ensures standardization of the test and easy interpretation and utilization of results without reliance on arbitrary definitions; and

5. Load-settlement curves can be easily duplicated by repeated tests.

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Bearing Capacity of Foundation Piles: State of the Art

HARRY M. COYLE and IBRAHIM H. SULAIMAN,
Texas Transportation Institute, Texas A&M University

The purpose of this paper is to present the state of the art concerning bearing capacity of foundation piles. Topics presented include (a) soil property changes due to pile driving in cohesionless and cohesive soils and (b) determination of bearing capacity for single piles and pile groups in cohesionless and cohesive soils. The methods presented for design of single piles include the static formula, the dynamic formula, and a recently developed numerical method. The numerical method has been used to predict immediate pile movement under static load as well as ultimate pile load. The methods for design of pile groups that are presented are empirical in nature and include design against bearing capacity failure and design against excessive pile and soil settlement. Because this is a state-of-the-art presentation, no definite conclusions are given concerning the use of one design method in preference to another. It is shown that no single method has been adopted for general use by designers at this time except for field load tests, which are generally used on large construction projects involving pile foundations.

• PILE FOUNDATIONS are used when the soil near the surface is not able to support foundation loads because of either low bearing capacity or the possibility of excessive settlement. The primary function of a pile is to transfer foundation loads to deeper soil strata that are stronger and less compressible. Even though in practice piles are generally used in groups, most of the published research in the United States and abroad, as surveyed by Kezdi (24), has dealt with single piles. Designers rely on the bearing capacity of single piles to forecast the bearing capacity of pile groups.

The bearing capacity of single piles is presently being determined by one or more of the following methods: static formulas, dynamic formulas, and field load tests. The static formula method relates soil shear strength, as determined from laboratory or in situ tests, to skin friction along the pile shaft and to end-bearing below the pile point. The 2 components, friction and end-bearing, are combined to estimate pile-bearing capacity for any selection of pile diameter and length. Depending on the designer's experience in his local area, the pile chosen may or may not constitute a final design. The dynamic formula method relates the resistance to penetration during driving to the static bearing capacity. In many areas designers have pile-driving records from numerous projects, and may use a dynamic formula for initial design. Bearing capacities as determined by the static or dynamic formula method may be considered conservative, and considerations of economy may favor the use of a field load test. This is especially true on large projects where it is desirous to obtain a more exact value of bearing capacity at one particular site. After the capacity is determined for a single pile, an adjustment for group action, coupled with an estimate for group settlement, would lead to a more or less final foundation design.

Paper sponsored by Committee on Foundations of Bridges and Other Structures and presented at the 49th Annual Meeting.

SOIL PROPERTY CHANGES DUE TO PILE DRIVING

When piles are driven into the ground, changes occur in the in situ geotechnical properties of the soil. The soil experiences considerable displacement. The resulting stress distribution in the soil surrounding the pile is far different from that which existed prior to driving. For a given pile shape, the mode of disturbance depends on the type of soil.

Cohesionless Soils

Upon cessation of driving in cohesionless soils, the relative density is increased within the limits of 7 to 12 pile diameters around the pile shaft, and 3 to 5 pile diameters below the point. These limits, as shown in Figure 1, have been established by Broms (5), and are based on the works of Meyerhof (33), Weele (61), Kishida (37), Kerisel (22), and Plantema and Nolet (42). The extent of the zones indicated has been observed primarily by use of probes or penetrometers and is influenced by soil density, hammer input energy, and pile type. Vesic (57) has shown that the increase in relative density is larger for loose material than for dense material. Robinsky and Morrison (46) have shown that the limit of the compacted zone is larger around tapered piles than around piles of constant cross sections.

When a pile comes to rest in a cohesionless soil, there are residual stresses acting on the pile shaft. These stresses have been observed and reported in the literature by Mansur et al. (29) and Hunter and Davisson (20). Residual stresses are negligible in jetted piles and in piles driven with vibratory hammers, but they may be significant in the lower portion of piles driven with impact hammers. The distribution of load between the shaft and the point is influenced by residual stresses.

Meyerhof (34), Fleming (16), and Pepper (41) report overlapping of compacted zones for pile groups in cohesionless soils. Also, the soil surrounding the center piles is compacted more than the soil near the periphery of the group.

Cohesive Soils

When a pile is driven into a cohesive soil, the material is compressed and remolded. According to Broms (5), this remolded zone extends from 1 to 3 diameters laterally and about 1 pile diameter below the pile point as shown in Figure 1. The stiffer the soil, the larger is the extent of the compressed zone.

Excess pore water pressures are induced in the cohesive soil surrounding a pile after it is driven. In soft sensitive clays that are normally consolidated, measurements by Reese and Seed (45) and by Bjerrum and Johannssen (3) indicate pore pressures,

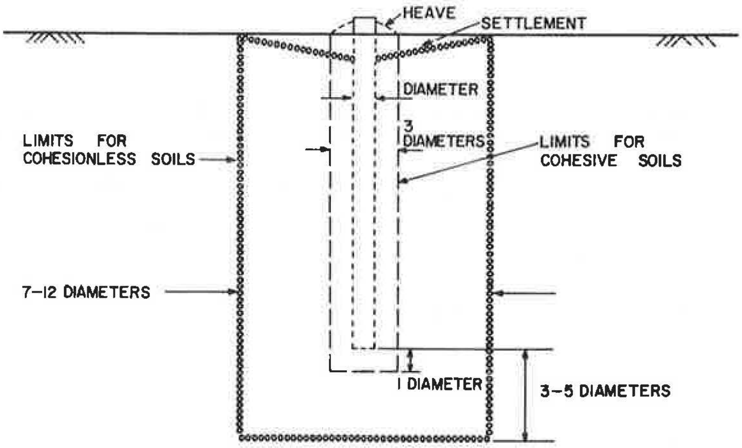

Figure 1. Zones of compaction and remolding due to pile driving (5).

which occur immediately after driving, that increase almost linearly with depth and approach the total overburden pressure. Similar findings were obtained for stiff preconsolidated clay by Airhart et al. (1). The increase in pore water pressure is not restricted to the zones shown in Figure 1 but has been detected by Reese and Seed (45) up to 15 pile diameters away from the pile.

Pore pressures induced by pile driving in the vicinity of natural slopes and within embankment sections cause a temporary reduction in shear strength that can lead to rather sudden slope failures. This situation has been reported by Bjerrum and Johannssen (3). Methods for predicting induced pore water pressure have not been very reliable. Therefore, if pore pressures are to be considered in design, field measurements are recommended. For detailed information on field instrumentation, the reader is referred to publications by Bjerrum and Johannssen (3) and Hanna (19).

The dissipation of induced pore water pressure and the consolidation of the soil are accompanied by movement of water away from the pile-soil interface. The rate of movement is governed by the permeability of the clay, the thickness of the layer, the pile spacing, and the pile material. The decrease in moisture content at the pile-soil interface causes an increase in the shear strength in the soil at the interface. In soft, normally consolidated clay, Reese and Seed (45) reported that pore pressures at the pile-soil interface dissipated about 24 days after driving. The final strength of the clay, as measured by the unconfined compression test, was approximately 1.5 times the in situ shear strength. For sensitive clays, Meyerhof (32) and Fellenius (15) reported that 90 percent of the undisturbed unconfined compressive strength of the clay was regained in 30 to 50 days.

The formation of a gap between the pile and the clay due to transverse vibrations during driving has been reported by Glanville et al. (17), Faber (14), Reese and Seed (45), and Tomlinson (55). The gap may occur in association with a soil heave at the ground surface. The gap should close in soft clays in a relatively short period of time because of consolidation. However, in stiff clays it may remain open for an extended period of time. When piles are driven in groups, the closing of gaps near the center of the group is expected, especially for a pile spacing of 2.5 to 3 pile diameters.

Tapered piles tend to compact the material surrounding the shaft during driving, similar to the action observed in sands. The fact that H-piles displace a smaller volume of soil during driving has been used by Casagrande (8), among others.

The preceding discussion concerning soil property changes due to pile driving is very brief and is qualitative in nature. There is a definite need for more quantitative data in the form of full-scale field measurements, especially in the area of the action of pile groups.

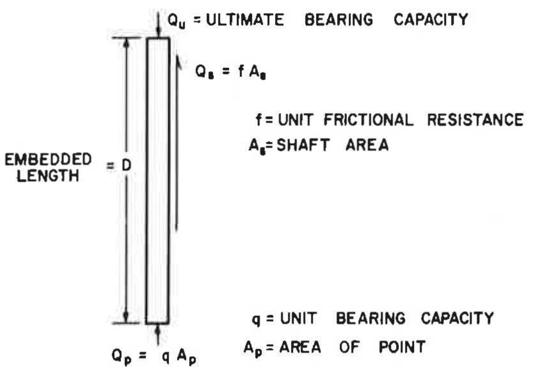

Figure 2. Ultimate bearing capacity, static formula method.

BEARING CAPACITY OF SINGLE PILES BY THE STATIC FORMULA METHOD

Two recent publications, one by McClelland et al. (30) and the other by Broms (5), give a good summary of the procedure presently used to determine bearing capacity of a single pile by the static formula method. The simplified concept is shown in Figure 2. The familiar static bearing capacity equation is as follows:

$$Q_u = Q_s + Q_p = fA_s + qA_p$$

Q_s and Q_p are the shaft and point resistance respectively, and the other terms are defined in Figure 2. Both types of resistance are determined from measured or estimated shear strength of the soil. Static and dynamic penetrometers may be used to measure the 2 components of resistance directly. The penetrometer may be a cone similar to the Dutch cone, which is commonly used in Europe and which has been reported on by Begemann (2), Kanty (21), and Schmertmann (49).

Design factors for cohesionless soils are as follows:

Friction

$f = KP_o \tan \delta$;
K = earth;
P_o = average effective overburden pressure = γD;
γ = effective unit weight;
D = pile embedded length; and
δ = angle of shaft friction.

End-bearing

$q = P_o N_q'$; and
N_q' = bearing capacity factor for deep circular base (Fig. 3).

Earth pressure coefficient K

Impact-driven piles
Steel and concrete = 1.0 to 1.5;
Tapered = 2.0 to 3.0; and
With jetting = 0.6 to 0.8.
Vibration-driven piles
Steel pipe = 0.75.

Ratio of δ/ϕ'
Dense saturated sand = 0.64 to 0.90.

Design factors for cohesive soils are as follows:

Friction

$f = kc$ (Figs. 5 and 6);
c = cohesion; and
k = soil resistance factor.

End-bearing

$q = cN_c'$; and
N' = bearing capacity factor = 9.0 for deep circular base.

These factors will be discussed separately for cohesionless and cohesive soils.

Cohesionless Soils

One method used in the past to determine frictional resistance in cohesionless soils was to conduct pull-out tests on full-scale piles. It has been shown by Hunter and Davisson (20) that frictional resistance measured during pull-out tests is approximately 30 percent less than the frictional resistance measured from compression tests on instrumented full-size piles. Also, from tests on instrumented piles, it has been shown by the same investigators that the measured friction depends on whether the residual stresses are included or excluded from the analysis. During a field load test, when a compressive load is applied, the resulting friction must overcome residual stresses in order for the pile to move. Once the pile moves, the frictional resistance tends to mobilize prior to tip resistance. The mobilization of frictional resistance, which occurs first at shallow depths, has been reported by D'Appolonia and Romuldi (12) and Coyle and Sulaiman (10). As additional loads are applied they are transferred in part or in whole to lower portions of the pile. In general, the frictional resistance decreases with depth and is independent of initial overburden pressure.

Current design methods incorporate neither pile movement nor residual stress. They do not consider the complete state of observed behavior. The simplified form of the formula generally used for computing frictional resistance in cohesionless soils is as follows:

$$f = Kp_o \tan \delta \tag{2}$$

The coefficient K is the earth pressure coefficient (2) and is defined as the ratio of horizontal to vertical earth pressure; p_o is the overburden pressure based on effective unit weights, and δ is the angle of friction between the soil and pile material. The magnitude of δ is usually estimated as being some fraction of ϕ', which is the effective angle of shearing resistance of the soil. Typical values of δ/ϕ' as recommended by Potyondy (43) range from 0.64 for smooth steel in saturated sand to 0.90 for rough concrete in saturated sand.

Considerable disagreement exists at present concerning the use of appropriate values of K for design purposes. A review of the works of Meyerhof (32), Nordlund (38), Mansur et al. (28, 29), D'Appolonia and Romuldi (12), and Hunter and Davisson (20) indicates the range of K-values as listed earlier.

The point resistance of piles in cohesionless soils is generally determined by the use of the bearing capacity equation originated by Terzaghi (54) and modified for deep foundations. The equation in its most simplified form is as follows:

$$q = p_o N_q' \tag{3}$$

The overburden pressure, p_o, is determined by multiplying an assumed effective unit weight times the embedded depth of the pile. The quantity N_q' is the bearing capacity factor for deep circular foundations. Values for N_q' have been determined theoretically by different investigators, and typical values are shown in Figure 3. The magnitude of N_q' varies widely, and these differences are the result of the assumptions used concerning the path of shear failure in the soil. In practice, the specific value of N_q' used by the designer is primarily a function of experience gained in a local area.

It has been shown by Kerisel (23) and Vesic (60) that q does not increase linearly with depth as implied by Eq. 3. As shown in Figure 4 for model piles, the point resistance increases to some limiting value and then remains more or less constant.

It should be noted at this point that Terzaghi's original bearing capacity equation contains a quantity that includes the parameter $N\gamma$. The magnitude of this term is so small in comparison with $(p_o N_q')$ that the quantity containing the parameter $N\gamma$ is neglected by most designers.

In order to determine N_q' for use in Eq. 3, it is necessary to evaluate ϕ', the effective angle of shearing resistance. The magnitude of ϕ' is a function of effective particle size, grain size distribution, relative density, and angularity of the particles. Representative values of ϕ' are usually determined from triaxial tests or standard penetration test results. DeBeer (13) has shown that crushing of sand grains under high intensity loads can establish a limiting value of q in Eq. 3 regardless of the magnitude of ϕ' or N_q'.

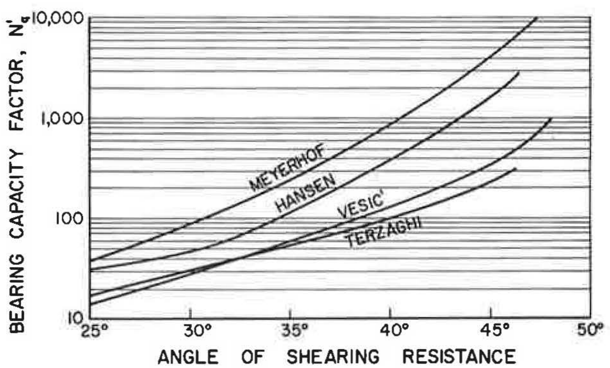

Figure 3. Bearing capacity factor for deep circular foundations (57).

Cohesive Soils

The frictional resistance of piles in cohesive soils is primarily a function of shear strength of the soil. The unit skin friction, f, is sometimes referred to as adhesion.

