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Design and Construction of Driven Pile Foundations

Workshop Manual - Volume II

National Highway Institute

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16. Abstract

This manual is intended to serve a dual purpose, first as a participant's manual for the FHWA's National Highway Institute courses on driven pile foundations and secondly as FHWA's primary reference of recommended practice for driven pile foundations.

The Design and Construction of Driven Pile Foundations manual is directed to geotechnical, structural, and construction engineers involved in the design and construction of pile supported structures. The manual is intended to serve as a practical reference on driven pile foundations. Volume I of the manual addresses design aspects including subsurface exploration, laboratory testing, static analyses, as well as specification and foundation report preparation. Volume II covers construction aspects including dynamic formulas, wave equation analyses, dynamic testing, static load testing, Statnamic testing, the Osterberg cell, as well as pile driving equipment, pile accessories, and pile installation inspection. Step by step procedures, workshop problems and solutions are provided to demonstrate use of the manual material.

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U.S. - SI Conversion Factors

From English	To SI	Multiply by	Quantity	From SI	To English	Multiply by
ft	m	0.3048	Length	m	ft	3.2808
inch	mm	25.40		mm	inch	0.039
ft^2	m^2	0.0929	Area	m^2	ft^2	10.764
$inch^2$	mm^2	645.2		mm^2	in^2	0.0015
ft^3	m^3	0.028	Volume	m^3	ft^3	35.714
$inch^3$	mm^3	16387		mm^3	$inch^3$	$61\text{x}10^{-6}$
ft^4	m^4	0.0086	Second Moment of Area	m^4	ft^4	115.856
$inch^4$	mm^4	416231		mm^4	$inch^4$	$2\text{x}10^{-6}$
lbm	kg	0.4536	Mass	kg	lbm	2.2046
lbm/ft^3	kg/m^3	16.02	Mass Density	kg/m^3	lbm/ft^3	0.062
lb	N	4.448	Force	N	lb	0.2248
kip	kN	4.448		kN	kip	0.2248
lbs/ft	N/m	14.59	Force/Unit-Length	N/m	lbs/ft	0.0685
kips/ft	kN/m	14.59		kN/m	kips/ft	0.0685
lbs/in^2	kPa	6.895	Force/Unit-Area; Stress; Pressure; Elastic Mod.	kPa	lbs/in^2	0.145
$kips/in^2$	MPa	6.895		MPa	$kips/in^2$	0.145
lbs/ft^2	Pa	47.88		Pa	lbs/ft^2	0.021
$kips/ft^2$	kPa	47.88		kPa	$kips/ft^2$	0.021

U.S. - SI Conversion Factors (continued)

From	To	Multiply by	Quantity	From	To	Multiply by
English	SI			SI	English	
lbs/ft^3	N/m^3	157.1	Force/Unit-	N/m^3	lbs/ft^3	0.0064
$kips/ft^3$	kN/m^3	157.1	Volume	kN/m^3	$kips/ft^3$	0.0064
lb-inch	N-mm	112.98	Moment; or	N-mm	lb-inch	0.0089
kip-inch	kN-mm	112.98	Energy	kN-mm	kip-inch	0.0089
lb-ft	N-m	1.356		N-m	lb-ft	0.7375
kip-ft	kN-m	1.356		kN-m	kip-ft	0.7375
ft-lb	Joule	1.356		Joule	ft-lb	0.7375
ft-kip	kJoule	1.356		kJoule	ft-kip	0.7375
s/ft	s/m	3.2808	Damping	s/m	s/ft	0.3048
blows/ft	blows/m	3.2808	Blow count	blows/m	blows/ft	0.3048

PREFACE

Engineers and contractors have been designing and installing pile foundations for many years. During the past three decades this industry has experienced several major improvements including newer and more accurate methods of predicting capacities, highly specialized and sophisticated equipment for pile driving, and improved methods of construction control.

In order to take advantage of these new developments, the FHWA developed a manual in connection with Demonstration Project No. 66, Design and Construction of Driven Pile Foundations. The primary purpose of the Manual was to support educational programs conducted by FHWA for transportation agencies. These programs consisted of (1) a workshop for geotechnical, structural, and construction engineers, and (2) field demonstrations of static and dynamic load testing equipment. Technical assistance on construction projects in areas covered by this Demonstration Project was provided to transportation agencies on request. A second purpose of equal importance was to serve as the FHWA's standard reference for highway projects involving driven pile foundations.

The original Manual was written by Suneel N. Vanikar with review and comment from Messrs. Ronald Chassie, Jerry DiMaggio, and Richard Cheney.

After a decade of use it was necessary that the Manual be updated and modified to include new developments that had taken place in the intervening years and to take advantage of the experience gained in using the Manual in the many workshops that were presented by Demonstration Project 66. The new version of the Manual was prepared by Goble Rausche Likins and Associates, Inc. under contract with the FHWA.

The Manual is presented in two volumes. Volume I addresses design aspects and Volume II presents topics related to driven pile installation, monitoring, and inspection.

The new Manual is intended to serve a dual purpose. First, as a workshop participant's manual for the FHWA's National Highway Institute Courses on Driven Pile Foundations. Similar to the earlier demonstration manual, this document is also intended to serve as FHWA's primary reference of recommended practice for driven pile foundations.

Upon completion of NHI Course 13221, participants will be able to:

1. Describe methods of pile foundation design.

2. Discuss driven pile construction materials and installation equipment.

3. Describe the timing and scope of the involvement of foundation specialists as a project evolves from concept through completion.

4. Perform a foundation economic analysis and determine the need for a driven pile foundation.

5. Recognize the pile type selection process and the advantages and disadvantages of common driven pile types.

6. Compute single and group capacities of driven piles to resist compression, tension and lateral loads.

7. Identify when and how dynamic formulas, wave equation analyses, dynamic pile testing and static load testing should be used on a project.

8. Discuss the components of structural foundation reports and controlling issues of specifications and contracting documents as related to a successful construction project.

9. Describe the concept and project influence of driveability, pile refusal, minimum and estimated pile toe elevations, soil setup and relaxation.

Upon completion of NHI Course 13222, participants will be able to:

1. Describe methods of driven pile construction monitoring and inspection practices and procedures.

2. Discuss pertinent driven pile specification and contract document issues.

3. Describe wave equation, dynamic testing and static testing results in terms of their application and interpretation on construction projects.

4. Identify the basic components and differences between various pile driving systems, associated installation equipment, pile splices and pile toe attachments.

5. Interpret a set of driven pile plan details and specifications.

6. Inspect a drive pile project with knowledge and confidence.

The authors' recognize the efforts of the project technical manager, Mr. Jerry DiMaggio, FHWA Senior Geotechnical Engineer, who provided invaluable guidance and input for the new manual.

The authors' also acknowledge the additional contributions of the following technical review panel members listed in alphabetical order:

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LIST OF SYMBOLS

A - Pile cross sectional area.

A_g - Pile area at gage location.

A_{np} - Net area of piston.

a - Acceleration.

a_m - Measured acceleration.

b - Pile diameter.

C - Wave speed of pile material.

C_4 - Statnamic damping constant.

E - Modulus of elasticity of pile material.

E_p - Energy transferred to pile.

E_d - Dynamic stiffness.

E_r - Manufacturers rated hammer energy.

F - Force.

F_a - Statnamic inertia force.

F_p - Statnamic pore water pressure force.

F_u - Statnamic static soil resistance force.

F_v - Statnamic dynamic soil resistance force.

F_{stn} - Statnamic induced force.

$F(t)$ - Force measured at gage location.

LIST OF SYMBOLS (continued)

Δf	-	Dominant frequency.
h	-	Hammer stroke.
J	-	Soil damping factor.
J_c	-	Dimensionless Case damping factor.
L	-	Total pile length.
ΔL	-	Length of pile between two measuring points under no load conditions.
L_g	-	Pile length below gage location.
m	-	Mass.
N_b	-	The number of hammer blows per 25 mm.
p_h	-	Pressure at hammer.
Q	-	Load.
Q_a	-	Allowable design load of a pile.
Q_{avg}	-	Average load in the pile.
Q_f	-	Failure load.
Q_h	-	Applied pile head load.
Q_o	-	Osterberg cell load.
Q_r	-	Load from reaction system.
Q_u	-	Ultimate bearing capacity of a pile.
q	-	Soil quake.

LIST OF SYMBOLS (continued)

R	-	Soil resistance.
R_1	-	Deflection reading at upper of two measuring points.
R_2	-	Deflection reading at lower of two measuring points.
R_s	-	Ultimate pile shaft resistance.
R_t	-	Ultimate pile toe resistance.
R_u	-	Ultimate soil resistance.
s_b	-	Set per blow.
s_f	-	Settlement at failure.
t_1	-	Time of initial impact.
t_2	-	Time of reflection of initial impact from pile toe (t_1+2L/c).
t_4	-	Time at Statnamic stage 4.
t_{umax}	-	Time of maximum displacement.
U	-	Displacement.
$V(t)$	-	Velocity measured at gage location.
v_i	-	Impact velocity.
W	-	Ram weight.
Δ	-	Elastic compression.
ϵ	-	Strain.
ϕ	-	Angle of internal friction of soil.

15. INTRODUCTION TO CONSTRUCTION MONITORING

Volume II of the Manual on Design and Construction of Driven Pile Foundations focuses on the construction aspects of driven pile foundations. Following this introductory chapter are chapters on pile capacity evaluation using dynamic formulas (Chapter 16), wave equation analysis (Chapter 17), dynamic testing and analysis (Chapter 18), static load testing (Chapter 19), the Osterberg load cell device (Chapter 20) and the Statnamic method (Chapter 21). These chapters on pile testing methods are followed by chapters detailing pile driving equipment (Chapter 22), driven pile accessories (Chapter 23), and pile inspection (Chapter 24).

15.1 THE ROLE OF CONSTRUCTION MONITORING

Proper pile installation is as important as rational pile design in order to obtain a cost effective and safe end product. Driven piles must develop the required capacity without sustaining structural damage during installation. Construction control of driven piles is much more difficult than for spread footings where the footing excavation and footing construction can be visually observed to assure quality. Since piles cannot be seen after their installation, direct quality control of the finished product is impossible. Therefore substantial control must be exercised over peripheral operations leading to the piles' placement within the foundation.

It is essential that any pile installation limitations be considered during the project design stage so that the piles shown on the plans can be installed as designed. For example, consideration should be given to how new construction may affect existing structures and how limitations on construction equipment access, size, or operation area may dictate the pile type that can be most cost effectively installed.

Construction monitoring should be exercised in three areas: pile materials, installation equipment, and the estimation of static load capacity. These areas are interrelated since changes in one affects the others. Table 15-1 highlights the items to be included in the plans and specifications that are the design engineer's responsibility, and the items to be checked for quality assurance that are the construction engineer's responsibility.

TABLE 15-1 RESPONSIBILITIES OF DESIGN AND CONSTRUCTION ENGINEERS		
Item	Design Engineer's Responsibilities	Construction Engineer's Responsibilities
Pile Details	Include in plans and specifications: a. Material and strength: concrete, steel, or timber. b. Cross section: diameter, tapered or straight, and wall thickness. c. Special coatings for corrosion or downdrag. d. Splices, toe protection, *etc.* e. Estimated pile length. f. Pile design load and ultimate capacity. g. Allowable driving stresses.	Quality control testing or certification of materials.
Soils Data	Include in plans and specifications: a. Subsurface profile. b. Soil resistance to be overcome to reach estimated length. c. Minimum pile penetration requirements. d. Special notes: boulders, artesian pressure, buried obstructions, time delays for embankment fills, *etc*.	Report major discrepancies in soil profile to the designer.
Installation	Include in plans and specifications: a. Method of hammer approval. b. Method of determining ultimate pile capacity. c. Compression, tension, and lateral load test requirements (as needed) including specification for tests and the method of interpretation of test results. d. Dynamic testing requirements (as needed). e. Special notes: spudding, predrilling, jetting, set-up period, *etc*.	a. Confirm that the hammer and driving system components agree with the contractor's approved submittal. b. Confirm that the hammer is maintained in good working order and the hammer and pile cushions are replaced regularly. c. Determination of the final pile length from driving resistance, estimated lengths and subsurface conditions. d. Pile driving stress control. e. Conduct pile load tests. f. Documentation of field operations. g. Ensure quality control of pile splices, coatings, alignment and driving equipment.

15.2 SELECTION OF FACTOR OF SAFETY

In the design stage, a design load is selected for the pile section as a result of static analyses and consideration of the allowable stresses in the pile material. A factor of safety is applied to the design load depending upon the confidence in the static analysis method, the quality of the subsurface exploration program, and the construction control method specified. Static analyses yield the estimated pile length, based on the penetration depth in suitable soils required to develop the design load times the factor of safety. Soil resistance from unsuitable support layers, or layers subject to scour, are not included in determining the required pile penetration depth.

During construction, the ultimate pile capacity to be obtained is the sum of the design load, times a factor of safety, plus the soil resistance from unsuitable layers not counted on for long term support or subject to scour. The plans and specifications should state the ultimate pile capacity to be obtained in conjunction with the construction control method to be used for determination of the ultimate pile capacity.

The factor of safety used should be based on the quality of the subsurface exploration information and the construction control method used for capacity verification. There are several capacity verification methods that can be used for construction control which are described in subsequent chapters. The factor of safety applied to the design load should increase with the increasing unreliability of the method used for determining ultimate pile capacity during construction. The recommended factor of safety on the design load for various construction control methods from Cheney and Chassie (1993) and/or AASHTO (1992) are shown below.

Construction Control Method	Recommended Factor of Safety
Static Load Test	2.00
Dynamic Measurements and Analysis coupled with Wave Equation Analysis	2.25
Indicator Piles coupled with Wave Equation Analysis	2.50
Wave Equation Analysis	2.75
Gates Dynamic Formula	3.50
Engineering News Formula	Not a Recommended Method

For example, consider a pile with a design load of 700 kN. If no unsuitable soil layers exist, and a static load test will be performed for construction control, then an ultimate pile capacity of 1400 kN would be specified. For this same example, an ultimate pile capacity of 1925 kN would be required when construction control is by wave equation analysis.

If unsuitable or scour susceptible layers exist, the resistance from these layers should be added to the required ultimate pile capacity. For a pile with a design load of 700 kN in a soil profile with 250 kN of soil resistance from unsuitable soils, or soils subject to scour, an ultimate pile capacity of 1650 kN would be required for construction control with a static load test. For this case, an ultimate pile capacity of 2175 kN would be specified for construction control by wave equation analysis.

15.3 COMMUNICATION

Proper construction monitoring of pile driving requires good communication between design and construction engineers. Such communication cannot always follow traditional lines and still be effective. Information is needed in a short time to minimize expensive contractor down time or to prevent pile driving from continuing in an unacceptable fashion.

Good communication should begin with a pre-construction meeting of the foundation designer and the construction engineer on all projects with significant piling items. Prior to the meeting, the construction engineer should review the project foundation report and be fully aware of any construction concerns. At the meeting, the designer should briefly explain the design and point out uncertainties and potential problem areas. The primary objective of this meeting is to establish a direct line of communication.

During construction, the construction engineer should initiate communication with the designer if proposed pile installation methods or results differ from the plans and specifications. The designer should advise the construction engineer on the design aspects of the field problems. The construction engineer should provide feedback on construction monitoring data to the design engineer.

The ultimate decision making authority should follow along the traditional lines of communication established by the state transportation agency. However, informal interaction between design offices and the field should be encouraged and will simplify and expedite decisions.

REFERENCES

American Association of State Highway and Transportation Officials [AASHTO], (1992). Standard Specifications for Highway Bridges. Fifteenth Edition, AASHTO Highway Subcommittee on Bridges and Structures, Washington, D.C., 686.