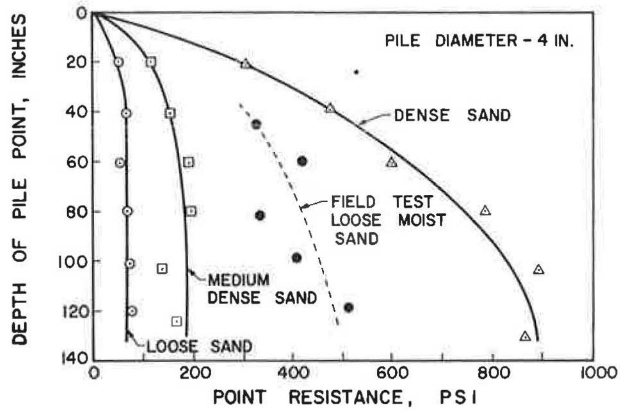

Figure 4. Variation of point resistance with depth (57).

The equation in a very simplified form is as follows:

$$f = kc \tag{4}$$

The cohesion, c, is the shear strength of the clay as determined by the unconfined, the in situ vane, or the triaxial quick test. The factor, k, is used to adjust the undisturbed shear strength. McClelland and Lipscomb (31) have defined k as the soil resistance factor. Numerical values for k have been determined from field load tests.

The relationship between f and c for soft-to-firm clay has been reported by Peck (39) and is shown in Figure 5. In this work, 36 piles were investigated, and the friction computed was approximately equal to the undrained strength of the clay. Relationships between f and c for firm-to-very-stiff, over-consolidated clay have been reported by Tomlinson (56) and are shown in Figure 6. It is noted that the unit frictional resistance is less than the undrained shear strength of the clay. Comparable results were also obtained by Woodward et

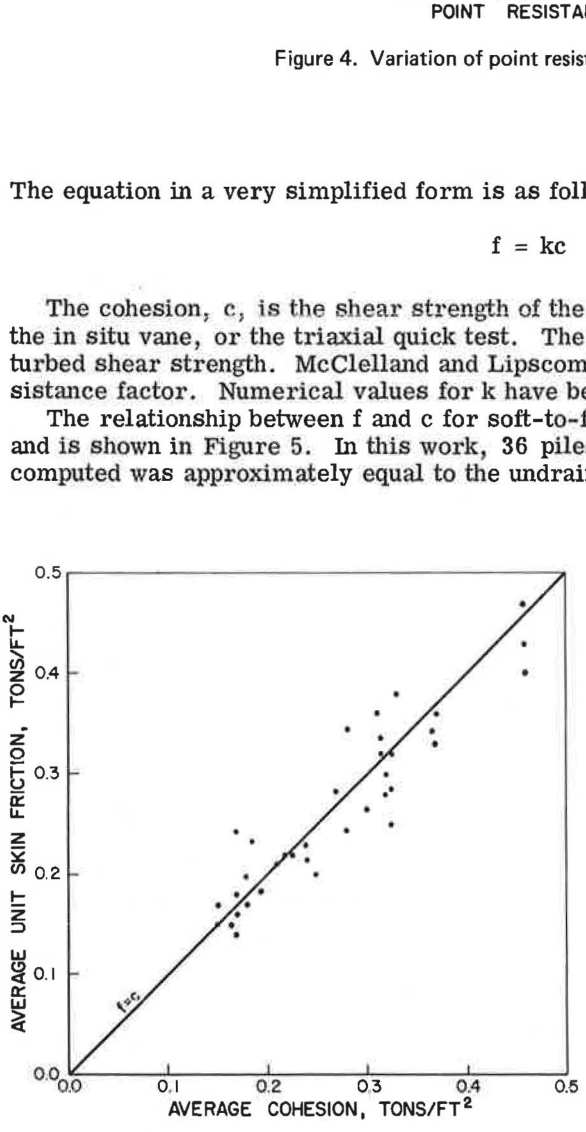

Figure 5. Average unit skin friction versus average cohesion (39).

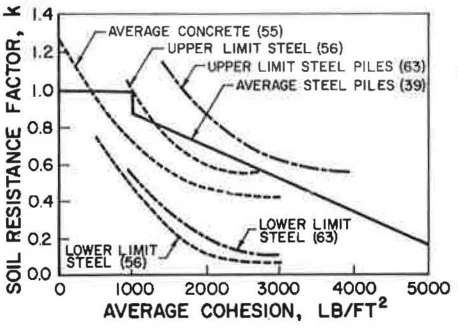

Figure 6. Soil resistance factor versus average cohesion (56).

al. (63). McClelland et al. (30) have suggested that f is approximately equal to c in normally consolidated stiff clay.

Values of frictional resistance shown in Figures 5 and 6 are the result of computations based on average values. Effects of pile movement and the distribution of friction with depth are not considered. A more exact analysis could conceivably introduce different values. For instance, Reese and Seed (45) and Coyle and Reese (9) have reported f-values for very soft clays ($c < 500$ psf) that were approximately equal to 1.5 c. These observations were based on instrumented pile test data in which the distribution of frictional resistance with depth along the pile was computed. From these studies on instrumented piles, the skin friction in clay appears to increase with depth.

Many designers neglect the point resistance of a pile in clay because as much as 85 percent of the pile load may be carried in skin friction. However, the point resistance in clay can be estimated by using the following equation:

$$q = cN_c' \tag{5}$$

The symbol c is the undrained shear strength of the clay. The factor N_c' is a bearing capacity factor for deep circular foundations. Equation 5 is based on Terzaghi's original bearing capacity equation, but only the cohesion term is considered.

According to Skempton (50) the value of N_c' for a round deep foundation is 9.0. It should be noted that the values of unit frictional resistance shown in Figures 5 and 6 are based on the assumption that the point bearing is equal to 9 times the cohesion. Therefore, values shown in Figures 5 and 6 should be used in conjunction with $N_c' = 9.0$.

BEARING CAPACITY OF SINGLE PILES BY THE DYNAMIC FORMULA METHOD

Although the emphasis has been given to the static formula method up to this point in this paper, it is felt that a short discussion is appropriate concerning the dynamic formula method. Much emphasis has been placed in the past on the use of dynamic formulas to predict the static bearing capacity of a pile. However, many investigators currently believe that dynamic formulas have been discredited to the point where their use has become limited in many areas. Lambe and Whitman (26) have summarized this attitude by stating that dynamic formulas are unreliable because of the difficulty in determining the energy lost during driving and the difficulty in relating resistance during driving to the static capacity of the pile.

The Michigan Pile Test Program (35) is a recent study that is worthy of note. This program has been summarized by Bowles (4), and the summary of the range of safety factors for the different dynamic formulas is given in Table 1. These data indicate that the modified Engineering-News formula is reasonably consistent over the range of loads considered. However, the wide range of values in these safety factors emphasizes the questionable reliability of dynamic formulas.

TABLE 1

SUMMARY OF RANGE OF SAFETY FACTORS FOR EQUATIONS USED IN THE MICHIGAN PILE TEST PROGRAM

Equation	Load Range (kips)		
	0-200	200-400	400-700
Engineering-News	1.1-2.4	0.9-2.1	1.2-2.7
Hiley	1.1-4.2	3.0-6.5	4.0-9.6
Pacific Coast	2.7-5.3	4.3-9.7	8.8-16.5
Redtebacher	1.7-3.6	2.8-6.5	6.0-10.9
Eytelwein	1.0-2.4	1.0-3.8	2.2-4.1
Navy-McKay	0.8-3.0	0.2-2.5	0.2-3.0
Rankine	0.9-1.7	1.3-2.7	2.3-5.1
Canadian National Building Code	3.2-6.0	5.1-11.1	10.1-19.9
Modified Engineering-News	1.7-4.4	1.6-5.2	2.7-5.3
Gates	1.8-3.0	2.5-4.6	3.8-7.3
Rabe	1.0-4.8	2.4-7.0	3.2-8.0

Note: Safety factor = Q_u/Q_d, where Q_d = safe load from formula.

Some designers use a combination of the dynamic formula method and a field load test to establish the bearing capacity of a single pile. For example, the Texas Highway Department uses the Engineering-News formula, which has been correlated with its cone penetrometer test as shown in Figure 7, to establish the initial pile design. A series of field load tests is then conducted at a bridge site to check for the possibility of using shorter piles and saving considerable money on a large job.

Another dynamic approach that has been developed recently is the wave equation analysis of pile-driving resistance. Work in this area has been done by Lowery et al. (27) and by Scanlan and Tomko (48), among others. Wave equation analysis can be used to estimate the bearing capacity of a pile that would be obtained if a pile were field-tested immediately after driving. In cohesionless soils this estimate may be fairly close to the actual ultimate bearing capacity of a pile. The main limitation in wave equation analysis at present is the lack of knowledge concerning the relationship between the dynamic and static soil resistance. These 2 components are linked empirically in the present analysis.

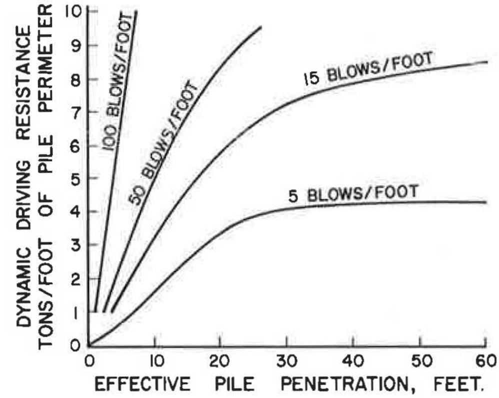

Figure 7. Texas Highway Department cone penetration test correlation (53).

PREDICTED LOAD VERSUS MOVEMENT RELATIONSHIPS FOR SINGLE PILES

The methods already presented are used to predict an ultimate bearing capacity independent of pile movement. Movement as used here refers to relative motion of the pile with respect to the soil in contrast to settlement, which as used here refers to soil consolidation or long-term settlement. In many instances, choosing an ultimate bearing capacity from field load tests is very much dependent on the magnitude and rate of deformation of both the pile top and the pile point.

In order to better predict ultimate bearing capacity, researchers have attempted to predict pile movement as well as pile load. D'Appolonia and Romuldi (12), Poulos and Davis (44), and Salas and Belzunce (47) have presented closed-form mathematical methods for piles either in linearly elastic or elastoplastic soils. Coyle and Reese (9) have presented a numerical method utilizing nonlinear soil properties. It is not possible to discuss all of these methods in this paper. However, the numerical method will be briefly presented because of its simplicity. The reader can refer to the References for more detail on the other methods.

Basically the numerical method works on the premise that, as a pile undergoes deformation under a load, the soil provides a frictional resistance along the side and a bearing at the tip that are functions of the load-deformation and strength characteristics of the soil. The mechanics of the procedure are valid, but the correct load-settlement curve will be obtained only if the correct load-deformation and strength characteristics of the soil are used in obtaining the solution. Therefore, any limitations of the method are the result of limitations in obtaining the correct load-deformation and strength characteristics of the soil.

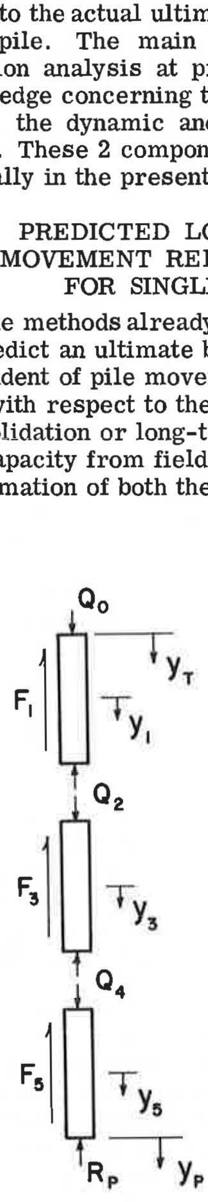

Figure 8. Axially loaded pile divided into segments.

Figure 8 shows an axially loaded pile divided into 3 segments with the forces acting on each segment. It is desired to determine the load, Q_0, at the top of the pile and the movement, y_T, at the top of the pile. The numerical procedure is initiated by assuming that a

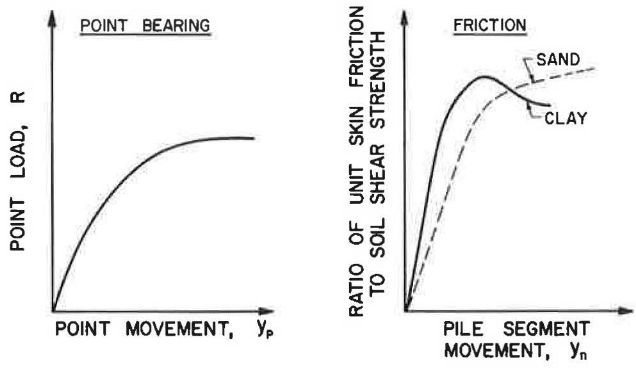

Figure 9. Soil load versus deformation characteristics.

small tip movement occurs at the bottom of the pile. The forces corresponding to the assumed movement in the bottom segment are determined from nonlinear load-deformation relationships for the soil acting on the pile. Typical soil load-deformation relationships are shown in Figure 9. Forces and movements are determined for each segment and added progressively from the bottom to the top of the pile until a load, Q_0, and a movement, y_T, are achieved (Fig. 8). The procedure is repeated and larger assumed tip movements are used until a series of values of Q_0 and y_T is obtained. These values can then be plotted as a predicted load-movement curve for the top of the pile as shown in Figure 10. The detailed step-by-step procedure is given in a recent text by Bowles (4). Because the procedure involves iteration, it is particularly adaptable for computer usage. A computer program is currently in use at Texas A&M University.

It is possible to develop relationships among skin friction, soil shear strength, and pile movement, such as the typical curves shown in Figure 9b, from the results of field load tests on instrumented piles. If the load is measured at different depths in an instrumented pile, a curve similar to that shown in Figure 11 can be established. Figure 11 shows that the average skin friction at depth 1 would be equal to the load at depth 0 minus the load at depth 2 divided by the circumferential area of the pile between depths 0 and 2. Also, the pile movement at depth 1 would be equal to the movement at the top of the pile, y_T, minus the elastic deformation occurring between depths 0 and 1. If skin friction

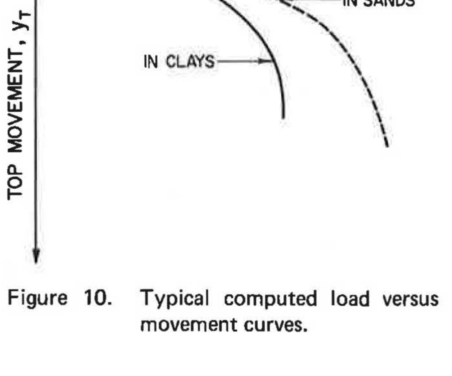

Figure 10. Typical computed load versus movement curves.