Cheney, R.S. and Chassie, R.G. (1993). Soils and Foundations Workshop Manual. Second Edition, Publication No. FHWA-HI-88-009, U.S. Department of Transportation, Federal Highway Administration and the National Highway Institute, Washington, D.C., 395.

16. DYNAMIC FORMULAS FOR STATIC CAPACITY DETERMINATION

Ever since engineers began using piles to support structures, they have attempted to find rational methods for determining the pile's load carrying capacity. Methods for predicting capacities were proposed, using pile penetration observations obtained during driving. The only realistic measurement that could be obtained during driving was the pile set per blow. Thus energy concepts equating the kinetic energy of the hammer to the resistance on the pile as it penetrates the soil were developed to determine pile capacity. In equation form this can be expressed as:

$$Wh = Rs_b$$

Where:
- W = Ram weight.
- h = Ram stroke.
- R = Soil resistance.
- s_b = Set per blow.

These types of expressions are known as dynamic formulas. Because of their simplicity, dynamic formulas have been widely used for many years. More comprehensive dynamic formulas include consideration of pile weight, energy losses in drive system components, and other factors. Whether simple or more comprehensive dynamic formulas are used, pile capacities determined from dynamic formulas have shown poor correlations and wide scatter when statistically compared with static load test result. <u>Therefore, except where well supported empirical correlations under a given set of physical and geological conditions are available, dynamic formulas should not be used</u>.

16.1 ACCURACY OF DYNAMIC FORMULAS

Wellington proposed the popular Engineering News formula in 1893. It was developed for evaluating the capacity of timber piles driven primarily with drop hammers in sands. Concrete and steel piles were unknown at that time as were many of the pile hammer types and sizes used today. Therefore, it should be of little surprise that the formula performs poorly in predicted capacities of modern pile foundations.

The inadequacies of dynamic formulas have been known for a long time. In 1941, an ASCE committee on pile foundations assembled the results of numerous pile load tests along with the predicted capacities from several dynamic formulas, including the Engineering News, Hiley, and Pacific Coast formulas. The mean failure load of the load test data base was 91 tons. After reviewing the data base, Peck (1942) proposed that a new and simple dynamic formula could be used that stated the capacity of every pile was 91 tons. Peck concluded that the use of this new formula would result in a prediction statistically closer to the actual pile capacity than that obtained by using any of the dynamic formulas contained in the 1941 study.

More recently, Chellis (1961) noted that the actual factor of safety obtained by using the Engineering News formula varied from as low as ½ to as high as 16. Sowers (1979) reported that the safety factor from the Engineering News formula varied from as low as ⅔ to as high as 20. Fragasny *et al.* (1988) in the Washington State DOT study entitled "Comparison of Methods for Estimating Pile Capacity" found that the Hiley, Gates, Janbu, and Pacific Coast Uniform Building code formulas all provide relatively more dependable results than the Engineering News formula. Unfortunately, many transportation departments continue to use the Engineering News formula, which also remains the dynamic formula contained in current AASHTO Standard Specifications for Highway Bridges (1994).

As part of a recent FHWA research project, Rausche *et al.* (1996) compiled a data base of static load test piles that included pile capacity predictions using the FHWA recommended static analysis methods, preconstruction and refined wave equations, as well as dynamic measurements coupled with CAPWAP analysis. The reliability of the various capacity prediction methods were then compared with the results of the static loading tests. The results of these comparisons are presented in Figure 16.1 in the form of probability density function curves versus the ratio of predicted load over the static load test result. The mean values and coefficients of variation for the methods studied are presented in Table 16-1.

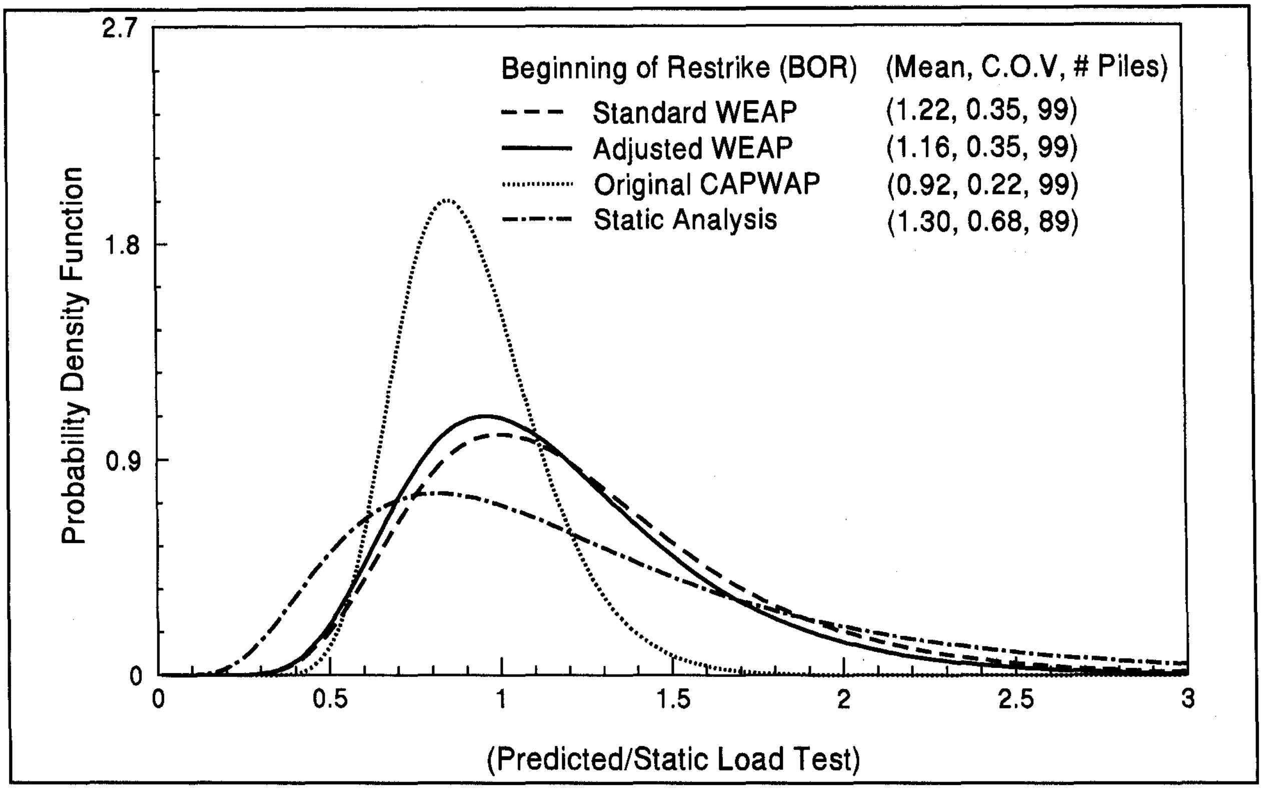


Figure 16.1 Log Normal Probability Density Function for four Capacity Prediction Methods (after Rausche *et al.* 1996)

TABLE 16-1 MEAN VALUES AND COEFFICIENTS OF VARIATION FOR VARIOUS METHODS

Prediction Method	Status	Mean	C.O.V.	# Piles
Standard WEAP*	BOR	1.22	0.35	99
Hammer Performance Adjusted WEAP*	BOR	1.16	0.35	99
CAPWAP*	BOR	0.92	0.22	99
Static Analysis*	-	1.30	0.68	89
Engineering News Formula	EOD	1.22	0.74	139
Engineering News Formula	BOR	1.89	0.46	122
Gates Formula	EOD	0.96	0.41	139
Gates Formula	BOR	1.33	0.48	122

* From Rausche *et al.* (1996)
EOD = End of Driving, BOR = Beginning of Restrike

The data base compiled by Rausche *et al.* (1996) has been modified to include capacity predictions from the Engineering News and Gates dynamic formulas at both the end of driving and beginning of restrike. The data base for the dynamic formulas has also been expanded, and includes additional data sets. For evaluation of dynamic formula performance, the allowable load determined using the Engineering News formula was compared to one half of the ultimate capacity determined from the static load test. The ultimate capacity from the Gates formula was compared directly to the ultimate capacity determined from the static load test. The correlation results of the dynamic formulas are included in Table 16-1.

Based on the end of driving data, the Engineering News formula had a mean value of 1.22 and a coefficient of variation of 0.74, while the Gates had a mean value of 0.96 with a coefficient of variation of 0.41. The coefficient of variation is the standard deviation divided by the mean value. Hence, the greater a method's mean value is from 1.0 the lower the accuracy of the method, and the larger the coefficient of variation the less reliable the method. Table 16-1 clearly shows the Engineering News formula has a tendency to overpredict pile capacity. The higher coefficient of variation also suggests that the Engineering News formula is significantly less reliable than the Gates formula.

Table 16-1 illustrates that evaluation of pile capacity, by either Gates or Engineering News dynamic formula from restrike set and energy observations, has a significant tendency to overpredict pile capacity. The Engineering News formula capacity results, from restrike observations, had a mean value of 1.89 and a coefficient of variation of 0.46. The Gates formula capacity results, from restrike observations, had a mean value of 1.33 and a coefficient of variations of 0.48

If the static load test failure loads are divided by the Engineering News allowable design loads, the data base indicates an average factor of safety of 2.3 as compared to the factor of safety of 6.0 theoretically included in the formula. More important, the actual factor of safety from the Engineering News formula ranged from 0.6 to 13.1. This lack of reliability causes the Engineering News formula to be ineffective as a tool for estimating pile capacity. The fact that 12% of the data base has a factor of safety of 1.0 or less is also significant. However, complete failure of a bridge due to inadequate pile capacity determined by Engineering News formula is unusual. The problem usually is indicated by long term damaging settlements which occur after construction when the maximum load is intermittently applied.

16.2 PROBLEMS WITH DYNAMIC FORMULAS

Dynamic formulas are fundamentally incorrect. The problems associated with pile driving formulas can be traced to the modeling of each component within the pile driving process: the driving system, the soil, and the pile. Dynamic formulas offer a poor representation of the driving system and the energy losses of drive system components. Dynamic formulas also assume a rigid pile, thus neglecting pile axial stiffness effects on driveability, and further assume that the soil resistance is constant and instantaneous to the impact force. A more detailed discussion of these problems is presented below.

First, the derivation of most formulas is not based on a realistic treatment of the driving system. Most formulas only consider the kinetic energy of the driving system. The variability of equipment performance is typically not considered. Driving systems include many elements in addition to the ram, such as the anvil for a diesel hammer, the helmet, the hammer cushion, and for a concrete pile, the pile cushion. These components affect the distribution of the hammer energy with time, both at and after impact, which influences the magnitude and duration of peak force. The peak force and its duration determines the ability of the driving system to advance the pile into the soil.

Second, the soil resistance is very crudely treated by assuming that it is a constant force. This assumption neglects even the most obvious characteristics of real soil behavior. The dynamic soil resistance is the resistance of the soil to rapid pile penetration produced by a hammer blow. This resistance is by no means identical with the static soil resistance. However, most dynamic formulas consider the resistance during driving equal to the static resistance or pile capacity. The rapid penetration of the pile into the soil during driving is resisted not only by static friction and cohesion, but also by the soil viscosity, which is comparable to the viscous resistance of liquids against rapid displacement under an applied force. The net effect is that the driving process creates dynamic resistance forces along the pile shaft and at the pile toe, due to the high shear rate. The soil resistance during driving, from the combination of dynamic soil resistance and available static soil resistance, is generally not equal to the static soil resistance or pile capacity under static loads.

Third, the pile is assumed to be rigid and its length is not considered. This assumption completely neglects the pile's flexibility, which affects its ability to penetrate the soil. The energy delivered by the hammer sets up time-dependent stresses and displacements in the helmet, in the pile, and in the surrounding soil. In addition, the pile behaves, not as a concentrated mass, but as a long elastic rod in which stresses travel longitudinally as waves. Compressive waves which travel to the pile toe are responsible for advancing the pile into the ground.

16.3 DYNAMIC FORMULAS

As noted in Section 16.1, the Engineering News formula is generally recognized to be one of the least accurate and least consistent of dynamic formulas. Due to the overall poor correlations documented between pile capacities determined from this method and static load test results, the use of the Engineering News formula is not recommended.

For small projects where a dynamic formula is used, statistics indicate that the Gates formula is preferable, since it correlates better with static load test results. The Gates formula presented below has been revised to reflect the ultimate pile capacity in kilonewtons and includes the 80 percent efficiency factor on the rated energy, E_r, recommended by Gates.

$$R_u = [7 \sqrt{E_r} \log(10N_b)] - 550$$

Where: R_u = The ultimate pile capacity (kN).

E_r = The manufacturer's rated hammer energy (Joules) at the field observed ram stroke.

$\log(10N_b)$ = Logarithm to the base 10 of the quantity 10 multiplied by N_b, the number of hammer blows per 25 mm at final penetration.

It is sometimes desirable to calculate the number of hammer blows per 0.25 meter (250 mm) of pile penetration, N_{qm}, required to obtain the ultimate pile capacity. For this need, the Gates formula can be written in the following form:

$$N_{qm} = 10(10^x)$$

Where: $x = [(R_u + 550)/(7\sqrt{E_r})] - 1$

Most dynamic formulas are in terms of ultimate pile capacity, rather than allowable or design load. For ultimate pile capacity formulas, the design load should be multiplied by a factor of safety to obtain the ultimate pile capacity that is input into the formula to determine the "set", or amount of pile penetration per blow required. A factor of safety of 3.5 is recommended when using the Gates formula. For example, if a design load of 700 kN is required in the bearing layer, then an ultimate pile capacity of 2450 kN should be used in the Gates formula to determine the necessary driving resistance.

Highway agencies should establish long term correlations between pile capacity prediction from dynamic formulas and static load test results to failure. The Federal Highway Administration has created a national data base of pile load test results that can be accessed by Highway agencies to supplement local test information.

16.4 ALTERNATIVES TO USE OF DYNAMIC FORMULAS

Most shortcomings of dynamic formulas can be overcome by a more realistic analysis of the pile driving process. The one-dimensional wave equation analysis discussed in Chapter 17 is a more realistic method. However as little as ten years ago, wave equation analyses were primarily performed on main frame computers. Therefore, wave equation analysis was often viewed as a tool for special projects and not routine use. With the widespread use of fast personal computers in every day practice, wave equation analysis can now be easily performed in a relatively short amount of time.

As indicated in Table 16-1, ultimate pile capacity estimates from standard wave equation analysis using restrike driving resistance observations had a mean value of 1.22 and a coefficient of variation of 0.35. The performance of the wave equation capacity predictions improved when adjusted for measured drive system performance from dynamic measurements.

Dynamic testing and analysis is another tool which is superior to use of dynamic formulas. This topic will be discussed in greater detail in Chapter 18. Table 16-1 illustrates that dynamic measurements with CAPWAP analysis performed better than either the Engineering News or Gates dynamic formulas. Ultimate pile capacity estimates from restrike dynamic measurements with CAPWAP analysis had a mean value of 0.92 and a coefficient of variation of 0.22.

Modern dynamic methods of wave equation analysis, as well as dynamic testing and analysis, are superior to traditional dynamic formulas. Modern methods should be used in conjunction with static pile load tests whenever possible, and the use of dynamic formulas should be discontinued.

16.5 DYNAMIC FORMULA CASE HISTORIES

To illustrate the variable performance of dynamic formulas compared to modern dynamic methods, three case histories will be briefly discussed. The case histories were selected to include a range of pile types and sizes, hammer types, and soil conditions.