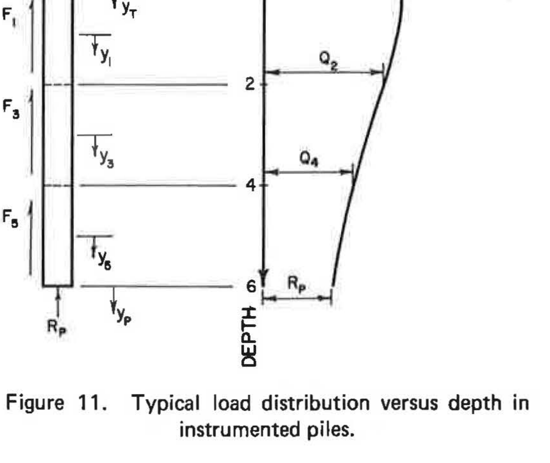

Figure 11. Typical load distribution versus depth in instrumented piles.

and pile movement values are established at different depths for different loads, and if the soil shear strength is known at the same depths, then a curve or curves similar to those shown in Figure 9b can be developed. Because the soil shear strength changes in most soils after a pile is driven, it is necessary to adjust the strength values obtained from in situ borings in the manner presented in the discussion on the static formula method.

Curves relating the ratio of skin friction to soil shear strength versus pile movement have been developed for clays by Coyle and Reese (9) and for sands by Coyle and Sulaiman (10). Additional work has been done on clays, sands, silts, and layered soils (11). Currently research is in progress at the Texas Transportation Institute involving an in situ testing device that measures skin friction and point-bearing as a function of movement. These data will be published in the near future and will give in situ curves of the type shown in Figure 9.

It should also be noted at this point that it is possible to develop relationships between tip load and tip movement such as the typical curve shown in Figure 9a from the results of field load test on instrumented piles. The tip load, R_p, is known directly from the instrumentation, and the tip movement can be computed by subtracting the elastic deformation in the pile from the movement, y_T, at the top (Fig. 11). Table 2 gives a summary of actual and computed ultimate loads and movements at safe loads obtained for piles in a variety of soils.

SAFE CAPACITY OF PILE GROUPS

There are 2 general criteria that must be satisfied in the design of pile groups. First, when subjected to maximum anticipated loads, the group must have an adequate safety factor against bearing capacity failure. Second, the settlement under both maximum and working loads must be tolerable. Within these limits, in actual practice the designer will probably base the final design on settlement rather than on bearing capacity. In any event, both criteria must be considered.

TABLE 2

ACTUAL VERSUS COMPUTED RESULTS

Pile No.	Pile Size (in.)	Pile Type	Embedded Length (ft)	Soil Type Along and Below Pile	Ultimate Load[a] (tons)		Movement at Safe Load[b] (in.)		Reference
					Actual	Computed	Actual	Computed	
1	12	Steel pipe	53.1	Sand and silty sand	150	125	0.1	0.08	(29)
2	16	Steel pipe	52.8	Sand and silty sand	194	180	0.14	0.09	(29)
3	20	Steel pipe	53.0	Sand and silty sand	228	230	0.15	0.125	(29)
H-14	16	Steel pipe	39.3	Sand	322	341	0.31	0.32	(59)
2	21	Steel pipe	65	48 ft silt then sand	299	305	0.14	0.17	(28)
4	17	Steel pipe	66	48 ft silt then sand	365	262	0.28	0.18	(28)
5	17	Steel pipe	45	Silt	120[c]	120[c]	0.06	0.06	(28)
6	19	Steel pipe	65	50 ft silt then sand	318	302	0.18	0.19	(28)
-	10 by 10	Precast concrete	100	15 ft silty sand, 45 ft clayey silt, 38 ft varved clay, then moraine	530[c]	530[c]	0.49	0.52	(6)
4	18	Steel pipe	62.6	17 ft soft clay[d], 13 ft sand, then stiff clay[d]	300[c]	300[c]	0.185	0.179	(64)
-	10.75	Steel pipe	110	Stiff clay[d]	207.5	157.5	0.3	0.25	(40)
-	10	Steel pipe	40	Stiff clay[d]	70	62.5	0.10	0.12	(30)
-	16	Steel pipe	40	Stiff clay[d]	140	67.5	0.10	0.07	(40)
-	30	Steel pipe	70	Soft clay[d]	160	165	0.10	0.10	(9)
-	12.75	Steel pipe	150	Stiff clay[d]	115	165	0.30	0.35	(40)
-	14	Steel pipe	335	Very soft clay[d]	115	97.5	0.50	0.48	(31)

[a]Ultimate load obtained by intersection of tangent lines drawn through initial and final parts of top movement curve at least 10 days after driving in clays.
[b]Safe load incorporates a safety factor of 2.
[c]Maximum load applied to pile.
[d]Measure of consistency after Terzaghi and Peck (54).

Depending on pile center-to-center spacing and soil type, there are 2 common empirical methods currently used to compute group bearing capacity. One method is based on group efficiency, and the other on block failure.

Piles driven into sands and gravels compact the surrounding soil. When the group is loaded, the piles and the soil between them move together as a unit reinforced by a rigid cap. The ultimate load for the group is as follows:

$$P_u = E \times n \times Q_u \tag{6}$$

where E is the group efficiency factor, n is the number of piles, and Q_u is the ultimate load for each individual pile. For cohesionless soils, the efficiency factor is equal to or greater than unity. Model tests on pile groups by Kezdi (25) and Stuart et al. (52) have shown that a maximum efficiency of two is given by piles in groups of 9 and 16 at a spacing of 2 diameters. The efficiency falls to unity when the spacing is increased to 5 or 6 diameters. Vesic (58) reports that the group efficiency factor is higher than unity because of an increase in frictional resistance of piles within the group compared to that of individual piles. The individual pile point loads remain approximately the same. In addition, pile caps resting on the soil contribute significantly to group capacity. Vesic (58) reports on a maximum group efficiency of 1.7 at spacings of 3 to 4 pile diameters. The efficiency reduces with an increase in pile spacing. Because these investigations were on model piles, with no direct correlation to the action of full-size piles, engineers tentatively are content with using E = 1.0 as indicated by Moorehouse and Sheehan (36). Therefore, to design for bearing capacity in cohesionless soils, the designer obtains Q_u from methods described previously, and multiplies this value by n, the number of piles in a group.

For cohesive soils, when closely spaced pile groups are loaded, piles and soil within the group may move together to result in a block failure. Model tests by Whitaker (62) showed block failure occurred at spacings closer than 1.5 diameters for groups of 9 piles, and closer than 2.25 diameters for 18 piles. For wider spacings, the piles failed individually, but group efficiency was about 0.7 at a spacing of 2.5 diameters, increasing to unity at a spacing of 8 diameters for piles 48 diameters long. Based on these

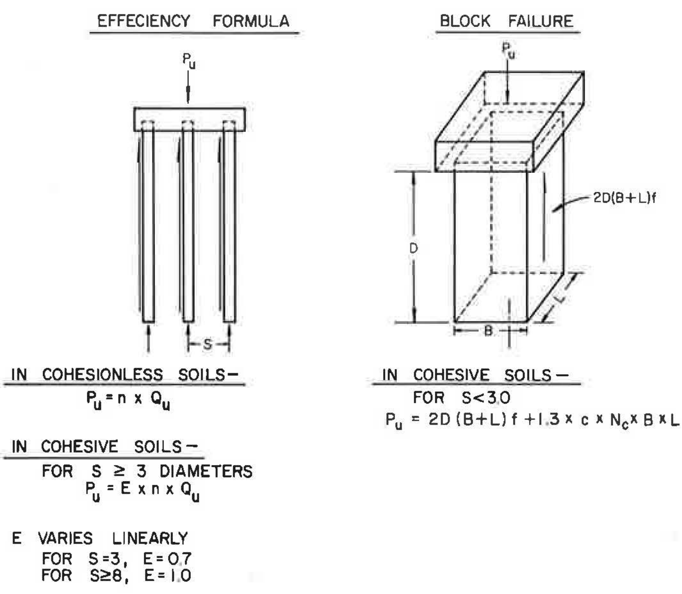

Figure 12. Bearing capacity of pile groups.

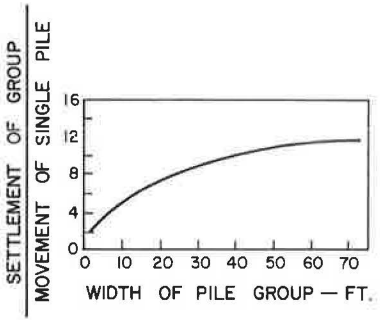

Figure 13. Settlement of pile groups in sand (51).

findings, Tomlinson (56) concludes that block failure can be eliminated at spacings equal to or greater than diameters of 3 piles.

For piles spaced closer than diameters of 3 piles, block failure can be considered by using the following formula after Terzaghi and Peck (54):

$$P_u = 2D(B + L)f + 1.3\ cN_cBL \tag{7}$$

where

D = embedded length of piles;
B = width of group;
L = length of group;
f = average frictional resistance around the group (Figs. 5 and 6);
c = cohesion of clay beneath the group; and
N_c = bearing capacity factor.

For piles spaced wider than 3 pile diameters, the group capacity can be found from Eq. 6 with the group efficiency varying from 0.7 for a spacing of 3 pile diameters to 1 at 8 pile diameters as recommended by Tomlinson (56). A summary for bearing capacity of pile groups is shown in Figure 12.

The settlement of a pile group in cohesionless soils is commonly greater than the settlement of the individual pile under the same load. This has been shown by Cambefort (7), Hanna (18), Skempton et al. (51), and Meyerhof (34). The difference between the settlements is attributed to the larger area influenced by load around and below the pile group. Group settlements in sands is estimated by tentative design curves that relate the settlement of a group of piles to the movement of a single pile. Such a curve as reported by Skempton et al. (51) is shown in Figure 13. These relationships will hold so long as the piles are entirely embedded in sand. If the sand stratum is thin or is underlain by clay, then a settlement analysis based on consolidation tests should be undertaken.

To estimate the settlement of pile groups in cohesive soils, it is necessary to distribute the piling loads in some manner to the soil. If the piles are embedded wholely in a compressible clay, then the load is assumed to be transferred to an elevation corresponding to two-thirds of the pile length below the top of the piles. If only a portion of the clay is compressible, then the load is assumed to be transferred to that portion that is susceptible to consolidation.

When piles are embedded in a soil that is consolidating because of either its own weight, surcharge loads, or drawdown of the water table, a load in addition to the working load will be transmitted from the soil to the pile by negative skin friction. Negative skin friction or downdrag may also occur in cohesive soils when excess pore water pressures that are initiated because of pile driving begin to dissipate. Conditions for this occurrence depend on the soil's susceptibility to consolidation and have been recognized qualitatively. However, difficulties arise when attempts are made to estimate the magnitude of induced downdrag forces. In cases where the soil is consolidating because of remolding effects, Zeevaert (65) assumes that the weight of the soil in the remolded zone is the upper limit for the downdrag force. If the soil surrounding a pile foundation settles because of surcharge loads or because of its own weight, the upper limit of downdrag forces would depend on maximum limiting frictional resistance f. Because of lack of representative values, design is tentatively based on the upper limit of f-values shown in Figure 6. In any case, the downdrag force should be added to the working load when pile foundations are designed.

CONCLUSIONS

Because this is a state-of-the-art paper, it would not be appropriate to give definite conclusions concerning the merit of using one particular method in preference to another for determining the bearing capacity of foundation piles. As a matter of fact it has been shown that no single method has been generally accepted for use by designers at this time. Probably most practitioners would agree that the field load test is the only foolproof method that can be used to determine bearing capacity at a particular site. Certainly, the static or dynamic method or both can be used for preliminary estimates of bearing capacity. In areas where designers have had considerable experience with a particular soil type, the static or dynamic method or both may be used with considerable confidence based on the experience factor.

The authors believe that the numerical method utilizing nonlinear soil properties related to movement shows much promise. Collection of data from instrumented field pile-loading tests is continuing in order to establish better relationships between skin friction, point-bearing, soil shear strength, and pile movement. Once reliable relationships are established, the numerical method can be used more effectively to determine the load-movement curve for a single pile and eliminate the need for costly pile loading tests.

If and when a reliable method is obtained for determining the bearing capacity of a single pile, the problem of final design involving pile groups still remains. Considerable improvement in the design of pile groups can be achieved by conducting more full-scale field group loading tests. Instrumentation of the groups would yield valuable data for the designers' use in determining pile spacing and distribution of the loads among piles in the group.

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Discussion

WILLIAM C. HILL, Oregon State Highway Division—The paper, within its brief allowance of time, has very adequately covered a subject that, as the authors noted, is very much an art and not a science. However, it has touched on only the problem of the designer, and this is only half of the problem of pile foundations. The other half of the problem, getting the pile into place so that it will carry the design load plus an added safety factor, has received no attention. In addition, the writers question the implied advocacy of load tests as the final answer to bearing capacity. The majority of structures requiring pile foundations are not of sufficient scope to warrant the cost of static load tests. In these run-of-the-mill projects, reliance must, for economic reasons, be on dynamic formulas.

Extreme refinements of these dynamic formulas must, more often than not, be considered in the same bracket as load tests requiring costs not economically feasible. Furthermore, because of the many unknowns inherent in the hammer, pile, and foundation material, these refinements are of questionable value. The day-to-day and sometimes hour-to-hour vagaries of the mechanical hammer on a job are common headaches for the foundation engineer in the field. The undisciplined operation of manually controlled mechanical pile hammers, the change in physical characteristics of the cushion material in the hammer helmet under continued impact of the hammer ram, and the changing soil characteristics as the pile compresses and consolidates the foundation zone are only a few of the multitude of variations in conditions and effects occurring in driving foundation piles under a single footing area.

Where projects, and even footings, are located in areas of mixed foundation material that may not have been thoroughly sampled and whose engineering properties may not have been thoroughly developed by laboratory tests, the use of complex dynamic pile bearing formulas requiring shear strengths and elastic deformation values of the soil is dangerous. The determination of the required pile penetration during driving operations is not made more exact through the use of such formulas, unless all parameters, even if developed by soil sampling and laboratory testing, are seldom accurate. In situ foundation materials vary notoriously. Layers and lenses of foundation material suddenly disappear or are encountered when not expected. The consistency of soils in the foundation zone often changes between the time of sampling and the driving of the piles. Samples often have been disturbed and altered when they arrive at the testing laboratory. Thus parameters placed in complex bearing capacity formulas more often than not reflect average conditions and values for any given area.

An approach to this pragmatic method has been made by the Bridge Section of the State Highway Division of the Oregon Department of Transportation. A statistical record of satisfactory pile-driving resistance was determined by use of the Engineering-News formula, simple for single-action mechanical hammers and modified slightly for double-action hammers. Soil-pile relationships have been generally recognized in that charts have been prepared for 3 pile types driven into groups of soil classes exhibiting generally similar driving resistances in tons per foot of foundation. To correlate these data with the in situ conditions, the pile-driving resistances have been plotted on log-

log graph paper against the standard split-barrel sampler blow counts (N) and blows of 1,000 ft-lb energy required to drive 1 ft (M) a miniature cylindrical tapered steel pile of approximately 2½ diameter and of 5 ft length. These correlations and explanations are being published by the Oregon State Highway Division: a preliminary report was published in 1965 (66). The success of this method covering the entire state of Oregon substantiates the conclusion of the authors that dynamic formulas can be used, but indicates that the method can be applied for many different soil classes and foundation conditions as determined by in situ tests and need not be limited to a single known soil class. As more field data are gathered, they are incorporated into a computer program from which graphic plots are made showing improved relationships. This program is described in the forthcoming report.