16.5.1 Case History 1

Case History 1 involves a 610 mm square prestressed concrete pile with a 305 mm diameter circular void at the pile center. The concrete pile was driven through loose to medium dense clayey sands to a dense clayey sand layer. A Vulcan 020 single acting air hammer operated at a reduced stroke of 0.9 meters and corresponding rated energy of 81 kJ was used to drive the pile. The pile was driven to a final penetration resistance of 34 blows per 0.25 meter. When restruck 13 days after initial driving, the pile had a penetration resistance of 118 blows per 0.25 meter. This pile was then statically load tested.

Using end of driving set observations, the Engineering News formula predicted an allowable design load of 1360 kN and the Gates formula predicted an ultimate pile capacity of 2476 kN. Modern dynamic methods of the wave equation and dynamic testing with CAPWAP analysis gave restrike ultimate pile capacities of 4561 and 4111 kN, respectively. The static load test pile had a Davisson failure load of 4223 kN. Hence, the Engineering News and Gates dynamic formulas significantly underpredicted the allowable and ultimate pile capacity, respectively. Dynamic test data indicated the restrike capacity was 2.5 times the capacity at the end of initial driving. This high setup condition most likely caused the underpredictions by the dynamic formulas.

16.5.2 Case History 2

Case History 2 involves a 356 mm O.D. closed end pipe pile driven into a dense to very dense sand and gravel. The pile had a design load of 620 kN and a required ultimate capacity of 1550 kN, which included an anticipated capacity loss due to scour. An IHC S-70 hydraulic hammer with a maximum rated energy of 69 kJ was used to install the pile. The IHC hydraulic hammers can be operated over a wide energy range and include a readout panel that indicates for each blow the hammer kinetic energy prior to impact. The static load test pile was driven to a final penetration resistance of 26 blows per 0.25 meter at a readout panel energy of 28 kJ. Restrike tests at the site indicated minimal changes in pile capacity with time.

Based on end of driving set observations, the Engineering News formula predicted an allowable design load of 387 kN and the Gates formula predicted an ultimate pile capacity of 1142 kN. The preconstruction wave equation analysis predicted an ultimate pile capacity of 1333 kN. Restrike dynamic testing with CAPWAP analysis predicted an ultimate pile capacity of 1605 kN. The static load test pile had a Davisson failure load of 1627 kN. Hence, both the Engineering News and Gates dynamic formulas significantly underpredicted the allowable and ultimate pile capacity, respectively. In this particular case, the poor performance of the dynamic formulas is most likely attributed to the high energy transfer efficiency of the IHC type hydraulic hammer relative to its kinetic energy rating based on the readout panel.

16.5.3 Case History 3

In Case History 3, a 356 mm O.D. closed end pipe pile was driven through loose to medium dense sands to toe bearing in a very dense sand. The pipe pile had a design load of 980 kN and a required ultimate pile capacity of 1960 kN. An ICE 42-S single acting diesel hammer with a rated energy of 57 kJ was used to drive the load test pile to a final driving resistance of 148 blows per 0.25 meter at a hammer stroke of 3 meters.

Using the end of driving set observations, the Engineering News formula predicted an allowable design load of 2180 kN and the Gates formula predicted an ultimate pile capacity of 2988 kN. Dynamic testing with CAPWAP analysis indicated an ultimate pile capacity of 2037 kN at the end of initial driving, that decreased to an ultimate capacity of 1824 kN during restrike. The static load test pile had a Davisson failure load of 1868 kN. Assuming a safety factor of 2, the allowable pile capacity would be 934 kN. Hence, the Engineering News formula overpredicted the allowable design load by more than 230% and the Gates formula overpredicted the ultimate pile capacity by 60%.

The magnitude of the overprediction by the dynamic formulas is at least partially attributed to the soil relaxation (capacity at end of driving higher than some time later) that occurred at the site. Pile capacities determined from dynamic formulas are routinely calculated from initial driving observations. Therefore, the time dependent decrease in pile capacity would not likely have been detected if only dynamic formulas had been used for pile driving control on this project.

The case histories above illustrate that different methods often result in a range of predicted capacities at a given site. The magnitude of pile capacity changes with time. Both hammer performance characteristics and soil behavior can be different from those than typically assumed. The three case histories presented illustrate that pile capacity evaluations with modern dynamic methods handle these variations better than traditional dynamic formulas.

REFERENCES

American Association of State Highway and Transportation Officials [AASHTO], (1994). Standard Specifications for Highway Bridges. Division 2, AASHTO Highway Subcommittee on Bridges and Structures, Washington, D.C.

Chellis R.D. (1961). Pile Foundations. Second Edition, McGraw-Hill Book Company, New York, 21-23.

Fragasny, R.J., Higgins, J.D. and Argo, D.E. (1988). Comparison of Methods for Estimating Pile Capacity. Report No. WA-RD 163.1, Washington State Department of Transportation, 62.

Peck, R.B. (1942). Discussion: Pile Driving Formulas. Proceedings of the American Society of Civil Engineers, Vol. 68, No. 2, 905-909.

Rausche, F., Thendean, G., Abou-matar, H., Likins. G.E. and Goble, G.G. (1996). Determination of Pile Driveability and Capacity from Penetration Tests. Final Report, U.S. Department of Transportation, Federal Highway Administration Research Contract DTFH61-91-C-00047.

Sowers, G.F. (1979). Introductory Soil Mechanics and Foundations. Fourth Edition, Macmillan Publishing Co., Inc., New York, 531-533.

STUDENT EXERCISE #9 - GATES FORMULA ULTIMATE CAPACITY

Use the Gates formula described in Section 16.3 to calculate the ultimate pile capacity of a 356 mm O.D. pipe pile driven with an ICE 42-S single acting diesel hammer to the driving resistances given in the table below. The field observed hammer strokes and corresponding manufacturer's rated energy are also included in the table. The Gates formula is presented below:

$$R_u = [\ 7\ \sqrt{E_r}\ \log\ (10\ N_b)]\ -\ 550$$

Where: R_u = ultimate pile capacity (kN).
E_r = manufacturer's rated energy at field stroke (joules).
N_b = number of hammer blows for 25 mm penetration.

Group Number	Pile Driving Resistance (blows / 250 mm)	Field Observed Stroke (m)	Manufacturer's Rated Energy (joules)	Gates Ultimate Pile Capacity (kN)
1	3	1.67	30,377	
2	7	2.43	44,202	
3	18	2.88	52,387	
4	37	3.10	56,389	
5	53	3.13	56,935	
6	72	3.02	54,934	
7	87	3.04	55,298	
8	107	3.04	55,298	
9	133	3.05	55,480	
10	168	3.05	55,480	

STUDENT EXERCISE #10 - GATES FORMULA DRIVING CRITERION

The Gates formula is to be used for construction control on a new bridge project. The piles have a design load of 620 kN and are to be driven through 5 meters of scourable soils that were calculated to provide 90 kN of resistance at the time of driving. A Kobe K 25 single acting diesel hammer will be used to drive the piles. First determine the required ultimate pile capacity. Then use the Gates formula provided below and described in Section 16.3 to calculate the required driving resistance for the ultimate pile capacity at the hammer strokes shown in the table below.

$$N_{qm} = 10(10^{x})$$

$$x = [(R_u + 550)/(7\sqrt{E_r})] - 1$$

Where: N_{qm} = number of hammer blows for 250 mm penetration.
R_u = ultimate pile capacity (kN).
E_r = manufacturer's rated energy at field stroke (joules).

Group Number	Field Observed Stroke (m)	Manufacturer's Rated Energy (joules)	Exponent (x)	Required Driving Resistance (blows / 250 mm)
1	1.50	36,870		
2	1.65	40,458		
3	1.80	44,136		
4	1.95	47,814		
5	2.10	51,492		
6	2.25	55,170		
7	2.40	58,848		
8	2.55	62,526		
9	2.70	66,204		
10	2.85	69,882		

17. DYNAMIC ANALYSIS BY WAVE EQUATION

17.1 INTRODUCTION

As discussed in previous chapters, dynamic formulas, together with observed driving resistances, do not yield acceptably accurate predictions of actual pile capacities. Moreover, they do not provide information on stresses in the piles during driving. The so-called wave equation analysis of pile driving has eliminated many shortcomings associated with dynamic formulas by realistically simulating the hammer impacts and pile penetration process. For most engineers, the term wave equation refers to a partial differential equation. However, for the foundation specialist, it means a complete approach to the mathematical representation of a system consisting of hammer, cushions, helmet, pile and soil, and an associated computer program for the convenient calculation of the motions and forces in this system after ram impact.

The approach was developed by E.A.L. Smith (1960), and after the rationality of the approach had been recognized, several researchers developed a number of computer programs. For example, the Texas Department of Highways supported research at the Texas Transportation Institute (TTI) in an attempt to reduce concrete pile damage using a realistic analysis method. FHWA sponsored the development of both the TTI program (Hirsch *et al.* 1976) and the WEAP program (Goble and Rausche, 1976). FHWA supported the WEAP development to obtain analysis results backed by measurements taken on construction piles during installation for a variety of hammer models. The WEAP program was updated several times under FHWA sponsorship, the last time (Goble and Rausche, 1986) when the WEAP86 program was released. Later, additional options, improved data files, refined mathematical representations and modernized user conveniences were added to this program on a proprietary basis, and the program is now known as GRLWEAP (Goble Rausche Likins and Associates, Inc., 1996). GRLWEAP has been accepted for use on public projects by a variety of agencies (*e.g.* AASHTO, 1992, US Army Corps of Engineers, 1993), State Departments of Transportation, and the FHWA for routine analyses. However, this should not be construed as a promotion or endorsement.

The wave equation approach has been subjected to a great number of checks and correlation studies. Studies on the performance of WEAP, the most widely accepted program, produced publications demonstrating the program's performance and utility (*e.g.* Blendy 1979, Soares *et al.* 1984).

This chapter will explain what a wave equation analysis is, how it works, and what problems it can solve. Example problems, highlighting program applications, will be demonstrated. Also, basic program usage and application of program results will be presented.

17.2 WAVE PROPAGATION

Input preparation for wave equation analyses is often very simple, requiring only very basic driving system and pile parameters in addition to a few standard soil properties. Thus, a wave equation program can be run without much specialized knowledge. However, interpretation of calculated results is facilitated, and errors in result application may be avoided, by a knowledge of the mechanics of stress wave propagation.

In the first moment, when a pile is struck by a hammer, it is only compressed at the ram-pile interface. This compressed zone, or force pulse, as shown in Figure 17.1, expands into the pile toward the pile toe at a constant wave speed, C, which depends on the pile's elastic modulus and mass density (or specific weight). When the force pulse reaches the embedded portion of the pile, its amplitude is reduced by the action of static and dynamic soil resistance forces. Depending on the magnitude of the soil resistances along the pile shaft and at the pile toe, the force pulse will generate either a tensile or a compressive force pulse which travels back to the pile head. Both incident and reflected force pulses will cause a pile toe motion and produce a permanent pile set if their combined energy and force are sufficient to overcome the static and dynamic resistance effects of the soil.

17.3 WAVE EQUATION METHODOLOGY

In a wave equation analysis, the hammer, helmet, and pile are modeled by a series of segments each consisting of a concentrated mass and a weightless spring. The hammer and pile segments are approximately one meter in length. Shorter segments

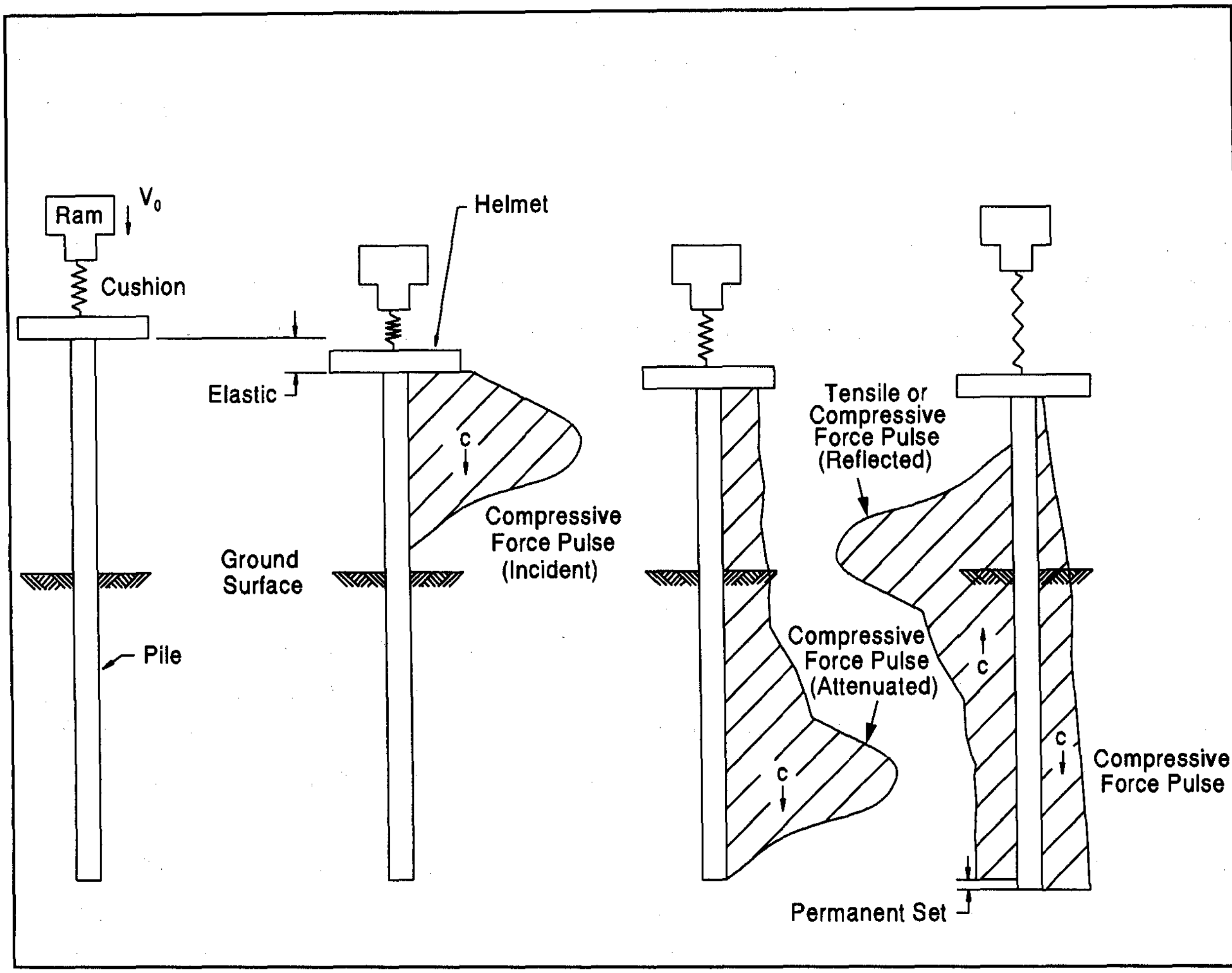


Figure 17.1 Wave Propagation in a Pile (adapted from Cheney and Chassie, 1993)

often improve the accuracy of the numerical solution at the expense of longer computer times. Spring stiffnesses are calculated from the cross sectional area and modulus of elasticity of the corresponding pile section. Hammer and pile cushions are represented by additional springs whose stiffnesses are calculated from area, modulus of elasticity, and thickness of the cushion materials. In addition, coefficients of restitution (COR) are usually specified to model energy losses in cushion materials, and in all segments which can separate from their neighboring segments by a certain slack distance. The COR is equal to one for a perfectly elastic collision which preserves all energy and is equal to zero for a perfectly plastic condition which loses all deformation energy. Partially elastic collisions are modeled with an intermediate COR value.

The soil resistance along the embedded portion of the pile and at the pile toe is represented by both static and dynamic components. Therefore, both a static and a dynamic soil resistance force acts on every embedded pile segment. The static soil resistance forces are modeled by elasto-plastic springs and the dynamic soil resistance by linear viscous dashpots. The displacement at which the soil changes from elastic to plastic behavior is referred to as the soil "quake". In the Smith damping model, the dynamic soil resistance is proportional to a damping factor times the pile velocity times the assigned static soil resistance. A schematic of the wave equation hammer-pile-soil model is presented in Figure 17.2.

As the analysis commences, a calculated or assumed ultimate capacity, R_{ut}, from user specified values is distributed according to user input among the elasto-plastic springs along the shaft and toe. Similarly, user specified damping factors are assigned to shaft and toe to represent the dynamic soil resistance. The analysis then proceeds by calculating a ram velocity using the input hammer efficiency and stroke. The ram movement causes displacements of helmet and pile head springs, and therefore compressions (or extensions) and related forces acting at the top and bottom of the segments. Furthermore, the movement of a pile segment causes soil resistance forces. A summation of all forces acting on a segment, divided by its mass, yields the acceleration of the segment. The product of acceleration and time step summed over time is the segment velocity. The velocity multiplied by the time step yields a change of segment displacement which then results in new spring forces. These forces divided by the pile cross sectional area at the corresponding section equal the stress at that point.

Similar calculations are made for each segment until the accelerations, velocities and displacements of all segments have been calculated during the time step. The analysis then repeats for the next time step using the updated motion of the segments from the previous time step. From this process, the accelerations, velocities, displacements, forces, and stresses of each segment are computed over time. Additional time steps are analyzed until the pile toe begins to rebound.

The permanent set (mm) of the pile toe is calculated by subtracting a weighted average of the shaft and toe quakes from the maximum pile toe displacement. The inverse of the permanent set is the driving resistance (blow count) in blows per meter that corresponds to the input ultimate capacity. By performing wave equation analyses over a wide range of ultimate capacities, a curve or "bearing graph" can be plotted which relates ultimate capacity to driving resistance.

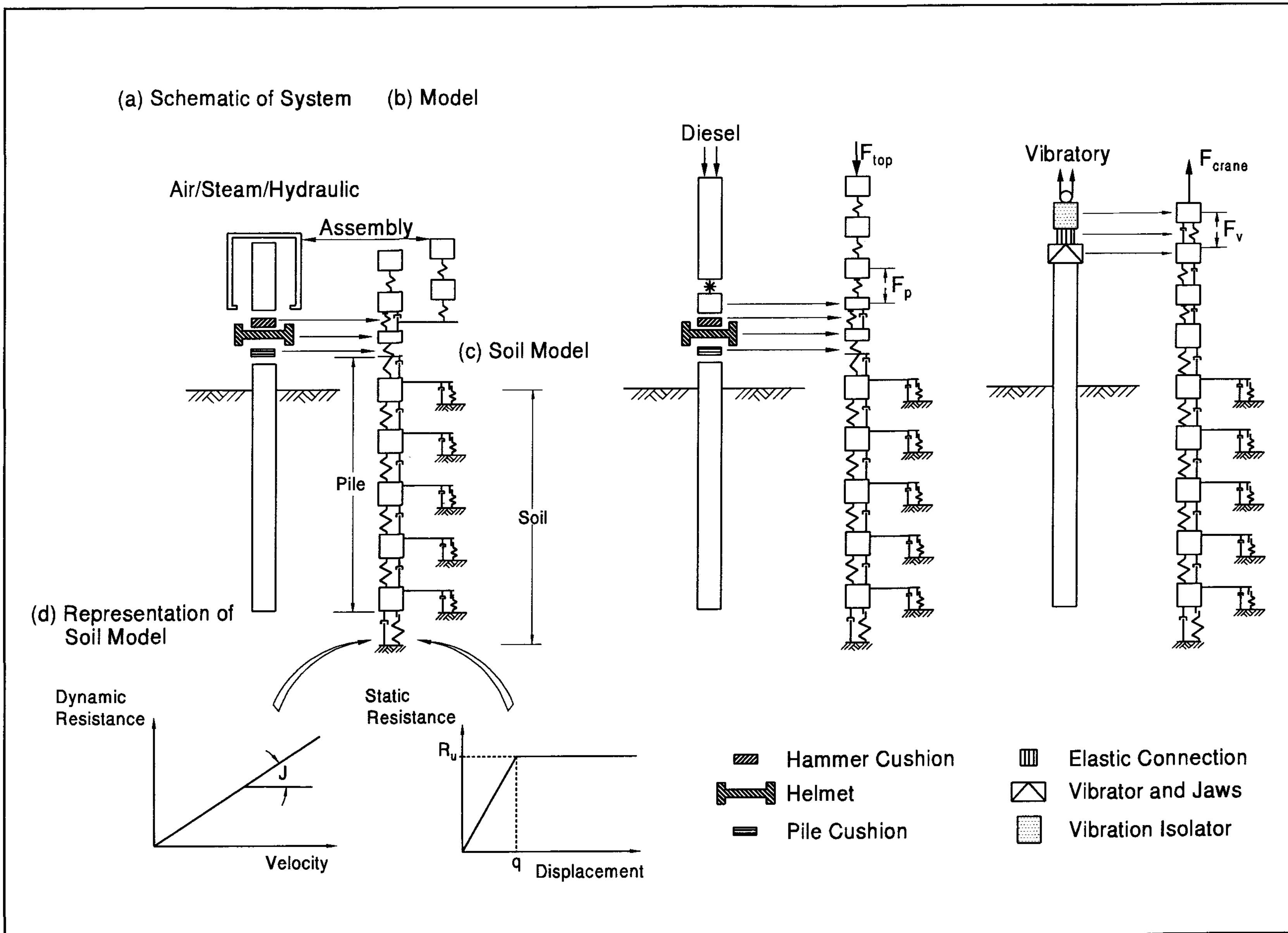


Figure 17.2 Typical Wave Equation Model

A wave equation bearing graph is substantially different from a similar graph generated from a dynamic formula. The wave equation bearing graph is associated with a single driving system, hammer stroke, pile type, soil profile, and a particular pile length. If any one of the above items is changed, the bearing graph will also change. Furthermore, wave equation bearing graphs also include the maxima of calculated compression and tension stresses.

In addition to the bearing graph, GRLWEAP provides options for two alternative results, the constant capacity analysis, or "inspector's chart", and the driveability analysis. The inspector's chart establishes a relationship between hammer energy or stroke and driving resistance for one particular, user specified, ultimate capacity value. Associated stress maxima are also included in the chart, enabling the user to select a practical hammer energy or stroke range both for reasonable driving resistances and driving stress control. This analysis option is described in greater detail in Section 17.5.2.

The driveability analysis calculates driving resistances and stresses from user input shaft and toe resistance values at up to 100 user selected pile penetrations. The calculated results can then be plotted together with the capacity values versus pile penetration. The resulting plot would depict those pile penetrations where refusal might be expected or where dangerously high driving stress levels could develop. In addition, a crude estimate of pure driving time (not counting interruptions) is provided by this analysis option. The driveability option is described in greater detail in Section 17.5.5.

17.4 WAVE EQUATION APPLICATIONS

A bearing graph provides the wave equation analyst with two types of information:

1. It establishes a relationship between ultimate capacity and driving resistance. From the user's input data on the shaft and toe bearing resistances, the analysis estimates the permanent set (mm/blow) under one hammer blow. Specifying up to ten ultimate capacity values yields a relationship between ultimate capacity and driving resistance (or blow count) in blows per meter.

2. The analysis also relates driving stresses in the pile to pile driving resistance.

The user usually develops a bearing graph or an inspector's chart for different pile lengths and uses these graphs in the field, with the observed driving resistance, to determine when the pile has been driven sufficiently for the required bearing capacity.

In the design stage, the foundation engineer should select typical pile types and driving equipment known to be locally available. Then by performing the wave equation analysis with various equipment and pile size combinations, it becomes possible to rationally:

1. Design the pile section for driveability to the required depth and/or capacity.

   For example, scour considerations or consolidation of lower soft layers may make it necessary to drive a pile through hard layers whose driving resistance exceeds the resistance expected at final penetration. A thin walled pipe pile may have been initially chosen during design. However when this section is checked for driveability, the wave equation analysis may indicate that even the largest hammers will not be able to drive the pipe pile to the required depth because it is too flexible (its impedance is too low). Therefore, a wall thickness greater than necessary to carry the design load, has to be chosen for driveability considerations. (Switching to an H-pile or predrilling may be other alternatives).

2. Aid in the selection of pile material properties to be specified based on probable driving stresses in reaching penetration and/or capacity requirements.

   Suppose that, in the above example, it would be possible to drive the thinner walled pile to the desired depth, but with excessive driving stresses. More cushioning or a reduced hammer energy would lower the stresses but would result in a refusal driving resistance. Choosing a high strength steel grade could solve this problem. For concrete piles, higher concrete strength and/or higher prestress levels may provide acceptable solutions.

3. Support the decision for a new penetration depth, design load, and/or different number of piles.

   In the above example, after it has been determined that the pile section or its material strength had to be increased to satisfy pile penetration requirements, it may have become feasible to increase the design load of each pile and to reduce the total number of piles. Obviously, these considerations would require revisiting geotechnical and/or structural considerations.

Once the project has reached the construction stage, additional wave equation analyses should be performed on the actual driving equipment by:

1. Construction engineers - for hammer approval and cushion design.

   Once the pile type, material, and pile penetration requirements have been selected by the foundation designer, the hammer size and hammer type may have a decisive influence on driving stresses. For example, a hammer with adjustable stroke or fuel pump setting may have the ability to drive a concrete pile through a hard layer while allowing for reduced stroke heights and tension stress control when penetrating soft soil layers.

   Cushions are often chosen to reduce driving stresses. However, softer cushions absorb and dissipate greater amounts of energy thereby increasing the driving resistance. Since it is both safer (reducing fatigue effects) and more economical to limit the number of blows applied to a pile, softer cushions cannot always be chosen to maintain acceptable driving stresses. Also, experience has shown that changes of hammer cushion material are relatively ineffective for limiting driving stresses.

   Hammer size, energy setting, and cushion materials should always be chosen such that the maximum expected driving resistance is less than 120 blows per 0.25 meter or 480 blows/m (exceptions are end-of-driving blow counts of pure toe bearing piles). The final driving resistance should also be greater than 30 blows per 0.25 meter (120 blows/m) for a reasonably accurate driving criterion (the lower the blow count the greater the possibility of inaccurate blow count measurements).

2. Contractors - to select an economical combination of driving equipment to minimize installation cost.

   While the construction engineer is interested in the safest installation method, contractors would like to optimize driving time for cost considerations. Fast hitting, light weight, simple and rugged hammers which have a high blow rate are obviously preferred. The wave equation analysis can be used to roughly estimate the anticipated number of hammer blows and the time of driving. This information is particularly useful for a relative evaluation of the economy of driving systems.

Additional considerations might include the cost of pile cushions which are usually discarded after a pile has been installed. Thus, thick plywood pile cushions may be expensive.

Near refusal driving resistances are particularly time-consuming and since it is known that stiffer piles drive faster with lower risk of damage, the contractor may even choose to upgrade the wall thickness of a pipe pile or the section of an H-pile for improved overall economy.

17.5 WAVE EQUATION EXAMPLES

This section presents several examples that illustrate the application of the wave equation analysis for the solution of design and construction problems. The factor of safety applied to the design load in the following examples is 2. This assumes that a static pile load test was performed on each project. As noted in Chapter 15, a factor of safety of 2.5 to 2.75 should be applied to the design load in wave equation analyses if static load testing or dynamic testing is not included in the project. The ultimate capacity in a wave equation analysis should consist of the factor of safety, times the design load, plus the sum of the ultimate resistances from any overlying layers unsuitable for long term support.

Note: The figures illustrating the following examples were generated from the proprietary program GRLWEAP. These figures are not intended as endorsements of Goble Rausche Likins and Associates, Inc. (GRL), its products or services.

17.5.1 Example 1 - General Bearing Graph

A primary application of a wave equation analysis is to develop a bearing graph relating the ultimate pile capacity to the pile driving resistance. For a desired ultimate pile capacity, the required driving resistance can be easily found from this graph. Consider the soil profile in Figure 17.3. In this example, a 356 mm by 8 mm wall, closed end pipe pile (yield strength 241 MPa) is to be driven into a deep deposit of medium sands. A static analysis performed using the SPILE program indicates that an ultimate pile capacity of 1480 kN can be obtained for the proposed pile type at a penetration depth

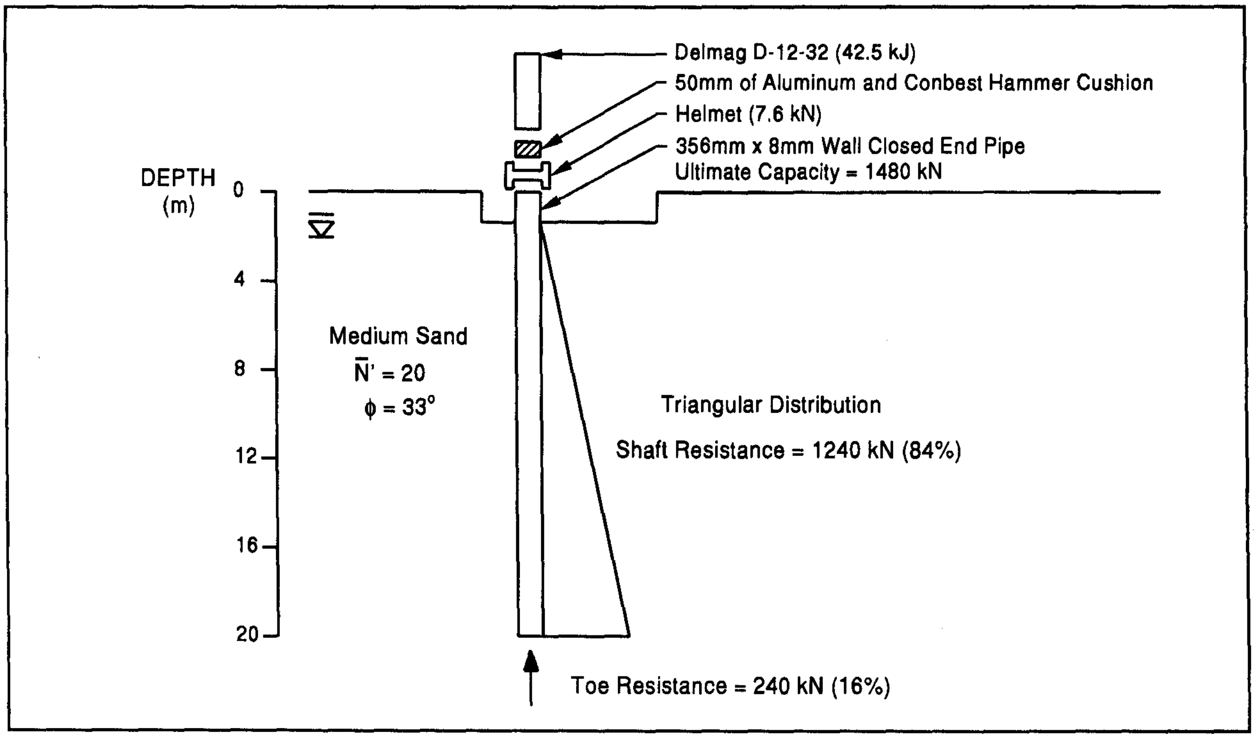


Figure 17.3 Example 1 Problem Profile

of 19 meters. The static analysis also indicates that the ultimate capacity is distributed as 84% shaft resistance and 16% toe bearing resistance with the shaft resistance being distributed triangularly along the pile shaft.