Nowhere is the problem of selecting hammer weights or size mentioned by the authors. Few designers realize that foundation piles can be only driven to an adequate depth by a hammer of sufficient size or weight to ensure that the impact energy reaches the pile tip. The Michigan tests showed conclusively that impact energy was largely lost in the hammer helmet and cushion material. The experience of the writer with prestressed concrete piles has shown that dangerously shallow penetrations will be the best that can be obtained, in most cases, through the use of dynamic formulas, unless the hammer ram weighs at least as much as the pile. Where large hollow cylindrical prestressed concrete piles are used, supplemental means of assistance, such as jets, are mandatory. Many such piles exceed 20 tons, the ram weight of the largest hammer common to the market. A rather comprehensive discussion by Olson and Flaate (67) of dynamic formulas for friction piles driven into sand indicates that the hammer-pile weight relationship should approach unity. The authors in their closing remarks can add greatly to the practical value of their state-of-the-art paper by discussing the relationship between ram weight and pile weight as a field problem.

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GREGORY P. TSCHEBOTARIOFF, Lawrenceville, New Jersey—This paper states that in cohesionless soils the earth pressure coefficient K varies from 1.0 to 1.5 in the case of impact-driven steel and concrete piles (Table 1).

The lower limit, K = 1.0, of this indicated range is much too high. During load tests, supervised and analyzed by the writer, in the West River area of the Connecticut Turnpike on steel friction piles driven into a deep glacial deposit of medium to dense fine sand and rock flour silt, the value of this coefficient was found to be K = 0.55 for steel monotube and pipe piles and K = 0.38 for steel H-piles (68). In this same paper the writer analyzed the results of a load test on an H-pile driven into a loose to medium sand layer as reported in a U. S. Steel catalog. The result was K = 0.56.

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Lateral Load Capacity of Piles

M. T. DAVISSON, Department of Civil Engineering, University of Illinois, Urbana

Pile foundations usually find resistance to lateral loads from (a) passive soil resistance on the face of the cap, (b) shear on the base of the cap, and (c) passive soil resistance against the pile shafts. The latter source is usually the only reliable one. Analysis of the problem yields deflections, rotations, moments, shears, and soil reactions as required for structural design. Beam-on-elastic-foundation theory is adequate for analysis of the problem. Most piles are relatively flexible and may be analyzed as though infinitely long. Only short rigid piles are likely to require consideration of the lower boundary conditions in analysis. Nondimensional solutions are available for both constant and linearly increasing modulus-depth relationships; solutions are also available for a stepped variation of modulus, k. Sufficient experimental data are now available to allow selection of the appropriate variation of k with depth. Typical values for k are available and have been related to readily observable soil characteristics. Simple lateral load tests also allow experimental determinations of the magnitude of k if greater accuracy is required.

•PILES are often required to resist lateral loads and moments in addition to their primary use as axially loaded members. The goals of designers are to determine deflections and stresses in the selected soil-pile system in order that they may be controlled within tolerable limits. Techniques for analyzing this problem in soil-pile interaction will be given in this paper.

A schematic representation of the loads acting on a pile foundation is shown in Figure 1. The pile cap may be subjected to moment, M, and shear, Q, loads in addition to the usual gravity load, W. Axial loads are resisted by the axial capacity of the piles and will not be discussed further here. The applied moment and shear are resisted to varying degrees by (a) passive soil resistance on the face of the cap, (b) shear along the base of the cap, and (c) moment and shear resistance of the piles at the junction to the cap. Clearly the moment and shear resistance of the piles are functions of the strength and stiffness of both the soil and the pile.

Passive soil resistance can be very effective in resisting lateral loads, but consideration must be given to the fact that it may not be permanent. Repairs, alterations, or other projects may be cause for removal of the soil; therefore, passive resistance is usually discounted or ignored. Shear along the base of the cap also can be very effective in resisting lateral loads. However, a slight settlement of the soil beneath the cap can essentially eliminate this resistance, and it is usually ignored for design purposes. The moment and shear resistances of the piles are usually the only factors considered sufficiently permanent for use in design. This discussion is aimed primarily at the resistance offered by the piles.

ANALYSIS

The deflected shapes of both a short and a long pile subjected to moment and shear loads are shown in Figure 2. A rotation θ can be used to define the deflected shape of a rigid member (Fig. 2a), whereas the flexural deflections become important for a

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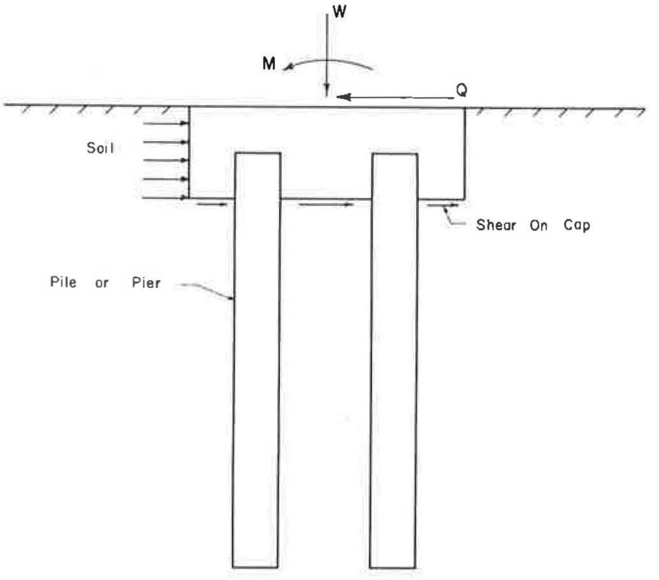

Figure 1. Sources of lateral resistance.

flexible member (Fig. 2b). Furthermore, the moment and shear at the lower end of a rigid member are quite important to a proper analysis, but they usually can be ignored for long flexible members (1).

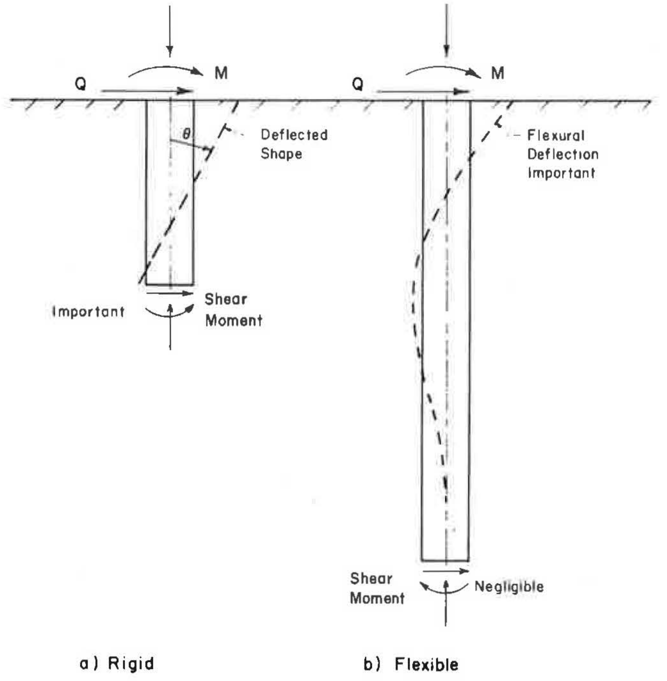

Figure 2. Rigid versus flexible pile or pier.

A practical procedure for the analysis of a soil-surrounded flexural member is needed so that a proper design can be made. The quantities needed are the deflections, moments, shears, and soil pressures. Deflection (and rotation) is important because of practical limitations on deformations of structures, and perhaps for determining the natural frequency for dynamic analyses. Moments and shears are needed for the usual structural design purposes, whereas soil pressures are required for checking against the allowable lateral soil pressures along the embedded portion of the piles.

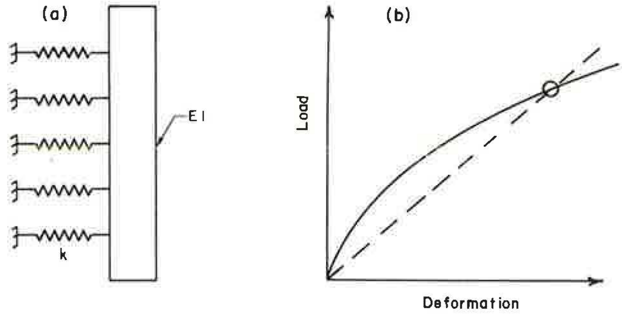

Figure 3. Subgrade modulus.

At present, the analytical techniques involving the theory of a beam on an elastic foundation are the most useful. The theory considers a continuous flexural member with stiffness EI (Fig. 3a) supported by infinitely closely spaced independent springs with stiffness k. However, the load-deformation characteristics of soils are not linear as shown in Figure 3b. It is necessary, therefore, to develop information on the secant modulus compatible with the deflection of the flexural member before proper use can be made of beam-on-elastic-foundation theory.

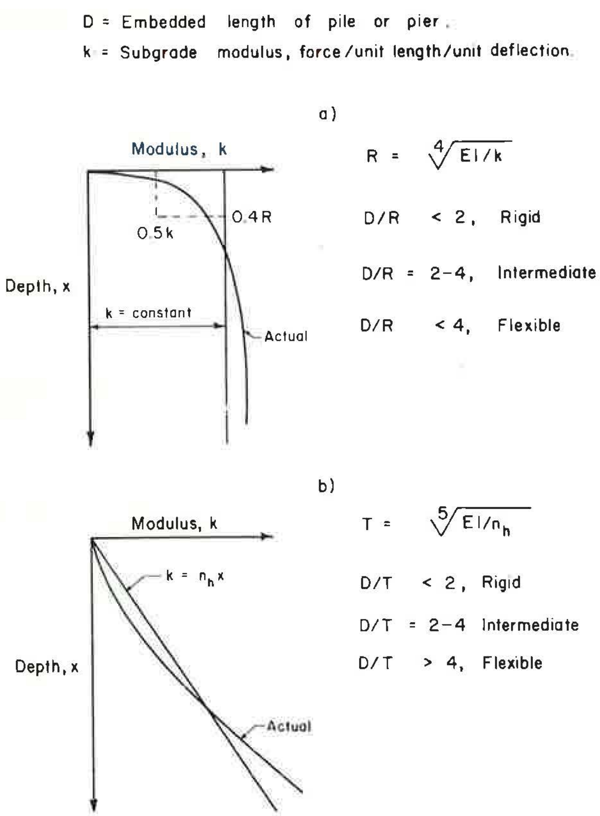

Figure 4. Relative stiffness factors.

A further complicating factor is that the soil stiffness is variable along the length of the pile. Therefore, beam-on-elastic-foundation theory must be modified to account for variations in the spring stiffness k. For example, preloaded clay actually has a variation of stiffness with respect to depth as shown in Figure 4a. A constant stiffness is usually assumed for analysis, but the errors may be 50 to 100 percent in both deflections and moment (2) because the analysis is unusually sensitive to soil stiffness variations in the zone adjacent to the ground surface. Granular soils and normally loaded cohesive soils, on the other hand, exhibit stiffness increasing almost directly with depth as shown in Figure 4b (3).

Beam-on-elastic-foundation theory involves the well-known equation

$$EI\frac{d^4y}{dx^4} + k_x y = 0$$

where EI is the flexural stiffness of the pile, x is the depth in the soil, y is the deflection, and k_x

is the spring stiffness or subgrade modulus. The subscript x indicates that k may be variable with depth x. As defined here, k has units of force per unit of length per unit of deflection (lb/in.^{2}); the width of the flexural member has already been considered.

Solutions to the differential equation are readily available for the cases where k equals a constant and $k = n_h x$; n_h is the coefficient of horizontal subgrade reaction (4). The latter is a linearly increasing modulus with respect to depth as shown in Figure 4b. Solutions can readily be obtained for other desired variations of k by hand methods or with the aid of electronic computers (5). The selection of appropriate values for k will be presented later.

NONDIMENSIONAL SOLUTIONS

Solutions for the aforementioned differential equation are readily available in nondimensional form. For constant values of k, the relative stiffness factor is defined as R where

$$R = \sqrt[4]{EI/k}$$

and has units of length. If the embedded length D is divided by R, the result is a dimensionless number indicative of the flexibility of the flexural member relative to the soil. Solutions for D/R values in excess of 4 are essentially equal to that for D/R equal to

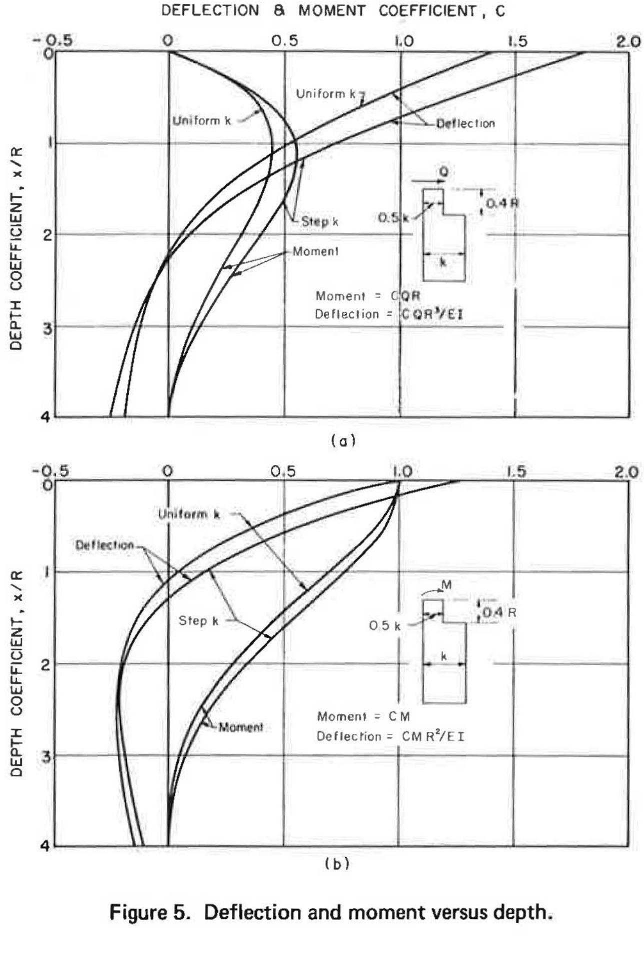

Figure 5. Deflection and moment versus depth.

infinity; almost all piles are in this category. This fortunate occurrence simplifies analysis because only one set of solutions is required and it is applicable to almost all problems. Solutions for deflection and moment for constant values of k, and also for a stepped variation in k, are available (2). The case where the soil to a depth of 0.4R has a modulus equal to 0.5k (Fig. 4a) is a better approximation for preloaded cohesive soils than the case where k is constant; such solutions are shown in Figure 5.