The contractor selected a Delmag D-12-32 single acting diesel hammer for driving the pipe piles. The contractor's hammer submittal indicates that the hammer cushion will consist of 25 mm of aluminum and 25 mm of Conbest with a cross sectional area of 1464 cm^2. A helmet weight of 7.6 kN is reported.

Based on this information, a wave equation analysis can be performed. The ultimate pile capacity of 1480 kN is input along with selected additional ultimate capacities. The wave equation analysis calculates the net set of the pile toe for each input ultimate capacity. The inverse of the set is the driving resistance for that ultimate capacity expressed in blows per meter (blows/m). By plotting the calculated driving resistances versus the corresponding ultimate capacities, a bearing graph is developed. The results of such a calculation are shown in Figure 17.4 for the 356 mm closed end pipe pile.

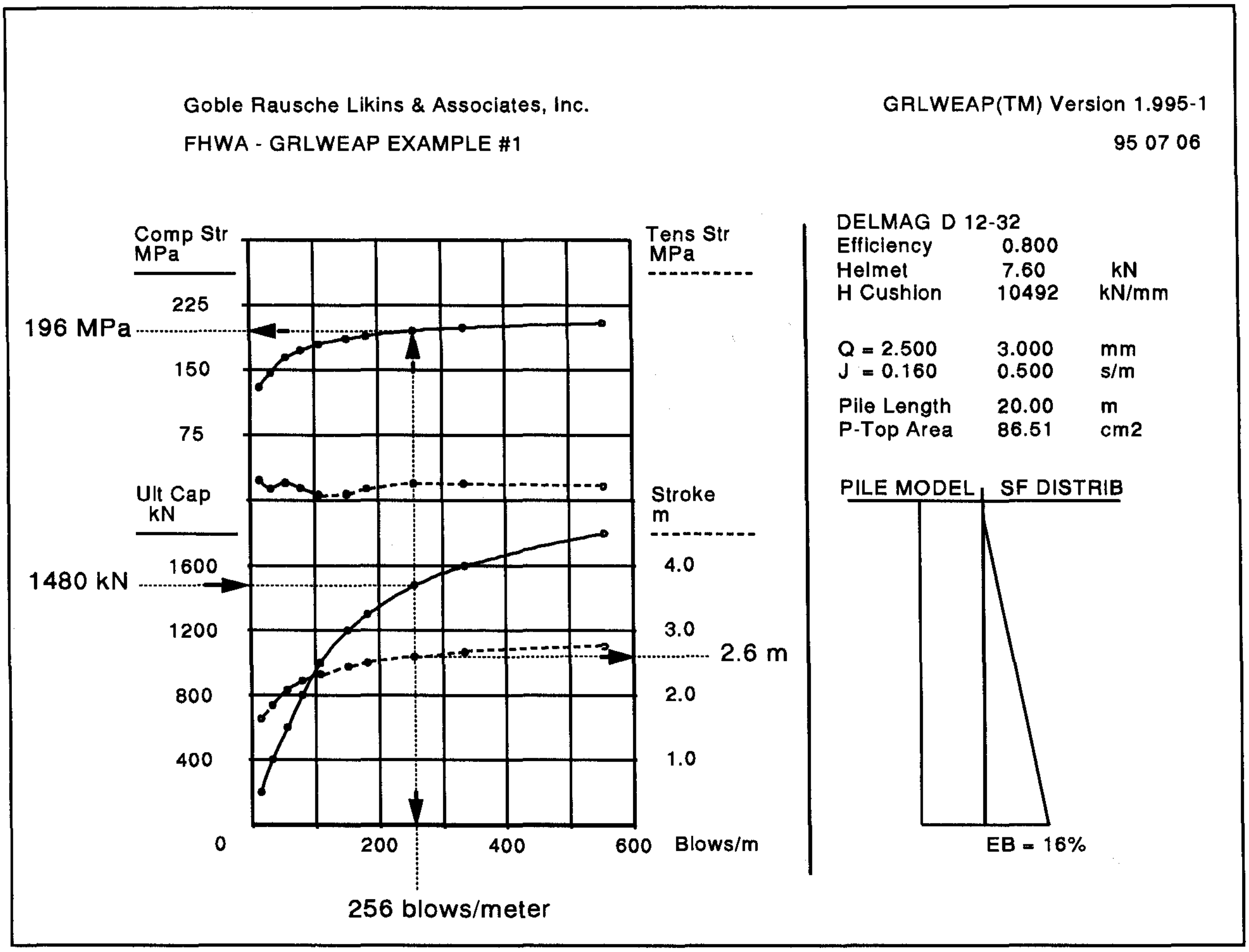


Figure 17.4 Example 1 Typical Bearing Graph

In the bottom half of Figure 17.4, the ultimate pile capacity versus driving resistance in blows/m is represented by the solid line. This graph shows that for an ultimate pile capacity of 1480 kN a blow count of 256 blows/m is required. (This requirement is really equivalent to a criterion of 64 blows per 0.25 meter penetration, or approximately 40 mm for 10 blows). As a driving criterion, this is a reasonable blow count requirement being neither excessively high which would demand extreme driving efforts nor very low and therefore inaccurate. Also in the bottom half of the graph, the corresponding hammer stroke versus driving resistance is represented by the dashed line. This curve is important for a check on hammer performance when the driving criterion is applied. In this case, the driving resistance of 256 blows/m for the 1480 kN capacity is based upon a hammer stroke of 2.6 meters. Should field observations indicate significantly (say more than 10% difference) higher or lower strokes, then a lower or higher blow count would be satisfactory (needed) for the same capacity. Hammer stroke information is therefore essential for field evaluation and control of the pile installation process. For significantly differing hammer field performance, new wave equation analyses would be necessary with a modified maximum combustion pressure or a fixed input stroke.

In the upper half of Figure 17.4, maximum compression and tension driving stresses are also plotted as a function of driving resistance. Of primary interest for a steel pile is the compression driving stress which is represented by the solid line. This curve shows that at the driving resistance of 256 blows/m associated with the required ultimate pile capacity, the maximum compression stress calculated in the pile is 196 MPa which is less than 90% of yield strength of steel (0.9 (241) = 217 MPa), and therefore acceptable. Note, however, that any non-axial stress components (such as from bending) are not included in the wave equation results and would be additional.

Though the analysis was conducted for an estimated penetration of 19 m, the required driving resistance may be reached at a shallower depth, or additional penetration may be required. The static analysis only serves as an initial estimate of the required penetration depth. The actual driving behavior and construction monitoring will confirm whether or not the static calculation was adequate. If the actual driving behavior is significantly different from the analyzed situation (say the driving resistance is already reached at 15 m penetration) then an additional analysis should be performed to better match the field observations. However, in general, the bearing graph is relatively insensitive to changes in penetration unless there is a significant change in the soil profile. Higher driving resistances from penetrating embankment fills or scour susceptible material, *etc.*, would also need to be considered in this assessment.

17.5.2 Example 2 - Constant Capacity / Variable Stroke Option

The hammer-pile-soil information used in Example 1 will be reused for a constant capacity (or Inspector's chart) analysis in Example 2. In this example, the driving resistance required for the 1480 kN ultimate capacity is evaluated at hammer strokes other than the predicted 2.6 meter stroke. This information will be helpful for field personnel in determining when pile driving can be terminated if the field observed hammer stroke varies from the hammer stroke predicted by the wave equation.

In the constant capacity/variable stroke option, a single ultimate capacity (usually the required ultimate capacity) is chosen and the hammer stroke is varied from minimum to maximum values. The necessary driving resistance at each hammer stroke is then calculated for the input ultimate capacity. The lower half of Figure 17.5 presents these results for an ultimate pile capacity of 1480 kN. When the point of intersection of the observed stroke and driving resistance plots below the curve, the ultimate pile capacity has not been obtained. Any combination of stroke and driving resistance plotting above the curve indicates that the required ultimate pile capacity has been reached. Hence this analysis option is useful for field control by inspection personnel.

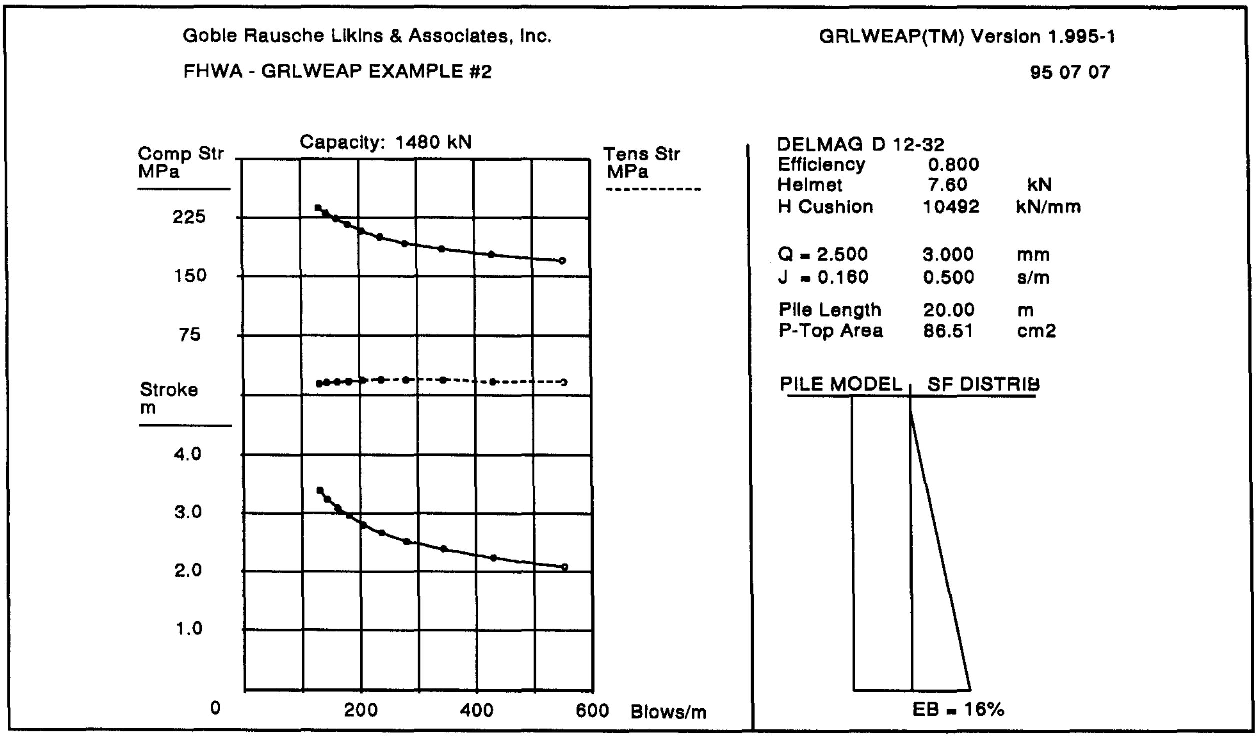


Figure 17.5 Example 2 Constant Capacity Analysis

17.5.3 Example 3 - Tension and Compression Stress Control

Example 3 illustrates the use of the wave equation for the control of tension stresses in concrete piles. The North Abutment of the Peach Freeway design problem presented in Chapter 13 will be used for this example problem. For the North Abutment, static calculations performed using the Nordlund method indicated that a 356 mm square prestressed concrete pile driven through 4 m of loose silty fine sand, 7 m of medium dense silty fine sand, and 0.5 m into a dense sand and gravel deposit could develop an ultimate pile capacity of 1807 kN (which is slightly more than the 1780 kN required). The static analysis also indicates that the ultimate capacity is distributed as 48% shaft resistance and 52% toe resistance, with the shaft resistance being distributed triangularly along the pile shaft. The soil profile for this problem is presented in Figure 17.6.

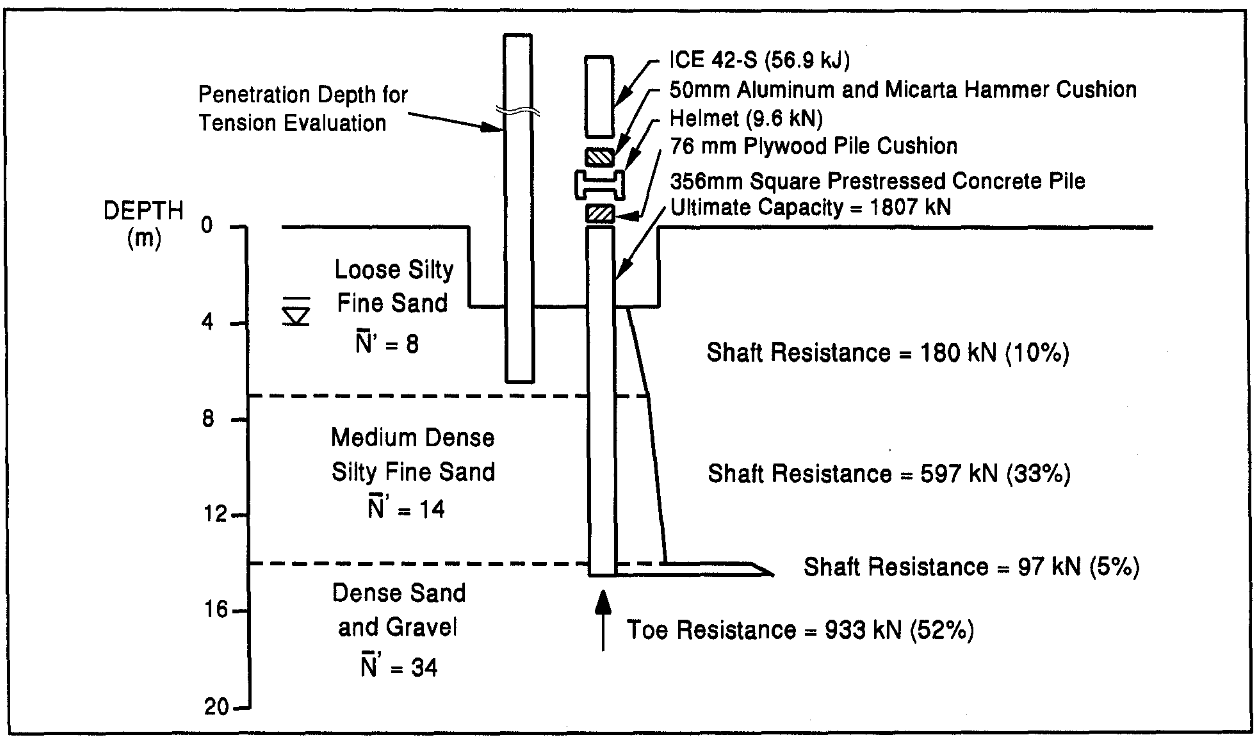


Figure 17.6 Example 3 Problem Profile

The contractor selected an ICE 42-S single acting diesel hammer for driving the prestressed concrete piles. The contractor's hammer submittal indicates that the hammer cushion will consist of 25 mm of aluminum and 25 mm of Micarta with a cross sectional area of 2568 cm^2. A helmet weight of 9.6 kN is reported. The proposed pile cushion is 76 mm of plywood. The pile will have a concrete compression strength of 37.9 MPa and an effective prestress after losses of 5.2 MPa. Using the AASHTO driving stress recommendations from Chapter 11, this results in a maximum recommended compression stress of 27.0 MPa and a maximum tension driving stress of 6.7 MPa.

One of the main concerns with concrete piles is the possibility of developing high tension stresses during easy driving conditions when the soil provides little or no toe resistance. Therefore, the wave equation should be used to evaluate the contractor's proposed driving system during both low and high resistance conditions.

First, it is necessary to evaluate tension stresses during easy driving. The weight of the pile and driving system is anticipated to be on the order of 100 kN. Hence, the pile penetration depth for the wave equation analysis should be selected below the depth to which the pile will likely penetrate or "run" under the weight of the pile and driving system. At a depth of 3.5 m, the pile is still within the loose silty fine sand stratum and tension driving stresses are anticipated to be near their peak. At this depth, a pile capacity of 357 kN was calculated from a static analysis.

The contractor has submitted a plywood pile cushion design comprised of four 19 mm sheets with a total thickness of 76 mm. The pile cushion stiffness will significantly affect the tension driving stresses. Therefore it is necessary to check whether or not the contractor's proposed pile cushion thickness is sufficient to maintain tension stress levels within specified limits. In the first analysis, the 76 mm pile cushion is anticipated to compress to about 80% of its initial thickness early in the driving process. Therefore, a pile cushion thickness of 61 mm will be used in the analysis instead of 76 mm to evaluate tension stresses during easy driving. The elastic modulus of the plywood at the start of driving is estimated to be 207 MPa.

Based on this information, the wave equation analysis indicates a maximum tension stress of 6.9 MPa at 6 m below the pile head. The magnitude of the tension stress exceeds the allowable driving stress limitation of 6.7 MPa. Thus, the pile cushion thickness should be increased, and another wave equation analysis should be performed. In the second analysis, eight sheets of 19 mm thick plywood with a total thickness of 152 mm are used for the pile cushion material. It is also assumed that this cushion thickness will compress to about 80% of its initial thickness or to 122 mm early in the driving process. The result of the second wave equation analysis indicates that the maximum tension stress reduces to 2.6 MPa for the thicker pile cushion, which is within specification limits. A comparison of the driving stresses for these two wave equation analyses along with standard bearing graphs is presented in Figure 17.7.

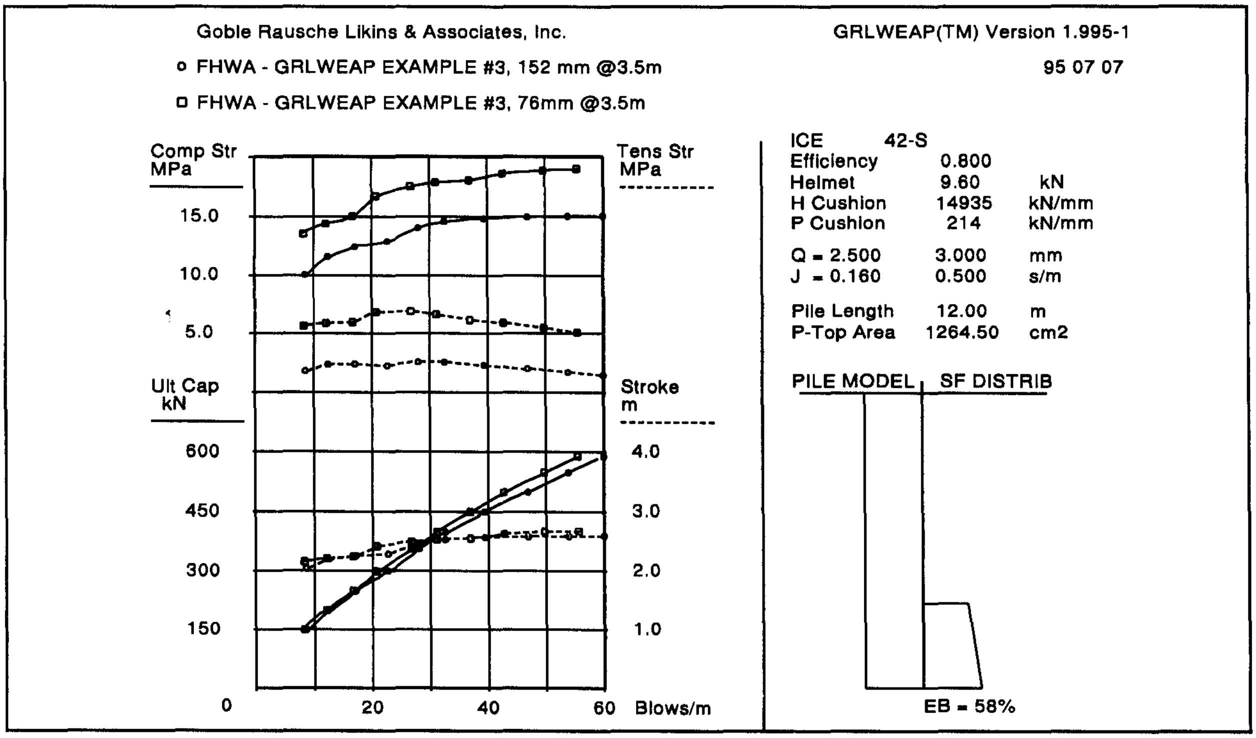


Figure 17.7 Example 3 Bearing Graph Comparison of Two Pile Cushion Thickness

Next, the driving stresses and driving resistance at final driving for the required 1780 kN ultimate pile capacity should be checked. In this analysis, it is assumed that the additional hammer blows required to achieve the final pile penetration depth have resulted in additional compression of the pile cushion material to 75% of its original height, or to a thickness of 114 mm. The additional hammer blows have also resulted in an assumed increase in the modulus of elasticity of the pile cushion material from 207 MPa to 414 MPa. Hence, these assumptions result in the pile cushion stiffness at final driving being approximately 2.7 times stiffer than during early driving.

The analysis indicates a final driving resistance of 236 blows per meter for a 1780 kN ultimate capacity which should result in efficient driving time. Figure 17.8 shows that the maximum compression stresses of 23.3 MPa at final driving are below the allowable limit of 27.0 MPa. Therefore, the thicker pile cushion (152 mm) is recommended for control of both tension stresses during easy driving and compression stresses at final driving.

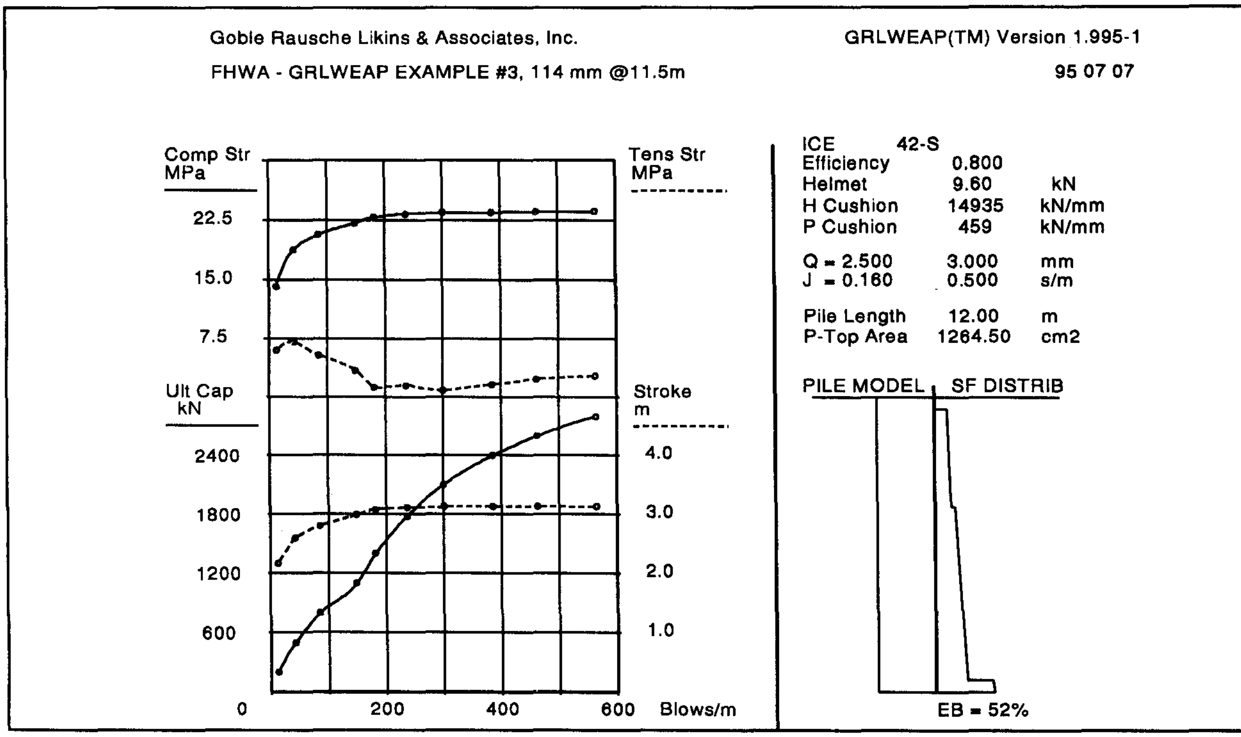


Figure 17.8 Example 3 Bearing Graph for End of Driving Condition

17.5.4 Example 4 - Use of Soil Setup

Consider the soil profile in Figure 17.9. In this example, a 305 mm square prestressed concrete pile is to be driven into a thick deposit of stiff clay. The stiff clay deposit has an average shear strength of 70 kPa. Based on field vane shear tests, it is estimated that the remolded shear strength at the time of driving will be 52.5 kPa, resulting in an expected soil setup factor of 1.33. A static analysis performed using the SPILE program indicates that an ultimate pile capacity of 1340 kN after setup can be obtained for the proposed pile type at a penetration depth of 15 meters. The static analysis also indicates that the ultimate capacity is distributed as 92% shaft resistance and 8% toe bearing resistance, with the shaft resistance being distributed uniformly along the pile shaft.

A Vulcan 08 single acting air hammer was selected by the contractor for driving the prestressed concrete piles. The contractor's hammer submittal indicates that the hammer cushion will consist of 216 mm of Hamortex with a cross sectional area of 958 cm^2. The pile cushion will consist of eight 19 mm sheets of plywood. It is anticipated

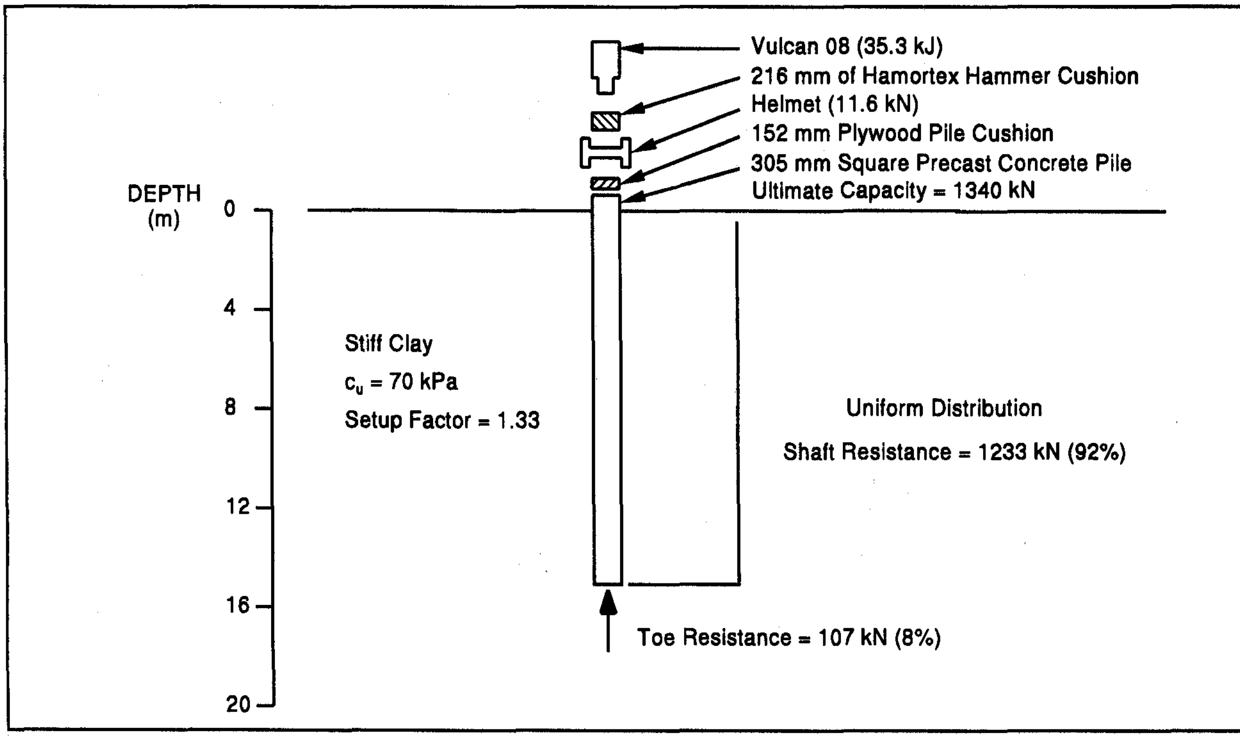


Figure 17.9 Example 4 Problem Profile

that the pile cushion will compress and stiffen during driving similar to that described in Example 3. The contractor's submittal indicates that the helmet weighs 11.6 kN.

Based upon the reported soil type and setup behavior, a 33% increase in pile capacity with time is expected at this site. Therefore, piles could be driven to a capacity of 1005 kN instead of the ultimate capacity of 1340 kN with remaining 335 kN of capacity expected from soil setup. As noted in Section 17.5 of this chapter, a static load test will be performed on the project to confirm the expected pile capacity.

The wave equation results presented in Figure 17.10 indicate a final driving resistance of 138 blows/m could be used as the driving criteria for a 1005 kN capacity. This is significantly less than the 259 blows/m required for an ultimate pile capacity of 1340 kN. Hence, significant pile length may be saved by driving the piles to the lower 1005 kN capacity instead of the required 1340 kN ultimate pile capacity, subject to confirmation of the anticipated soil setup.

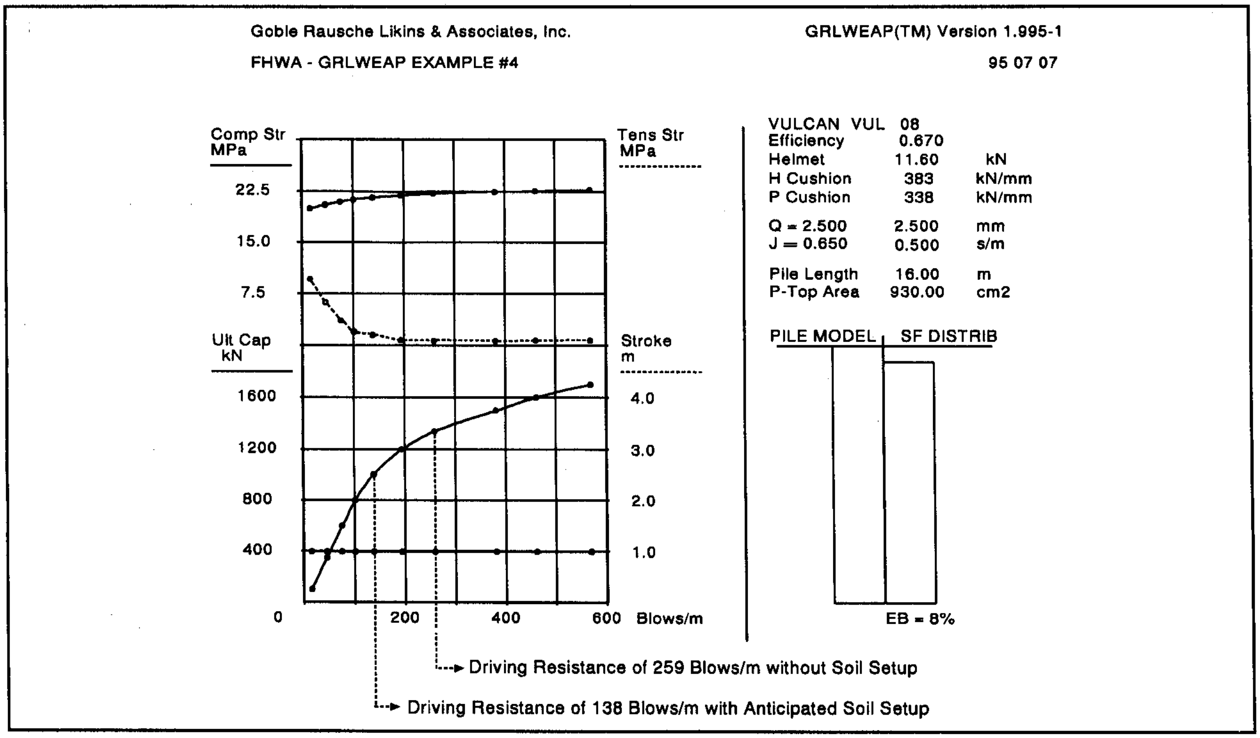


Figure 17.10 Example 4 Using of Bearing Graph with Soil Setup

17.5.5 Example 5 - Driveability Studies

The effect of scour and seismic design considerations on pile foundations often result in increased pile penetration requirements. Therefore, the ability of a given pile to be driven to the depth required by static analysis should be evaluated in a design stage wave equation driveability study, as presented in this example.