For the case where $k = n_h x$ the relative stiffness factor T is defined as

$$T = \sqrt[5]{EI/n_h}$$

and has units of length. If the embedded length D is divided by T, the result is a dimensionless number indicative of the flexibility of the system (Fig. 4b). In Figure 6 the nondimensional deflection coefficient has been plotted versus the nondimensional depth coefficient x/T where x is the depth below the ground surface; this plot has been made for a shear load Q for various values of D/T. Note that the deformations for D/T = 2 are essentially due to rotation (relatively rigid member), whereas deformations for D/T = 4 are essentially the same as for D/T = 5 and D/T = 10 and are dependent on the flexural deflections. In most practical cases D/T exceeds 4 and only one set of solutions is needed; such solutions are readily available (5, 6).

Fixity at the top of the flexural member strongly influences both deflection and moment. This is shown in Figure 7 (7), where the nondimensional moment coefficient has been plotted versus nondimensional depth x/T. A fixity factor F (Fig. 7) has been used to describe the degree of restraint at the top of the flexural member; thus, the influence of both moment and shear loads has been combined in one diagram. An F-value of zero corresponds to a free-head case, and the maximum moment occurs at a depth of 1.35 T. An F-value of -0.93 corresponds to full fixity, and the maximum moment occurs at the top. As a practical matter the degree of fixity that can usually be developed is, in the writer's experience, approximately -0.4 to -0.5; note that in this case the positive and negative moments are approximately equal, perhaps an aid to efficient use of flexural resistance. Nondimensional deflections versus depth are shown in Figure 8 in a similar manner.

For relatively flexible flexural members embedded in a relatively rigid concrete cap, an estimate of fixity at the pilehead can be obtained by considering the problem as a beam on an elastic foundation wherein an analysis is made of the pile embedded in concrete. In this case, the concrete controls the modulus of subgrade reaction. Note that axial loads aid fixity and that conditions at the pile top are likely to exert considerable influence on behavior. This occurs with short embedments where the beam cannot be considered infinitely

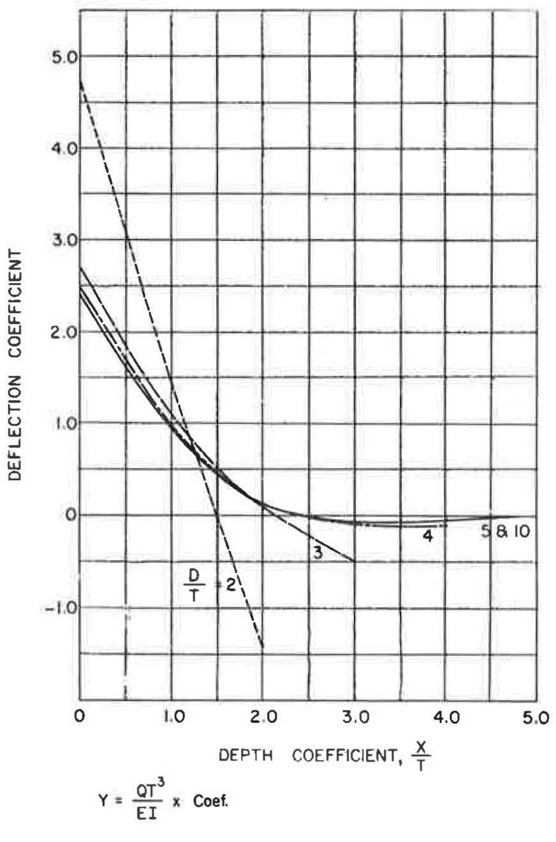

Figure 6. Deflection versus depth.

embedded. In other structural schemes, the structural connection at the top of the piles can be considered to sit on springs with axial, lateral, and rotational stiffnesses that are a function of both the pile and the structural characteristics of the connection.

The analysis also allows a calculation of the soil reactions ky. These reactions can be checked against the allowable lateral pressures determined from theory and soil strength parameters (8).

SOIL MODULUS

Typical values for k are available for a wide variety of soils. For a given soil, k increases as density increases, as would be expected. The values for k given in Table 1 are based on both the literature and the writer's experience. On the basis of simple soil tests, such as the standard penetration test or the unconfined compression strength, reasonable values can be selected for k.

There are 2 phenomena that have a marked effect on k, namely, group action and repeated loading. With respect to group action, the spacing in the direction of the load is of primary importance. At a spacing center to center of 8d or more

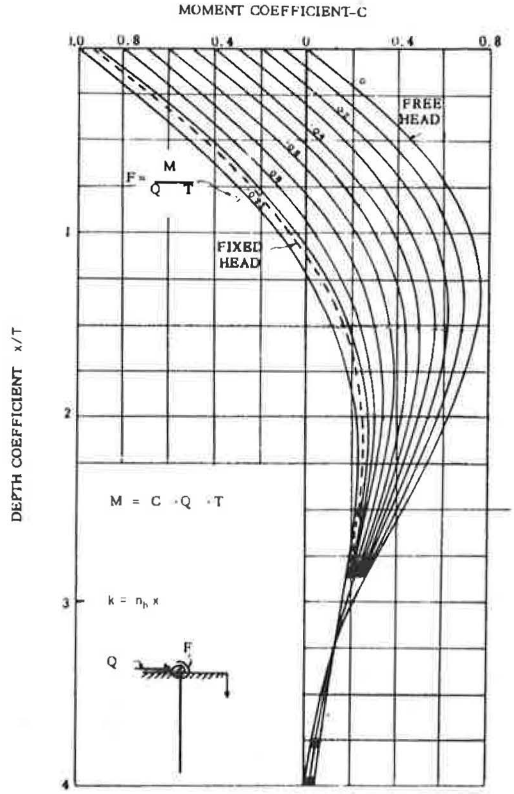

Figure 7. Moment versus depth.

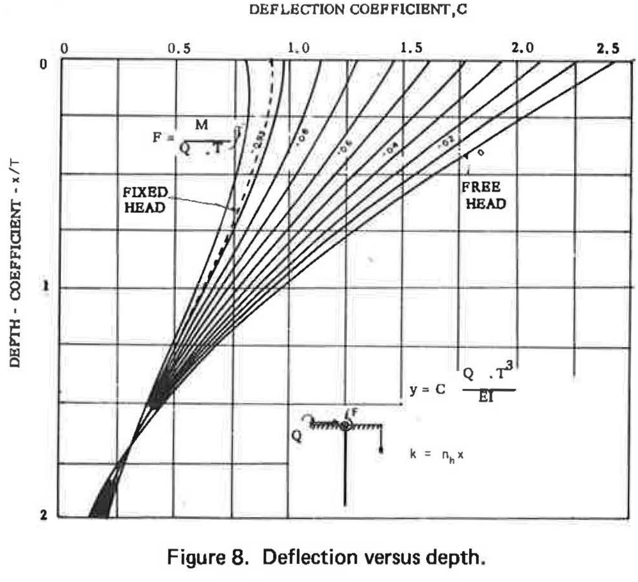

Figure 8. Deflection versus depth.

TABLE 1

ESTIMATED VALUES FOR k

Soil Type	Value
Granular soils	n_h ranges from 1.5 to 200 lb/in.^{3}, is generally in the range from 10 to 100 lb/in.^{3}, and is approximately proportional to relative density
Normally loaded organic silt	n_h ranges from 0.4 to 3.0 lb/in.^{3}
Peat	n_h is approximately 0.2 lb/in.^{3}
Cohesive soils	k is approximately 67 C_U, where C_U is the undrained shear strength of the soil

Note: The effects of group action and repeated loading are not included in these estimates.

(d is the pile diameter), there is essentially no influence of one pile on another providing the spacing normal to the direction of loading is at least 2.5d (Fig. 9). When the spacing parallel to loading is less than 8d, the effective value of k (k_{eff}) is less than that for an isolated pile. At a spacing of 3d, k_{eff} is approximately 0.25k. For other spacings, k_{eff} can be determined by interpolation between 3d and 8d. This information is based on a model study on piles in sand (7).

Repeated loading causes some deterioration of the soil resistance, effectively reducing the modulus k. The net effect is that the deflection observed under first application of a load is essentially doubled if the load is cycled 50 times or more (1, 7, 9). Moments are also increased and occur over an increased depth of embedment. Repeated loading has the effect of reducing k to approximately 30 percent of the applicable to initial loading.

If both group effects and repeated load effects must be considered, k_{eff} can be as low as 10 percent of that applicable to initial loading of an isolated pile (7).

It is the writer's experience that for most problems an analytical investigation based on reasonable values for k, determined with the aid of routine soil tests and judgment based on data given in Table 1, will lead to the decision that an adequate design can be developed without further information. For the remaining problems, it is relatively easy to make in situ tests to get more accurate design information if the potential benefits outweigh significantly the additional cost of an acceptable design based on available data.

A simple lateral load test on a pile will provide accurate design information. For simplicity the loads should be applied and the deformations measured at the ground

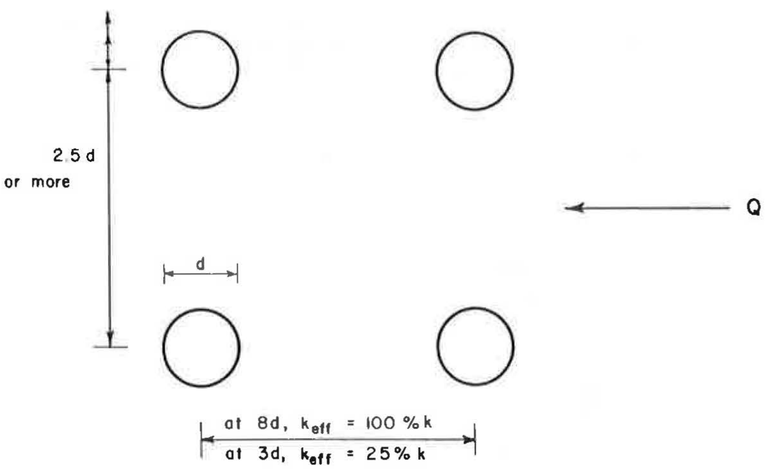

Figure 9. Effect of group action.

surface; however, this is not essential. Further, the test pile need not be a prototype. It is only necessary that the pile be of sufficient depth to be considered infinitely long for theoretical evaluation. It is necessary to make an assumption regarding the nature of the variation of k with respect to depth; for example, constant, stepped, or triangular. Then the appropriate nondimensional coefficients and expressions are used to back-calculate k or n_h. Corrections may then be applied, as described previously, to account for group action and cyclic loading.

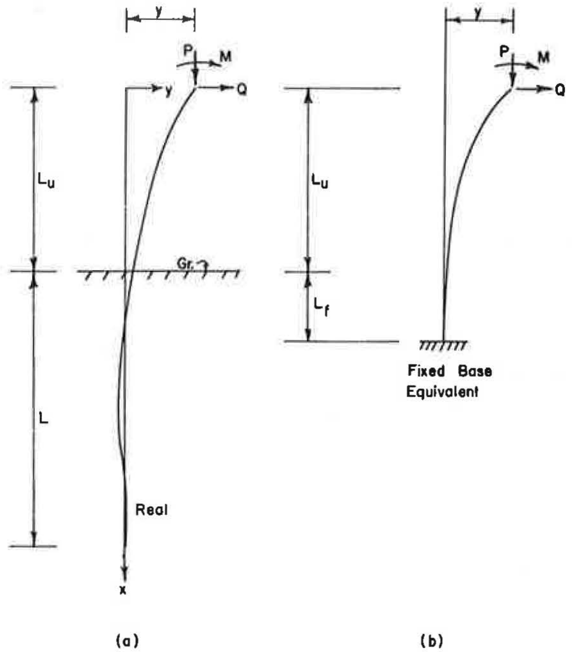

Figure 10. Partially embedded pile.

PARTIALLY EMBEDDED PILES

Often, with partially embedded piles, the top of the pile is fixed to some degree and the structure is then statically indeterminate. It is most convenient to the structural engineer if the pile (Fig. 10a) can be replaced for the purpose of analysis by an equivalent free-standing pile (Fig. 10b) that is fixed at some depth, L_f, below the ground surface. A theoretically correct solution for determining the depth to fixity, L_f, for long piles, i.e., D/T or D/R > 4, is available (10). The solution satisfies the conditions that the deflection and rotation at the top of the equivalent pile as well as the critical buckling load are the same as for the real pile.

The depth to fixity is dependent on the stiffness of the pile and the magnitude and variation of the soil resistance but is reasonably constant when expressed in terms of the dimensionless parameters given previously. L_f can be determined with little approximation from the following:

$$\text{If } k = \text{constant and } \frac{L_u}{R} > 2, \text{ then } L_f = 1.4\,R$$

$$\text{If } k = n_h \cdot X \text{ and } \frac{L_u}{T} > 1, \text{ then } L_f = 1.8\,T$$

The equivalent cantilever beam-column defined can be used in conventional frame analyses for determining moments and loads at the top of the pile and for determining the buckling load for the pile. However, the moment computed for the fixed end of the equivalent pile will be considerably larger than the actual moment in the real pile. Therefore, to analyze the embedded portion of the pile it is necessary to resort to the procedures previously discussed, using the moments and loads at the groundline. These can be determined from basic principles of statics once the conditions at the top of the pile have been determined from the frame analysis (10).

SUMMARY

Pile foundations usually find resistance to lateral loads from (a) passive soil resistance on the face of the cap, (b) shear on the base of the cap, and (c) passive soil resistance against the pile shafts. The latter source is usually the only reliable one. An analysis of the problem should yield deflections, rotations, moments, shears, and soil reactions as required for purposes of structural design. Beam-on-elastic-foundation theory is adequate for analysis of the problem. A brief study indicates that most piles are relatively flexible and may be analyzed as though infinitely long. Only short rigid piles are likely to require consideration of the lower boundary conditions in analysis.

Nondimensional solutions are available for both constant and linearly increasing modulus-depth relationships; solutions are also available for a stepped variation of k. Sufficient experimental data are now available to allow selection of the appropriate variation of k with depth. Typical values for k are available and have been related to readily observable soil characteristics. The applicable nondimensional solutions coupled with simple lateral load tests also allow experimental determinations of the magnitude of k if required.

Group action can cause a reduction in effective modulus to 25 percent of that applicable to an isolated pile. Further, cyclic loading can cause deflections to double, approximately, compared to that for the first load cycle. This causes a further reduction in the effective modulus. If both effects are present, the effective modulus may be only 10 percent of that for first loading of an isolated pile.

Fixity at the top of the pile is difficult to attain with the structural details commonly used. A fixity of 50 percent is usually attainable and has the advantage of approximately equal positive and negative moments, thus making efficient structural use of uniform flexural members. If deflections must be minimized, then increasing fixity is a very efficient way of achieving stiffness.

A technique is available for analyzing partially embedded piles utilizing the same nondimensional parameters presented for fully embedded piles. A depth to fixity is introduced based on both soil and pile stiffnesses, thus eliminating the objections to similar procedures involving arbitrary depths to fixity.

Analyses based on conservative, assumed values for k will usually indicate that an acceptable design can be obtained economically. If, however, the analysis indicates that better design data may yield significant savings, it is relatively simple to generate a field test program that will provide the data.

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Current Construction Practices in the Installation of High-Capacity Piling

BEN C. GERWICK, JR., Santa Fe-Pomeroy, Inc., San Francisco

Structural and economic considerations are causing a trend toward the use of high-capacity piling for highway bridges. They are being employed in combined loading to resist bearing, uplift, and lateral forces with design loads from 200 to 1,500 tons. These piles must be installed to penetrations in soil sufficient to develop their capacity, and this requires special techniques and equipment. Installation techniques include weighting, driving, vibration, jetting, drilling, rotation, and lubrication. Because the capacity of a pile is determined by both the structural capacity of the pile and the capacity of the soil, i.e., the pile-soil system, installation techniques must not permanently decrease the soil-supporting capacity. After installation, pile-soil capacities may be improved by consolidation of surrounding soils, concrete plugs, grout injection, and expansion of the pile tip. A review of important recent installations of high-capacity piles of various types is instructive in illustrating the various combinations of techniques that have been successfully employed. A review and analysis of problems also directs attention to those areas requiring further development. The variables facing both the designer and the contractor include character of the soils, depth of water or soft material, loads to be carried, access for equipment, magnitude of the job, available equipment for transporting, lifting, and installing, and available facilities for fabrications or manufacture. It is essential that the design and installation be integrated if success is to be obtained with these high-capacity piles. Thus, the maximum benefits of high-capacity piles can be made more widely available to the bridge engineering profession.

•AS HIGHWAY BRIDGES are built in congested waterfront areas and in deep water, both structural and economic requirements demand higher capacity piling. Such piles preferably serve as structural columns as well as piles, taking combined bending and direct load and extending up as high as possible to at least the groundline or waterline and, where feasible, on up to the underside of the deck.

High-capacity piles involve the interaction of pile and soil. They must penetrate a sufficient distance to develop the bearing capacity, be installed in such a manner as to take the lateral bending capacity (pile-soil interaction), and be installed with sufficient accuracy to minimize eccentricities. Inherently, these piles are long, large, heavy, and expensive. They require large equipment for transporting and handling. Proper methods must be developed for their successful installation. These methods must be considered both by the designer and by the constructor so that a comprehensive and well-integrated procedure is attained.

High-capacity piles are being simultaneously developed on at least 3 major fronts: bridges for highways and railroads, building foundations, and marine structures for harbor, coastal, and offshore facilities. Design loads range from 200 to 1,500 tons. Each of these applications is making use of the technology developed by the others, and this is mutually stimulating. The total number of such installations to date is relatively limited; therefore, it is important to gather experience from as many of these related applications as possible.

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Installation techniques are primarily directed at achieving the required penetration without reducing the carrying or lateral capacity of the soil. A secondary purpose may be to consolidate (or prestress) the soil during installation in order to improve its carrying capacity.

For high-capacity piles, it is frequently extremely difficult to obtain the required penetration. Many different techniques may be required. Frequently, simultaneous or consecutive use of two or more of these techniques is desirable or necessary. Basic techniques include weighting; driving; vibration; jetting; predrilling; drilling out of core; lubrication by injection, electro-osmosis, or air-bubbling; and rotation and oscillation.

Pile capacity, after obtaining penetration, may be improved by techniques such as consolidating surrounding soils, as by vibration; using a concrete plug; injecting grout; and expanding pile tip.

TYPICAL INSTALLATIONS OF HIGH-CAPACITY PILING

A review by specific cases or categories of some of the important uses of high-capacity piling of different types may give a broad view of the scope involved.

1. Steel H-piles used for a highway bridge in California have 200- to 225-ton capacity, on 14-in. by 14-in. by 200-lb piles 140 ft long and have been driven to end-bearing through mud and sand into soft rock.
2. Composite prestressed and H-piles, i.e., the top half is prestressed concrete, and the bottom half is steel H-pile, have been used for highway bridges in California (200-ton capacity and 213 ft long) and in New South Wales, Australia (240-ton capacity and 200 ft long).
3. Drilled-in-caissons, i.e., pipe piles drilled into rock, are much used for building foundations in New York and are occasionally used elsewhere, e.g., at a large paper mill in Oregon. Typically, they are 24 in. in diameter with ½-in. walls filled with concrete. They take loads up to 300 tons. By inserting a structural steel core, loadings have been increased to 1,000 tons per pile and more.
4. Pipe piles, both closed and open-ended, have been driven through varying strata to bearing on rock or in sand. They are usually filled with concrete to increase their structural load-carrying capacity.
5. Prestressed concrete piles have been used extensively for building foundations at very high capacities; e.g., high-rise buildings in San Francisco, have 200-ton capacity piles, 18- by 18-in. square section, 138 ft long (Fig. 1).
6. Prestressed concrete cylinder piles (Fig. 2) for bridges and harbor structures have design loads to 200 and 300 tons. Piles have been both closed and open-ended with 36- to 54-in. diameters and up to 250-ft lengths and are capable of taking large lateral loads and bending movements as well as vertical loads. They are used on major highway and railroad bridges in California, Oregon, Washington, Louisiana, South Dakota, Virginia, and New York. They are also used for harbor structures in Malaya, Fiji Islands, Indonesia, and Singapore and for offshore platforms in Lake Maracaibo. Similar cylinder piles, although usually not prestressed, are extensively used in Russia for river crossings.
7. Prestressed concrete caisson piles have very large diameters (4 m or 14 ft) and are used for the Oosterschelde Bridge in The Netherlands (Fig. 3) where they are up to 165 ft in length.
8. Large reinforced concrete piles for offshore structures have been installed in the Gulf of Mexico and especially Lake Maracaibo. Of particular interest are tapered piles that have increased cross section at point of maximum bending.
9. Steel cylinder piles, concrete filled, are used in the Lower Yarra River (Westgate) Bridge in Melbourne, Australia, where they are sunk through silts and decomposed basalt into hard basalt rock. At a Naval shipyard in California, similar steel cylinder piles were sunk through muds and debris, then a socket was drilled ahead into soft rock and concreted.
10. Steel cylinder piles for offshore platforms have ranged from 30 to 42 in. in diameter. They are characterized by extreme length (up to 300 ft of penetration in soil in water depths of 300 ft, for a total length of 600 ft). They have been driven to extremely

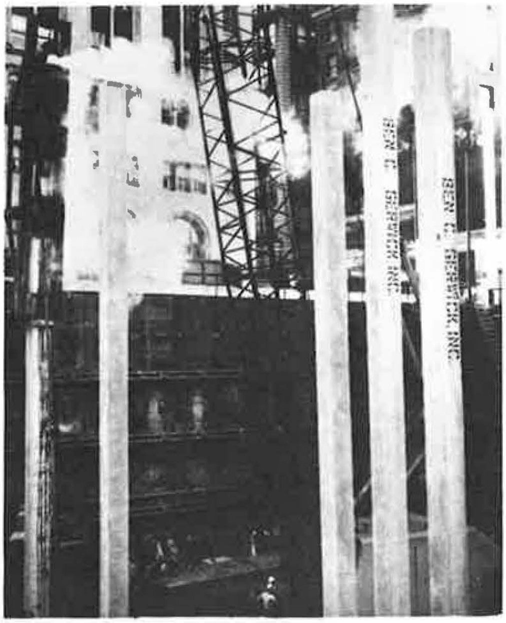

Figure 1. High-capacity prestressed concrete piles used in Wells-Fargo Building, San Francisco.

Figure 2. Prestressed concrete cylinder pile used in Napa River.

high ultimate loads (up to 3,000 kips). When penetration has reached refusal above the predetermined tip, then insert piles of smaller diameter have been driven ahead and beyond to the required penetration.

11. Steel caisson piles up to 12 ft in diameter and 200 ft in length (Fig. 4) have been used in marine terminals in Cook Inlet, Alaska, to take combined vertical and horizontal loads (due to ice, wind, mooring, current, and earthquakes). These have been sunk

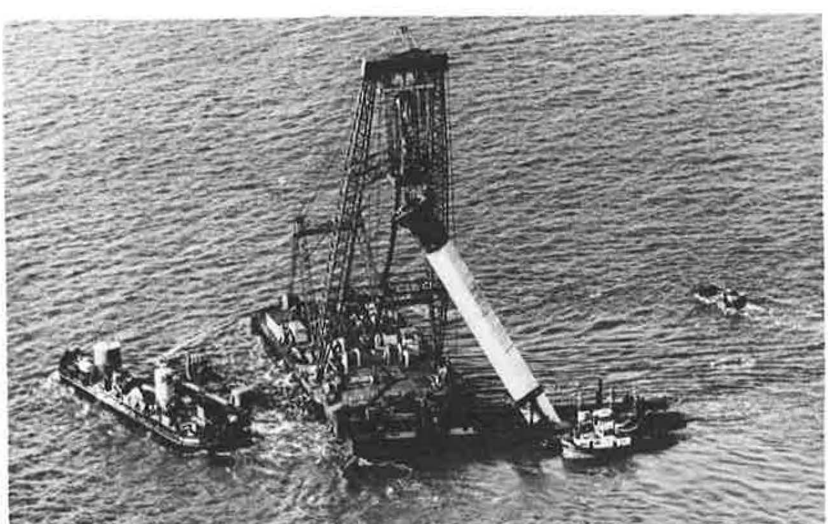

Figure 3. Prestressed concrete caisson pile used in Oosterschelde Bridge, The Netherlands.

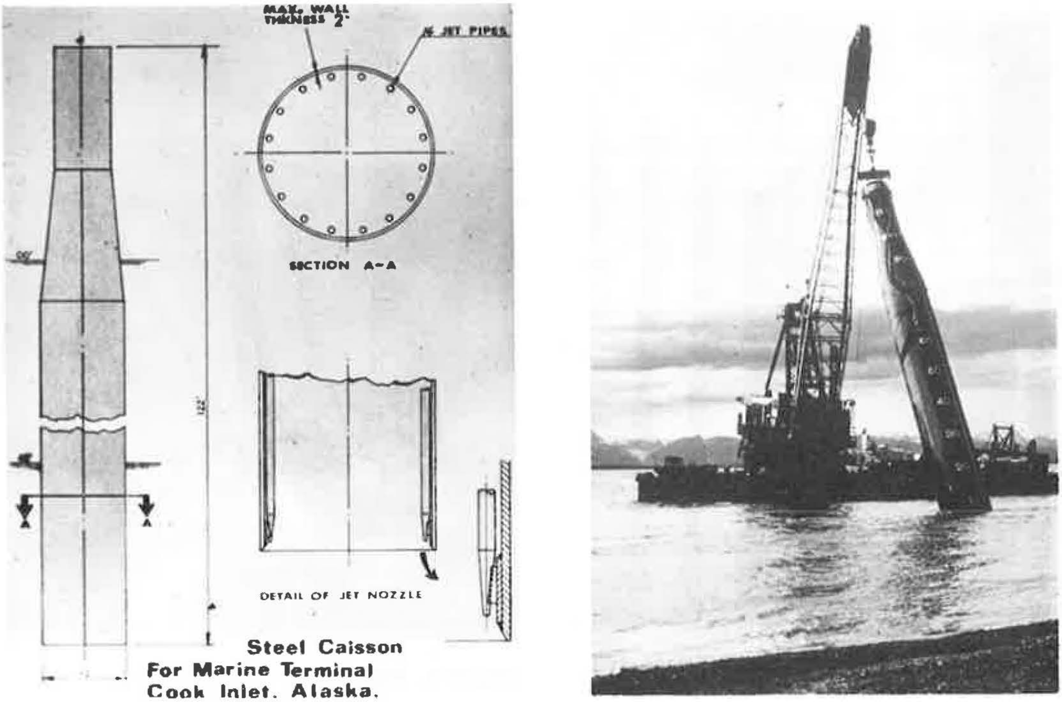

Figure 4. Steel caisson pile used in Cook Inlet, Alaska, to withstand vertical and horizontal loads.

through sands, gravels, cobbles, and glacial till. Such caisson piles have been proposed for the future Turnagain Arm Highway Bridge near Anchorage, where difficulty of installation is combined with extremely high ice loads.

12. Drilled-in piles (caissons) have been drilled in various diameters and depths, a reinforcing cage placed, and concrete poured. On occasion, steel casings are used to line the drilled hole. Precast column sections and structural steel sections have been set in drilled holes and grouted or concreted to lock the core to the soil.

TYPICAL INSTALLATION TECHNIQUES

Weighting

Extremely large concrete piles have been sunk in the soft silts of Lake Maracaibo by the application of weights. Concrete blocks, in increments, up to a total of several hundred tons, are placed, forcing the pile to the required tip elevation and bearing capacity.

It has been proposed to apply pull-down force by means of prestressing tenders inserted in holes drilled through the pile to rock or firm soil, anchored by grout, and then jacked against the pile. Such a method could be very effective and practicable in favorable site conditions. These weights, or pull-down forces, are much more effective, particularly in granular soils, if applied in intermittent, repeated fashion rather than as just a static load. This can be accomplished if the deadweight can be supported separately, i.e., on the adjacent ground or water, and the force applied by hydraulic jacks.

For the Oosterschelde Bridge in The Netherlands, 14-ft diameter concrete caisson piles were sunk in unique fashion. A yoke was placed over the pile and was attached to a matching yoke on the bow of the derrick barge. The barge was literally lifted up onto the pile, exerting a downward thrust of 600 to 1,000 tons. Sinking was aided by internal excavation while the combined weight of the caisson and thrust of the barge was applied.

Driving

Increasingly large hammers have been employed to install high-capacity piles. As a result of experience and, more recently, information from the wave equation theory, rams are made ever heavier, but the velocity of impact is held about the same, e.g., the equivalent of 3 ft of free fall. Ram weights for steam hammers are commercially available up to 60,000 lb, and even larger ones are under development.

The wave equation theory has established that an increased cross-sectional area of the pile gives a greater total force for penetration, although, of course, an increased cross-sectional area may also develop greater soil resistance. Thus for steel piles, thicker walls increase drivability substantially. For offshore piles, minimum wall thicknesses of 1 in. and greater are commonly employed.

One favorable result from the wave equation, confirmed by experience, is that, except for dampening, there is no decrease in drivability due to increased pile length. The old belief that the mass of the pile had to be accelerated is shown to be erroneous.

Proposals have been made to use much increased ram weights, up to 200 to 400 tons and even 1,000 tons, raised and lowered by hydraulic means to achieve even more drivability and greater bearing capacity. One such hammer that is under development for offshore piles (U. S. patent applied for) provides a means for release of the water pressure (hydraulic ram effect) that would oppose the impact from the hammer.