Figure 17.11 illustrates the installation conditions at interior Pier 2 of the Peach Freeway design problem from Chapter 13. A cofferdam will be required for pier construction. The interior of the cofferdam will be excavated 5 meters below original grade prior to pile installation. The extremely dense sand and gravel layer was estimated to have a soil friction angle, ϕ, of 43°. This ϕ angle was used in the static calculations of toe resistance. However a limiting ϕ angle of 36°, for hard angular gravel, was used for shaft resistance calculations in this layer as discussed in Section 9.5.

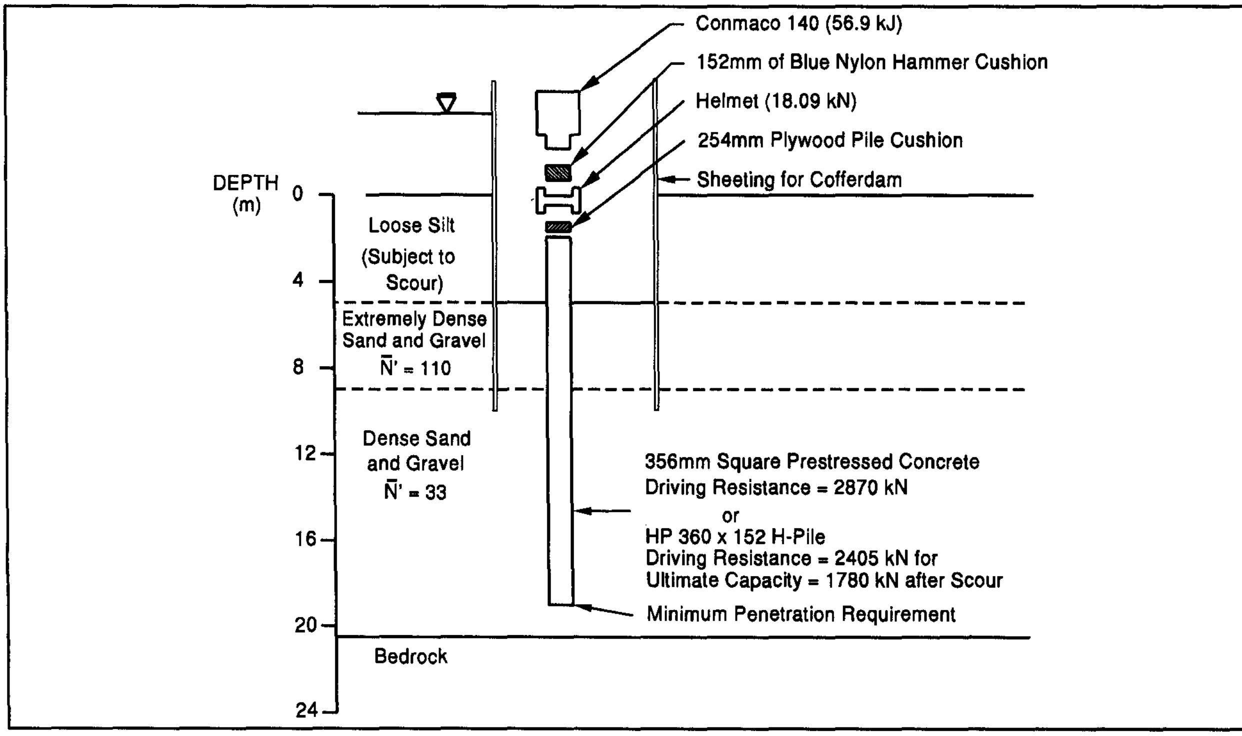


Figure 17.11 Example 5 Problem Profile

During construction, the silt soils will be removed inside the cofferdam area. However, the weight from the silt soils outside the cofferdam will still be present at the time of construction. Therefore, the soil resistance to pile driving should be calculated including the overburden pressure from these materials. However, the 5 meters of loose silt may be completely eroded by channel degradation scour according to hydraulic experts. Thus, for long term pile capacity, static calculations should ignore the effective weight of the silt layer. For the pile installation condition however, the silt layer would be present. Therefore a higher soil resistance than required to meet the static load requirements must be anticipated during pile installation. The soil resistance calculations for the driving conditions should therefore include the effective weight of the silt layer.

Initial static analyses indicate a 356 mm square prestressed concrete pile would develop the ultimate capacity of 1780 kN, primarily through toe bearing, at a depth of 3 meters below the cofferdam excavation level. However, when the reduction in the effective overburden pressure from removal of the silt layer from scour is considered, the piles would have an ultimate capacity of only 924 kN at the 3 meter depth. Additional static capacity calculations were performed at multiple pile penetration depths to determine the depth where a 1780 kN ultimate capacity pile could be obtained. These calculations indicated a 1780 kN ultimate capacity pile could again be obtained through shaft and toe resistance at a depth of 10 meters below cofferdam excavation level. However, once the overburden pressure reduction from channel degradation scour is considered, an ultimate pile capacity of only 1321 kN would be obtained at this depth. To obtain the desired 1780 kN ultimate pile capacity after scour, static calculations indicate a penetration depth of 14 meters below the cofferdam excavation level will be required, and the pile will need to be driven against a soil resistance of 2300 kN to reach this penetration depth.

The static pile capacity calculations versus depth were then input into a wave equation driveability study. Since this study is conducted in the design stage, a locally available single acting air hammer driving system was assumed as a typical driving system that might be used during construction. The driveability analysis indicated that the 356 mm concrete pile would encounter a driving resistance of 255 blows/m in the extremely dense sand and gravel deposit. While this is a relatively high value, the driving resistances decreased considerably after penetrating through this upper stratum and it could be concluded that the 356 mm concrete pile could be driven to the 14 meter penetration depth required.

This would be an erroneous conclusion. The static analyses would likely provide an adequate assessment of soil resistance for the first pile driven. However, an increase in the ϕ angle from group densification could significantly affect the resistance to driving of additional displacement piles, particularly within the added confinement from the cofferdam. The dense deposits are also likely to develop negative pore pressures during shear, resulting in a temporary increase in soil resistance to driving. If it is assumed that these factors cause a 33% increase in both shaft and toe resistances during driving of subsequent piles, a second driveability analysis would indicate that the piles practically refuse at a penetration depth of 3.5 meters with a driving resistance of 582 blows/m. A soil resistance of 2870 kN must also be overcome during driving to reach the 14 m penetration depth. Maximum compression driving stresses approach 31 MPa so a larger hammer would not appear to be a viable option. If displacement piles are used, predrilling or jetting would be required to advance the piles through the upper stratum. A low displacement pile would be a more attractive foundation solution.

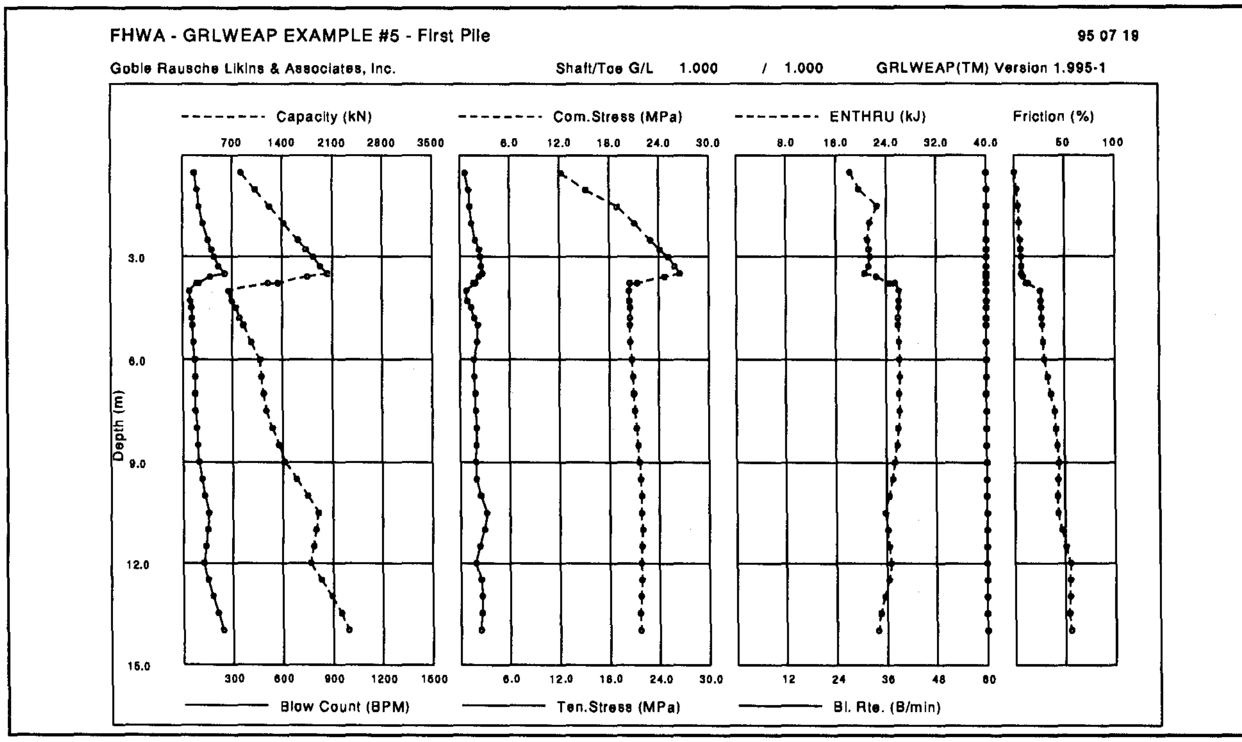


Figure 17.12 Example 5 Driveability Results for First 356 mm Concrete Pile

The wave equation driveability results for the first and later 356 mm concrete piles driven in the above soil profile are presented in Figures 17.12 and 17.13, respectively. Wave equation driveability results for a low displacement HP 360 x 152 H-pile are shown in Figure 17.14. Note that the penetration depths in these figures correspond to the depth below cofferdam excavation level. The maximum driving resistance calculated for the H-pile to penetrate the extremely dense sand and gravel stratum is only 94 blows/m. Compression driving stresses do not exceed 218 MPa and are therefore within the recommended limits for H-piles given in Chapter 11. Based on the driveability study results, the low displacement H-pile would be a preferable foundation design. The results also indicate the H-pile could be driven to bedrock, and therefore likely for a higher ultimate pile capacity.

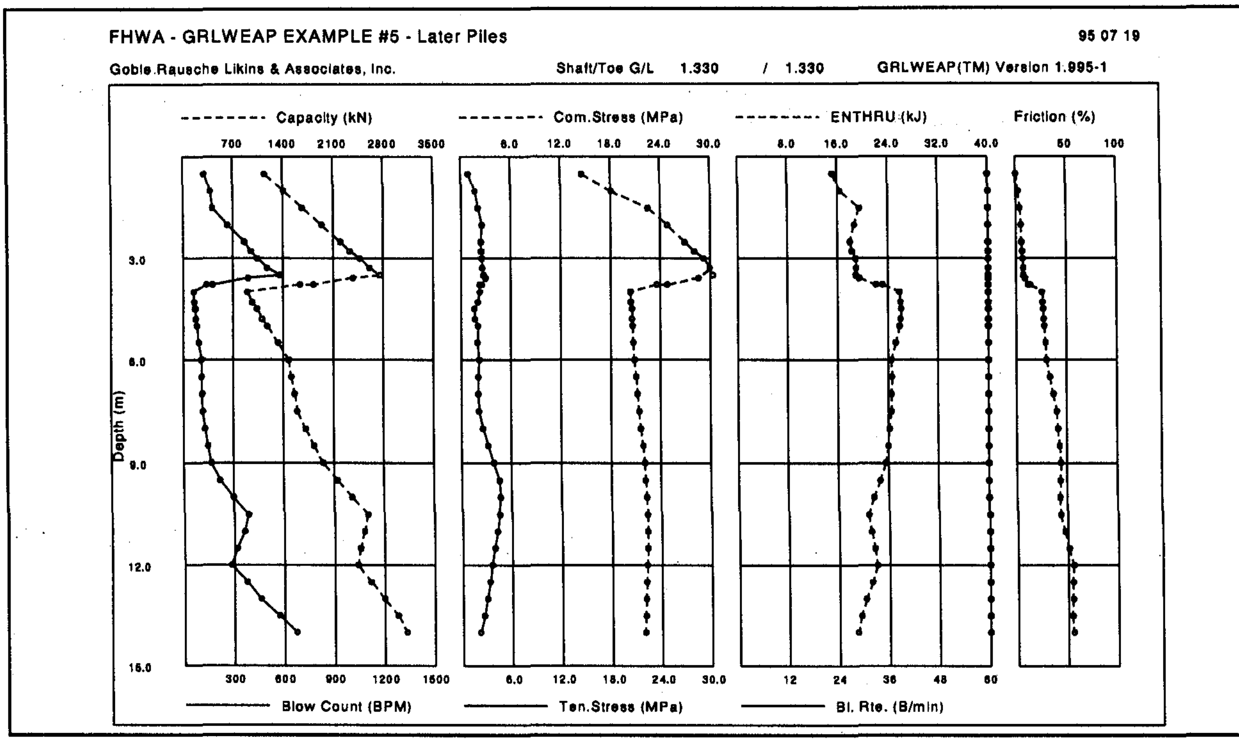


Figure 17.13 Example 5 Driveability Results for Later 356 mm Concrete Piles with Densification