Jetting

Jets may be effectively used to cut ahead of the pile and to lubricate the sides against skin friction. Cutting jets must be of high pressure and must be located at the tip. Lubricating jets are low pressure, high volume, and must deliver water at intervals along the sides.

Jets may be built into the piles, such as internal jets in prestressed concrete piles. Details and operation must be such as to prevent plugging or blocking of the jet during driving. Side nozzles may be provided to permit lubricating water to escape.

External jets must be capable of control so that the nozzle can be kept in proper relation to the pile. Sometimes external jets tend to become stuck in sand. Tiny holes cut in the jet pipe at intervals along the sides will lubricate it and prevent sticking.

Jetted and Driven Steel Caissons

Large diameter (4 to 12) steel caissons have been installed in the sands, silts, and glacial till of Cook Inlet, Alaska, by a combination of jetting and driving. Because the weight of these caissons is more than 100 tons, it is obvious that the driving energy available, say 60,000 to 90,000 ft-lb, is inadequate in terms of conventional driving formulas. However, the hammer does send very effective compressive waves from the head to the tip.

Jetting is needed to sink the caisson, but the problem is how to get the water to the cutting edge. After considerable development and experience, the solution that has emerged is to install jet nozzles at 24-in. intervals around the circumference just inside the caisson walls with the nozzles held back 2 to 6 in. from the tip, and thus protected by it. These jet nozzles are fed by riser pipes welded to the pile walls and manifolded at a ring just below the pile head, where hoses from special jet pumps are connected.

Jetting alone is used to sink the caisson as far as possible, the caisson being alternately raised and lowered a few feet. Then the hammer is used with the jets still running until the tip is within a few inches (12 to 24 in.) of desired elevation. The pile is then seated by hammer alone.

In addition to lubricating the sides and breaking up the sand ahead, the jets also prevent a plug of densified sand from forming in the caisson tip.

Vibration

A number of heavy-duty vibrators have been developed to sink large piles through granular materials. One of these, the Bodine hammer, utilizes a substantial power at frequencies up to the sonic range. It has been extremely effective in demonstrations after all jetting has ceased, the sand grains will be reconsolidated. It is not a question of driving 2 ft or 5 ft or some arbitrary distance; rather, it is a minimum number of blows even if the pile only moves 1 in. During this period the water will drain outward from the pile, causing further consolidation. In some poorly graded sands, it will be found desirable to drive say 100 blows and then, after an interval of 30 minutes to an hour or more, drive another 100 blows. This later driving will often achieve a few more inches' penetration and a secondary consolidation of the sand.

Expansion of Pile Tip

The pile tip may be expanded to increase end-bearing capacity. Such expansion not only increases the bearing area but also consolidates the soil and mobilizes its resistance. One means of expansion is the ramming of concrete from the tip. Through a hollow casing, a load of fresh concrete is placed and a ram is installed and driven down, forcing the concrete out as the casing is slightly retracted. This same effect can be accomplished by air pressure; the casing is capped, and air is applied.

Other methods proposed for expanding the tips of steel pipe piles include controlled explosives and hydraulic rams. So far these have been judged unsuitable for high-capacity piles; however, they have been used for anchor piles. Also a mechanically spreading cone can be enlarged by driving on a ram. This also has so far been limited to small-sized anchors but may be developed for highly loaded piles in the future.

Concrete Plugs

Where the core of the pile has been removed to or near the tip, a concrete plug may be placed to increase the end-bearing area. Densely compacted soil at the extreme tip does not need to be removed; it is usually satisfactory to place the concrete plug on top of this soil "plug" and hold it in position.

Concrete plugs may be placed by tremie methods. Care must be taken that the hydrostatic head of the fresh concrete does not crack or burst the walls of the pile. Plugs also may be placed by placing a course of gravel, then grout-injecting it. In such cases the walls of the pile should be cleaned by jetting prior to placing the concrete to ensure bond.

PROBLEMS OF INSTALLATION AND SOLUTIONS

The installation of high-capacity piling, involving the use of unusual and large equipment and the necessity to penetrate deeply through firm strata, has very naturally been accompanied with problems. The individual problems deserve careful analysis; however, within the scope of this paper all that can be done is to call attention briefly to some of the problems that have arisen and to note the corrective or preventive actions needed. On occasion these problems have been serious to the designer or contractor or both, but in general they have been successfully overcome and they can, with foresight, be alleviated for future installations. This can only be done, however, when both the design engineer and the construction contractor work together. The following are among these problems.

1. Steel H-piles driven to soft rock show extreme variations in penetration into the rock, making it difficult to determine lengths. A displacement type of pile, such as a pipe pile or precast concrete pile, would mobilize the supporting capacity of the rock with a shorter and more uniform penetration. If H-piles are to be used, radical length variations should be anticipated and the piles brought to the site purposefully long. After being driven to the required indicated bearing (blows per inch resistance), they can be cut off and the cutoff top section respliced for subsequent use.

2. After drilled-in caissons are seated and the core excavated, sand runs in under the tip during drilling of the socket. This generally requires that the pile be reseated with the hammer once or twice to seal off the tip. The pile should be kept full of water (saltwater is even more effective), and the operation of drilling and baling tools should be controlled to prevent sudden drop in effective head at the tip during withdrawal of the tools.

3. Prestressed concrete piles fail in horizontal cracking under driving because rebound tensile stresses occur during the period when the tip of the pile has little or no resistance, i.e., during soft driving. The driving compressive wave then reflects from the tip as a tensile wave and causes cracking, usually at the upper third point. The solution includes the use of a new thick cushion block of softwood on the head of each pile to be driven. The velocity of impact of the ram should be reduced. This can be done by shortening the stroke of the hammer. The "free-end" condition should be minimized by predrilling, rather than driving, through an overlying crust into soft mud below, or by control of jetting or drilling so that the tip always has reasonably firm resistance.

4. Longitudinal cracking of prestressed concrete cylinder piles can be caused by a variety of phenomena including excessive buildup of hydrostatic head inside during jetting, wedging of soil during driving, or freezing. Spiral requirements for prestressed concrete cylinder piles have often been on the minimal side. They should be increased, throughout the length, but particularly at the head and tip. Vents of large size should be provided to enable any excess hydrostatic head to be vented. If the driving ram must work with the pile head below water, very large vents must be provided in the driving head to prevent a hydraulic ram-bursting effect. To prevent freezing in cold weather, vents well below water surface will allow water circulation. Styrofoam or wood logs have been floated inside hollow core piles to reduce excessive pressures from freezing. When prestressed concrete piles are filled with concrete, the internal head will increase very rapidly. Rate of placing must be closely controlled to the time of set to prevent bursting the pile.

5. Piles sunk by jetting and lubrication show inadequate lateral resistance. The soil has been disturbed, and the grains have been spread apart by the jetting action. To reconsolidate these, the most easily applied step, in many cases, is to consolidate the soil by the vibration and shock of continuing hammer blows. In some cases, a required number of hammer blows, e.g., 200, has been specified to aid in this reconsolidation. Another means of overcoming this problem is by grout injection of the soil surrounding the pile tip.

6. Problems with drilled-in piles involve sloughing of the walls of the socket during drilling and prior to concreting. This phenomena often occurs in serpentine and shale rocks. The basic solution is to reduce the time of exposure and to prevent air from contact with the rock by keeping the hole filled with water. The plug should be poured by tremie concrete techniques immediately after excavation.

7. For high-capacity piles, conventional means of determining bearing capacity are no longer applicable. There are, however, several ways for evaluating bearing capacity. (a) From soil mechanics study of shear, friction, and cohesion values and a knowledge of the shape and surface characteristics of the pile, a bearing value can be computed for a specific penetration. (b) Use of an adequate dynamic formula, such as the wave equation, is relatively valid if a large enough hammer is employed, but it must be interpreted in the light of soil test data. (c) Load tests may be used, although it is very difficult to find practicable means of load testing piles whose design capacity is far above conventional values. Nevertheless, load tests have been performed by reaction against dead loads and by reaction piles. A method with promise is to drill in prestressing tendons into underlying rock and to jack against these to supply the downward thrust for the test load. (d) Load tests on a scale device may also be used. In Lake Maracaibo, Heerema has jacked a small-diameter pipe ahead of the pile tip, working through a hollow core in the pile. Skin-friction and end-bearing values are determined for this and extrapolated to the pile itself.

8. Inability to achieve the required predetermined penetration is perhaps the most common and most serious problem. Hollow-core piles permit removal of the core and drilling ahead. They also permit, as an ultimate remedy, the installation of an insert pile that can be driven ahead, freed as it is from skin friction. With solid-tip piles, corrective steps are extremely difficult on piles that have already been driven to refusal. Side jetting may reduce skin friction, and a heavier hammer may provide more drivability. For subsequent piles, however, a number of effective steps may be taken, such as predrilling, increase in hammer size, and jetting or lubrication of sides.

9. To prevent damage and distortion to tips of piles when rocks or boulders are hit, the tip should be reinforced by a shoe of high-yield point steel, either pipe or box section, filled with concrete to prevent local distortion.

It is interesting to note that one or more of these problems have arisen in the first attempts at installation of several of the types of high-capacity piles referred to in the review of typical installations. At the same time, in all of these cases, solutions such as those given were found and the installations were completed satisfactorily.

EVALUATION OF PILE TYPES AND INSTALLATION METHODS

The selection of pile types for high-capacity piles must be based on structural performance, economics, and practicability. This paper is essentially a discussion of the latter. It should be interpreted in a positive sense, for the ability and ingenuity of contractors and equipment manufacturers should not be underrated.

Certain conclusions concerning practicability may be drawn. Piles that are open-ended permit the use of auxiliary techniques to overcome obstacles such as boulders, harder strata than anticipated, rock, and debris. Piles with inherent rigidity such as heavy-walled pipe piles and prestressed concrete piles suffer less deformation upon encountering obstacles. The piles having greater section modulus such as cylinder piles and caisson piles have the ability to give lateral support in both bending and shear provided that the installation methods adopted do not weaken the soil. There are a number of steps available by which bearing capacity and lateral support may be restored or increased.

Obsolete formulas for pile-driving should be revised in the light of new field data and the information obtained from the wave equation theory. Where high-capacity piles are involved, specifications must either require performance or else specify in detail the equipment and methods to be employed, but not both. Furthermore, limitations and restrictions should be imposed on techniques that may reduce the carrying capacity of the soil. Installation of high-capacity piles requires an integration of the efforts of the design engineer and the constructor if the best results are to be obtained.

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Pile Load Test by Impact Driving

G. G. GOBLE and FRANK RAUSCHE, Case Western Reserve University

The use of measurements made during impact driving to predict static pile capacity is a widely used concept. The results are sometimes not satisfactory because of a lack of knowledge of hammer energy, poor estimates of losses in cushions, inaccurate set measurements, substantial strength change after the end of driving, and other factors. A method having a different basis has been proposed for predicting static capacity from dynamic measurements made by impact driving. By applying this method to measurements made after a setup period, the difficulties mentioned are avoided. This method has been applied to 25 piles that were also statically load-tested. The average difference between the static capacity obtained from a constant rate of penetration test and that predicted from dynamic measurements was 10 percent. A wide variety of soil conditions and hammer types is represented in the data. All piles were of hollow steel pipes. The use of this system for routine measurements is not feasible if reliance must be placed on the analysis of records taken on high-speed oscillographs because a substantial time lag occurs between data acquisition and analysis. In addition, equipment of this type is hardly satisfactory for routine field use by personnel having modest backgrounds in electronics. To avoid these problems, a special purpose computer was designed and constructed to perform the necessary computations in the field and to display the results. With this device, the necessary calculations for capacity prediction are made in real time and the results are displayed. Thus, a number of consecutive blows can be recorded to improve the accuracy of the prediction. This device should improve the reliability of pile test data because it is realistic to obtain load test data by driving a number of "test" piles and obtaining dynamic predictions from all of them.

•THE USE OF MEASUREMENTS made during impact driving to predict static pile strength is a widely used concept. Many pile-driving formulas use energy considerations to predict bearing capacity. The results are sometimes not satisfactory because of a lack of knowledge of hammer energy, poor estimates of losses in cushions, inaccurate set measurements, substantial strength change after driving, assumption of a resistance that is constant during the blow, and other factors. A method having a different basis has been proposed for predicting static capacity from dynamic measurements made during impact driving (1, 2, 3). By applying this method to measurements made after a setup period, the difficulties mentioned are avoided. This procedure has been applied to 25 piles that were also statically load-tested. The difference between the static capacity obtained from a constant rate of penetration test and that predicted from dynamic measurements is much smaller than that commonly observed by using pile formulas. A wide variety of soil conditions and hammer types are represented.

A special purpose, electronic computer has been designed and constructed to automatically perform the necessary computations in the field and display the results. With this device, a predicted capacity is displayed within 2 milliseconds after the end of the hammer blow. Thus, a number of consecutive blows can be recorded to improve the

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accuracy of the prediction. This device should increase the reliability of pile test data because it is realistic to obtain test data by driving a number of "test" piles and obtaining dynamic predictions from all of them.

Procedures are now being developed jointly with the Ohio Department of Highways and the researchers at Case Western Reserve University to begin implementing these methods and equipment into Ohio construction practice. It is anticipated that the researchers will gradually decrease their involvement with the routine application of the technique.

DATA ACQUISITION, EVALUATION, AND CORRELATION

A simple force-balance method has been proposed to relate dynamic measurements to static capacity. The pile is assumed to be a rigid body struck by a time-varying hammer force. Motion of the pile is resisted by a force, R, given by the expression

$$R(t) = R_0 + R_1 v + R_2 v^2 + R_3 v^3 + \ldots \qquad (1)$$

where v is the velocity of the pile, and R_0, R_1, R_2, ... are constants. Thus, R_0 represent the static capacity. Under the action of these forces at the instant of zero velocity and by use of Newton's Second Law, the resistance is found to be

$$R_0 = F(t_0) - ma(t_0) \qquad (2)$$

where m is the mass of the pile, $a(t_0)$ is the acceleration at time, t_0, when the velocity is zero. and $F(t_0)$ is the force at the top of the pile at the same time.

Subsequent studies have shown that the reliability of the capacity predictions can be improved by averaging the acceleration over some time increment around the zero velocity time to avoid extreme acceleration values arising from elastic waves in the pile. In the following discussion, Eq. 2 will be referred to as the Phase I method and the same equation containing an averaged acceleration will be referred to as the Phase II method. Theoretical studies, recently completed but not yet reported, have shown why these simple approaches provide such good results and have given some indication of the limitations to be expected. These simple methods will be less reliable for application in highly cohesive soils and for very short piles. A Phase III method has been developed that shows promise in providing good capacity predictions for all soil types. It will not be discussed here.

The application of Eq. 2 requires the measurement of force and acceleration at the top of the pile. Velocity can be obtained by integration of the acceleration record. Developments in electronic instrumentation and transducers in the past few years make it possible to obtain these measurements on a routine basis. Equipment for making the measurements was assembled and extensively tested. A high reliability was attained for obtaining successful records under the normal rather difficult field conditions. Force measurements were made during the early project phases by using resistance strain gages attached to the pile and later with transducers either on top of the pile or attached to the pile. Accelerations were measured with piezoelectric accelerometers attached to the pile a short distance below its top. Force and acceleration were recorded continuously on a high-speed oscillograph.

Data were obtained from full-scale piles driven for static load testing on highway bridge projects. After completion of the usual Ohio Highway Department load test, project personnel performed an additional load test by using a constant rate of penetration. In this test, the goal was to reach ultimate bearing capacity, as a basis of correlation with dynamic results. The load test was then removed, and dynamic measurements were made during several hammer blows sufficient to obtain a permanent set of the pile. This procedure has become quite routine. At present, the instrumentation is assembled, and all readings are taken by only 2 people from the research project. For the past 3 years, successful records have been obtained from every attempt. It should be noted that in all cases the contractors have donated the time and services required to redrive the pile. Thus, the data have been obtained at a very low cost.

The data were analyzed by manually transforming them to digital form on punched cards. The computations outlined earlier were made by a digital computer, and drawings of the results were obtained automatically from a computer-controlled plotter. A sample result is shown in Figure 1.

The dynamic prediction is expected to correlate with ultimate capacity, and for this reason the constant penetration load test was performed because the standard Ohio Department of Highways load test does not go to ultimate load.

A comparison of dynamic predictions with static measurements is given in Table 1. Additional results have been obtained that are not reported because the procedures outlined were not followed exactly. These results are reported in another paper (3). Some comments are appropriate. Piles 10 through 15 and 20 through 23 were special test piles driven under research project control and having additional special instrumentation. Static load tests were performed, and the dynamic predictions make use of only the usual measurements.

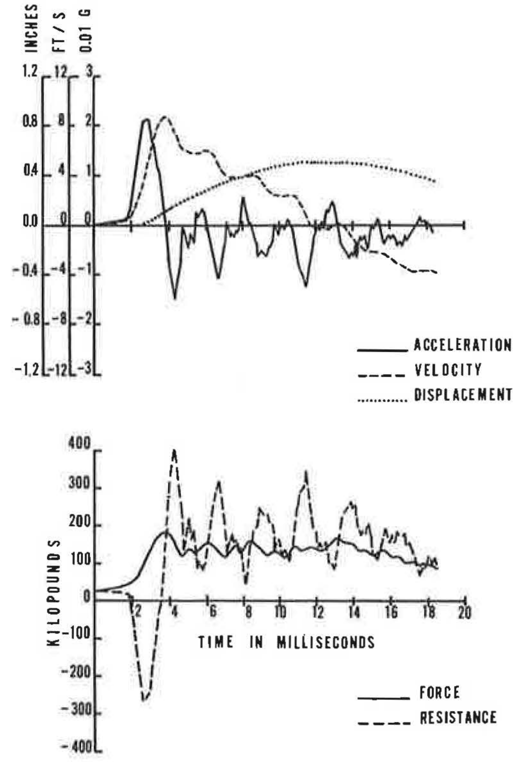

Figure 1. Dynamic results of pile 12.

Substantial differences between static measurements and the Phase II predictions exist only for 2 piles, 22 and 24. Pile 22 had a very low capacity so that a 21-ton difference results in a large percentage. Furthermore, studies conducted on small-scale piles and recent analytical studies have raised questions about the Phase II method on piles in highly cohesive soils. An application of the preliminary Phase III procedure showed better correlation for the piles driven in fine-grained soils. The agreement on pile 24 was substantially better, and it was almost exact on pile 22. However, it should be emphasized that this was only a very early application of Phase III, and considerable further study is required.

Piles 4 through 9 were driven into a material having a static behavior that does not exhibit a sharp break in the load-set curve. Considerable "strain hardening" appears at large set values. The current analytical studies show promise in providing a solution to this difficulty.

Pile 5 was a steel pipe pile that had been filled with concrete prior to making dynamic measurements. Thus, it had a large mass typical of all precast concrete piles. It is interesting to note that the agreement with static test results was good. These were the only data obtained from "high mass" piles.

The average difference between the static ultimate strength and the dynamic prediction was 29 percent for Phase I and 17 percent for Phase II. If piles 22 and 24 are dropped, the differences are 22 and 10 percent for Phase I and Phase II respectively.

SPECIAL PURPOSE COMPUTER

The instrumentation system outlined earlier is satisfactory for use by experienced personnel. However, it hardly represents an acceptable system for routine use by engineers whose primary concerns are with other activities. Effort has been devoted, therefore, to the design and development of a special electronic computer and the necessary associated transducers.

TABLE 1

COMPARISON OF STATIC ULTIMATE CAPACITY WITH PHASE I AND PHASE II DYNAMIC PREDICTIONS

Pile	Length (ft)	Soil	Static Resistance (tons)	Phase I Dynamic Prediction (tons)	Difference (percent)	Phase II Dynamic Prediction (tons)	Difference (percent)
1	76.5	Gray silt and clay	126	139	+13	131	+4
2	74.0	Medium coarse sand	148	200	+26	174	+18
3	78.0	Sand silt and clay	122	141	+14	139	+14
4	59.0	Gravelly sand	96	95	-1	94	-2
5	76.0	Gravelly sand	106	110	+4	106	0
6	71.0	Gravelly sand	95	87	-12	91	-4
7	71.0	Gravelly sand	100	72	-28	73	-27
8	77.0	Gravelly sand	99	92	-7	92	-7
9	84.0	Gravelly sand	125	101	-20	105	-16
10	31.5	Silt and sand	52	61	+17	60	+15
11	31.5	Silt and sand	57	58	+2	63	+11
12	50.0	Silt and sand	112	139	+24	97	+13
13	50.0	Silt and sand	119	142	+19	113	-5
14	59.0	Silt and sand	102	144	+41	109	+7
15	59.0	Silt and sand	121	164	+36	111	-8
16	71.0	Coarse gravel	95	126	+33	95	0
17	58.0	Gravelly sand	113	138	+22	114	+1
18	41.0	Gravel and sand	105	158	+52	101	-4
19	52.0	Clayey silt	111	164	+48	130	+17
20	50.0	Gray silt and clay	35	33	-6	47	+34
21	50.0	Gray silt and clay	48	37	+23	53	+10
22	60.0	Gray silt and clay	22	43	+96	43	+96
23	60.0	Gray silt and clay	43	50	+16	47	+9
24	56.0	Gray silt and clay	47	100	+113	88	+87
25	78.0	Gray silt and clay	88	135	+54	78	-11

The piezoelectric accelerometers used during most of the project were quite satisfactory except for one problem. The signal coming from the accelerometer requires considerable amplification before it can be recorded. Therefore, there is a problem with lead wire noise. Recently, accelerometers of this type with an amplifier built into the transducer have become available. Their output signal is large enough so that long lead wires can be used without difficulty. These devices have made the acceleration measurements much easier. The accelerometers were attached directly to a small steel block in a semi-permanent fashion. This total system was attached directly to the pile wall by drilling and tapping a hole in the pile wall and bolting the steel base directly to the wall. This setup is shown in Figure 2.

During most of the time that the project has been active, force measurements were obtained by attaching resistance strain gages to the pile near the top. The Ohio Department of Highways usually uses steel pipes for friction piles. Two strain gages were attached to the exterior pile walls on a diameter to cancel out gross bending of the pile. Force was then determined by using the nominal area. This system gives very reliable force measurements but is unsatisfactory for routine application because it requires the attachment of at least 2 resistance strain gages to the pile under field conditions. The use of Eastman 910 contact cement results in a gage attachment that can be used immediately. However, the process of attachment is, at best, a tedious process and unsatisfactory in routine application.

Two transducers were developed for force measurement. One system uses a piece of pipe of the same diameter as the pile with strain gages attached to the pipe. One of these devices is shown in Figure 3. A plate is then welded to the top of the pile, and the transducer is bolted to the plate. An adapter attached to the top of the transducer accepts the driving system in the same manner as does the pile. The second transducer

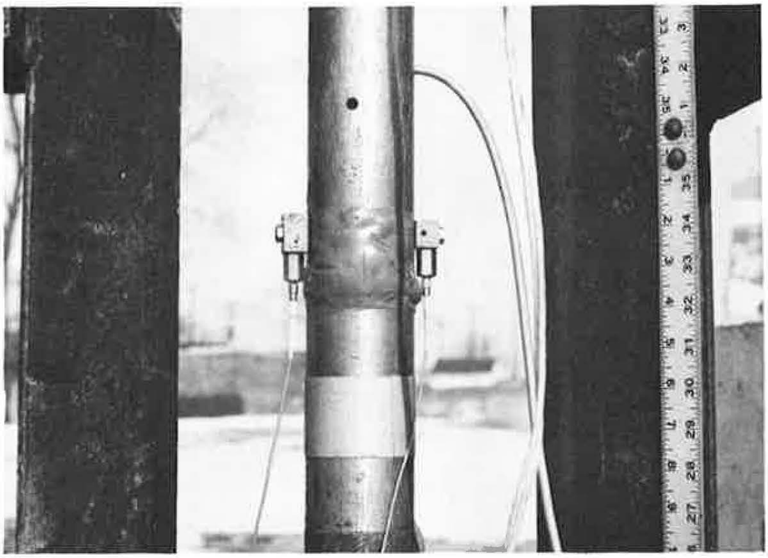

Figure 2. Accelerometer attachment.

could best be described as a strain transducer. A small curved strap was designed to be quite flexible in the longitudinal direction. Strain gages were attached to the strap in points of stress concentration to provide a large strain output from the transducer. This bears a linear relationship to the strain in the material to which it is attached and can be calibrated. The transducer is shown in Figure 4 attached to a 3-inch diameter pipe. On the left is the transducer, and on the right is the transducer with the template used for supporting it during the attachment operation. In actual use, 2 transducers are applied, one on each side of the pile. A typical setup of the instrumentation attached to the pile is shown in Figure 5.

Both transducers provided satisfactory records for this application. The first device has the advantage of providing an output that has been calibrated to force. It is, however, heavier and bulkier than the second device, and, because it has substantial mass, the dynamic behavior is affected to some degree. The second device is light and easily attached, but, because it is calibrated on strain and thus dependent on a knowledge of pile area, it will not be as accurate as the first device.

Figure 3. Force transducer.

The strain and acceleration signals, after appropriate conditioning, were recorded on a high-speed oscillograph. The resulting records were then examined and digitized manually for subsequent automatic analysis. A more rapid, simpler data processing system is necessary. Therefore, a special purpose electronic computer was constructed to perform the computations of Eq. 2 and provide a readout in real time. It is shown in Figure 6. In use, the device is calibrated to a particular pile by the operator. (The accelerometer and force transducer calibrations and the pile mass

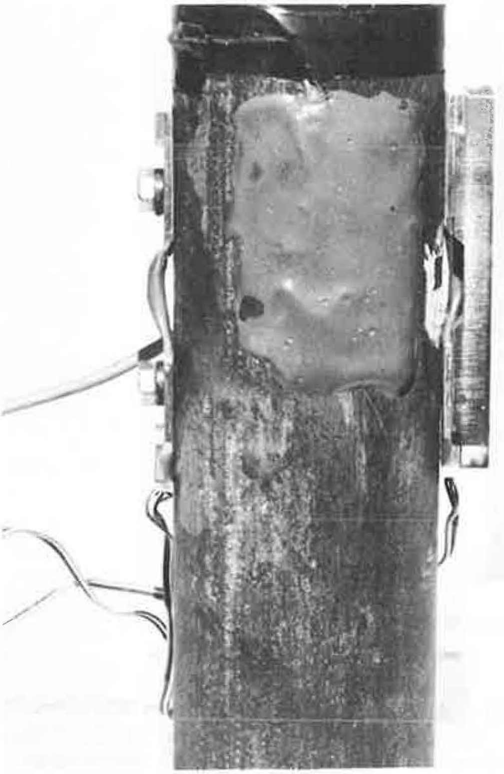

Figure 4. Strain transducer.

Figure 5. Force transducer and accelerometer in driving system.

must be introduced.) The computer then provides a visual output of 3 digits on a nixie tube display. All blows or selected blows can be recorded.

The operational characteristics of this system of computer and transducer indicate that it could be used by the construction engineer with only brief training.

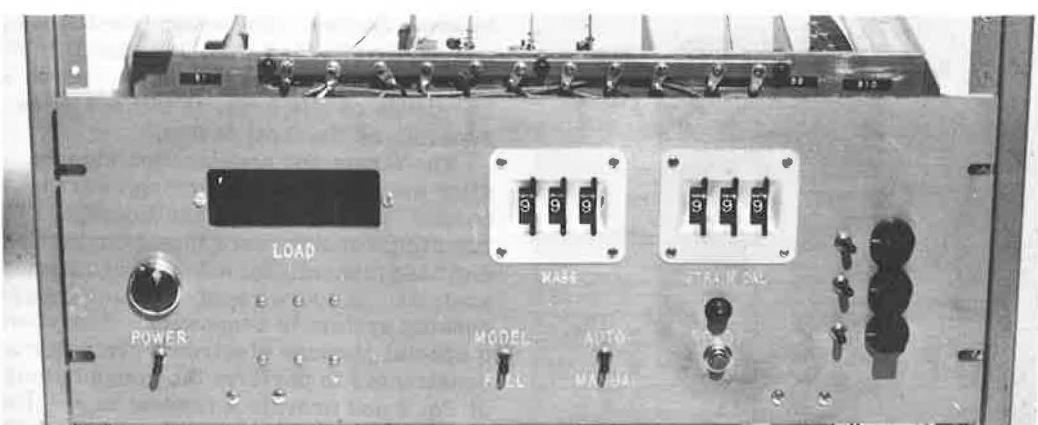

Figure 6. Operator control panel of special purpose computer.

PLANNED APPLICATION IN FOUNDATION CONSTRUCTION CONTROL

It is unrealistic to expect that a drastic change in pile-driving control, such as the system outlined, can be introduced without an extended period of trial. In the first place, only a limited amount of correlation with static load tests has been obtained. Furthermore, all data to date have been gathered by research project personnel. The first step toward routine use will be to make the Phase I computer available to the Ohio Department of Highways. Department personnel plan to use it for continued correlation with static test piles and the driving formula. If correlation is poor, project personnel will be available to obtain dynamic records for more thorough study. In many smaller jobs, the requirement of a load test pile is not considered economical. Thus, occasionally, if engineers are surprised by the pile lengths obtained from a driving formula, the computer would be used to provide further information on which to base a decision.

A Phase II computer will also be constructed, tested, and introduced into use as with Phase I. Present data indicate that this device should be satisfactory for all conditions except perhaps in uniform highly cohesive soil deposits. A totally different system may be necessary for application of the Phase III procedure.

It is hoped that it will be possible to obtain substantial reductions in foundation costs as a result of the use of this method. Static load tests will be less frequently necessary, but probably more important will be the use of the additional information to reduce the rather large margins between indicated bearing and capacity required by design. The economic use of multiple test piles will further add to the reliability of test results.

Additional data and study are required for piles having a large mass and for piles driven in highly cohesive soils.

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