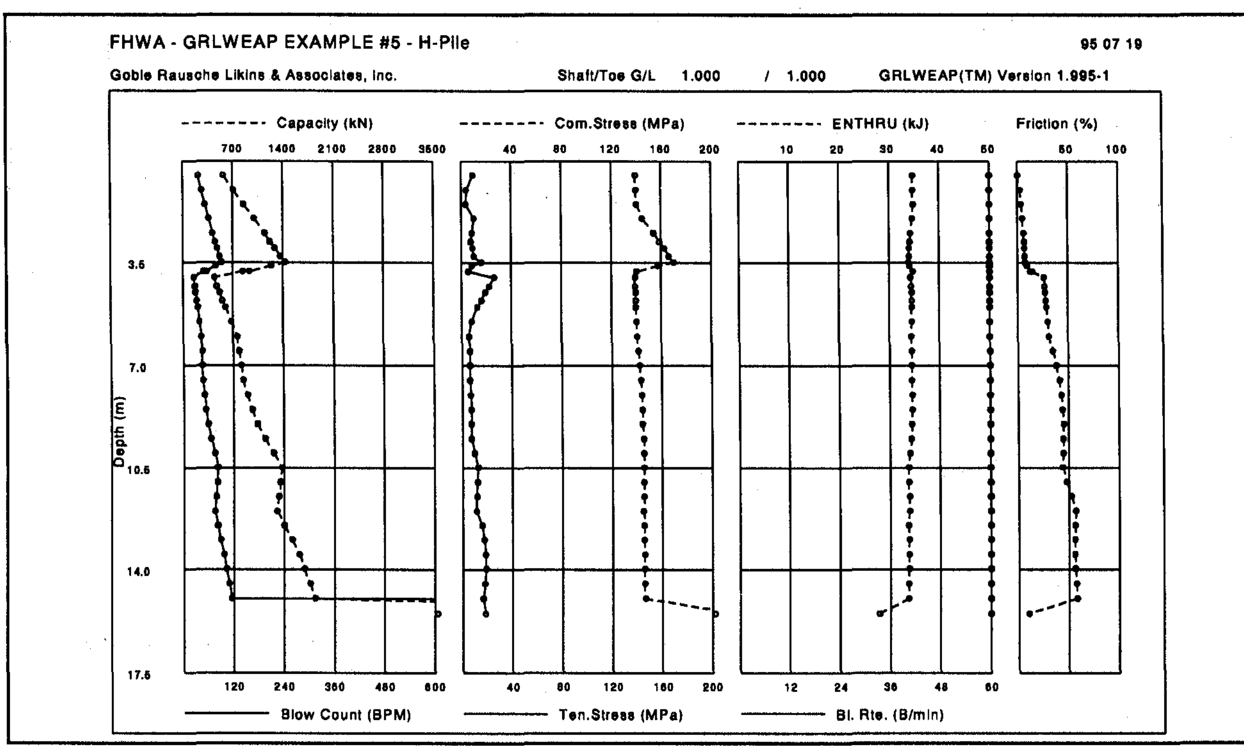


Figure 17.14 Example 5 Driveability Results for H-Pile

17.5.6 Example 6 - Driving System Characteristics

Example 6 presents a wave equation comparison of two hammers having the same potential energy. Both Engineering News and Gates dynamic formulas consider only the potential energy of the driving system. The driving resistance required for a specific capacity by either of these formulas would be the same as long as two hammers had the same potential energy. The wave equation predicted driving resistances for the two hammers in the same pile-soil condition is, however, quite different.

In this example problem, a 356 mm by 9.5 mm wall closed end pipe pile is to be driven to an ultimate pile capacity of 1800 kN. The pile has a furnished length of 20 meters and an embedded length of 16 meters. A static analysis indicates that the soil resistance distribution will be 30% shaft resistance and 70% toe resistance. The shaft resistance will be distributed triangularly along the embedded portion of the pile shaft. Experience has shown that the materials near the pile toe are highly elastic and therefore have a larger (7.5 mm) than normal (3.0 mm) toe quake. The example problem's soil profile is presented in Figure 17.15.

The contractor is considering using either a Vulcan 014 or an ICE 42-S to drive the piles. Both hammers have a rated energy of about 57 kJ. For the Vulcan 014 hammer, the rated energy is based upon a 62.3 kN ram and a 0.91 m stroke whereas the ICE 42-S hammer is rated based upon a 18.2 kN ram and a 3.13 m stroke. The helmet weights for the Vulcan 014 and ICE 42-S are 7.45 and 9.12 kN, respectively. The contractor indicates that for the Vulcan 014, the hammer cushion will consist of 152 mm of Nycast with an elastic modulus of 1428 MPa, and a cross sectional area of 1508 cm^2. For the ICE 42-S, the hammer cushion will consist of 51 mm Blue Nylon, which has an area of 2568 cm^2, and an elastic modulus of 1257 MPa.

Wave equation results for the two hammers are plotted on the same bearing graph in Figure 17.16. For the pile and soil condition analyzed, the Vulcan 014 (Hy Ram) requires a driving resistance of 275 blows/m for an 1800 kN ultimate pile capacity whereas the ICE 42-S (Lt Ram) requires a driving resistance of 533 blows/m. Hence, even though both hammers have the same potential energy, the required driving resistance for an 1800 kN ultimate pile capacity is quite different. The Vulcan 014 requires a lower driving resistance because the duration of the hammer blow is longer and is more efficient in advancing the pile in this particular example of a highly elastic soil.

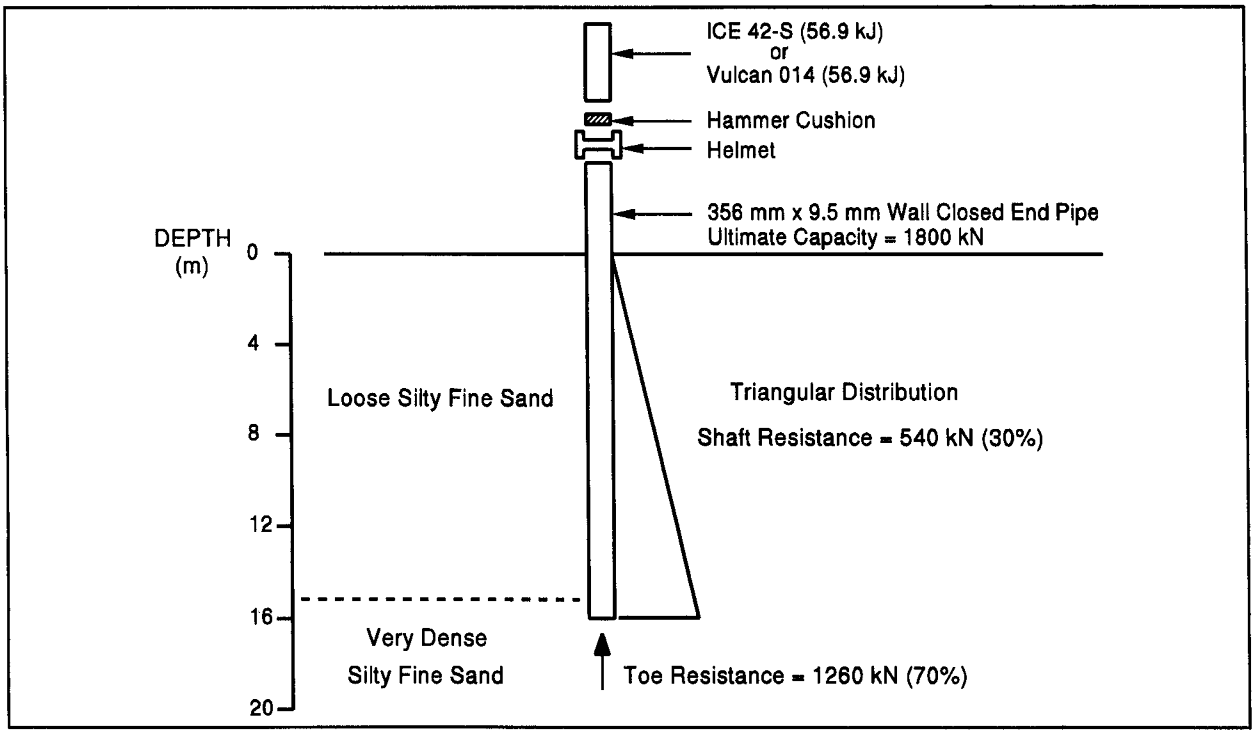


Figure 17.15 Example 6 Problem Profile

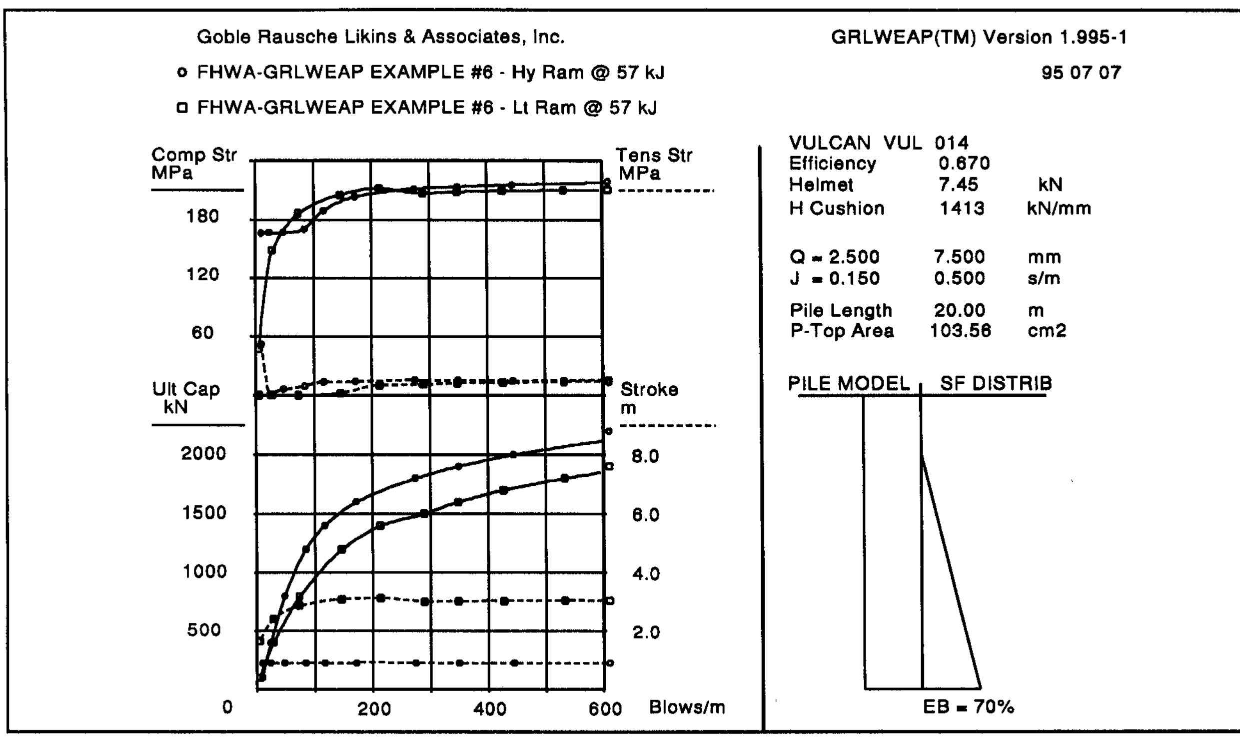


Figure 17.16 Example 6 Bearing Graph Comparison of Two Hammers with Equivalent Potential Energy

This example illustrates the dynamic complexities of hammer-pile-soil interaction. Clearly the potential energy alone, which is all that is considered in dynamic formulas, is not an adequate assessment of pile driveability.

17.5.7 Example 7 - Assessment of Pile Damage

Another pile driving construction problem is pile damage. It is frequently assumed that steel H-piles can be driven through boulders and fill materials containing numerous obstructions. Investigations reveal this is not true. H-piles without commercially manufactured pile toe reinforcement are one of the most easily damaged pile types. The damage occurs because of the ease with which flanges can be curled, rolled and torn. Pile damage has detrimental effects on both driving resistance and ultimate capacity.

This example will illustrate how the wave equation was used to obtain insight into a construction problem involving pile damage. The project conditions are shown in Figure 17.17. The HP 310 x 79 H-piles were 10.5 meters in length with a design load of 845 kN and an ultimate pile capacity of 1690 kN. The soil profile consisted of 4.5 to 5.0 m of miscellaneous fill, including some bricks and concrete. Below the fill was 4.5 m of silty clay that increased in strength with depth. The clay overlaid bedrock which was encountered at a depth of about 10 m.

The contractor selected an MKT DE-40 single acting diesel hammer with a rated energy of 43.4 kJ to drive the piles. Using the Engineering News formula specified in the contract documents, the required driving resistance was 590 blows/m for this hammer. Figure 17.18 presents the wave equation results. These results indicate that the maximum compression stress at a driving resistance of 590 blows/m was 285 MPa, which is well in excess of the 248 MPa yield stress of the pile material. The wave equation results indicate this maximum compression stress is located at the pile toe. In addition, the pile capacity at that driving resistance is well in excess of the 1690 kN required ultimate capacity. The wave equation results clearly indicate the Engineering News formula driving criteria resulted in significant overdriving of the piles at very high driving stress levels.

In accordance with the contract requirement, several load tests were conducted. In all cases the piles failed to carry the 1690 kN ultimate capacity, in spite of the fact that several of the piles were driven to a dynamic resistance exceeding 800 blows/m with no indication of damage at the pile head. As a result of the high driving resistances to

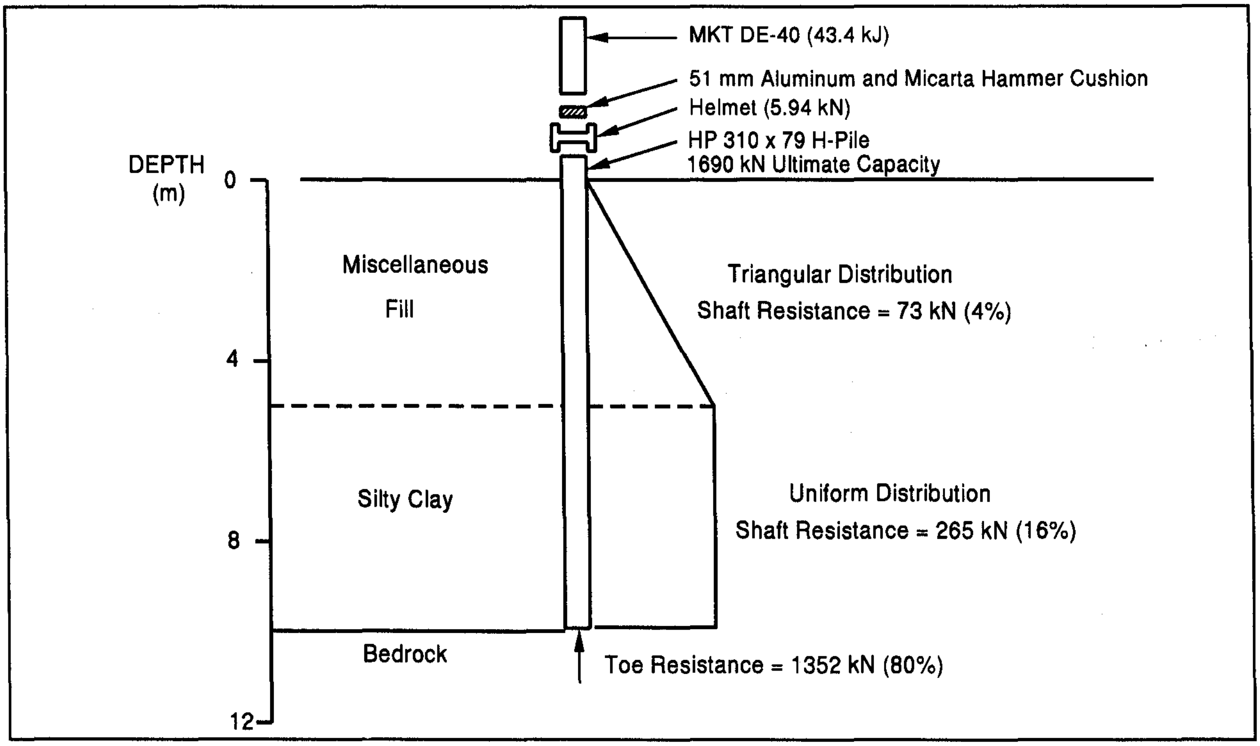


Figure 17.17 Example 7 Problem Profile

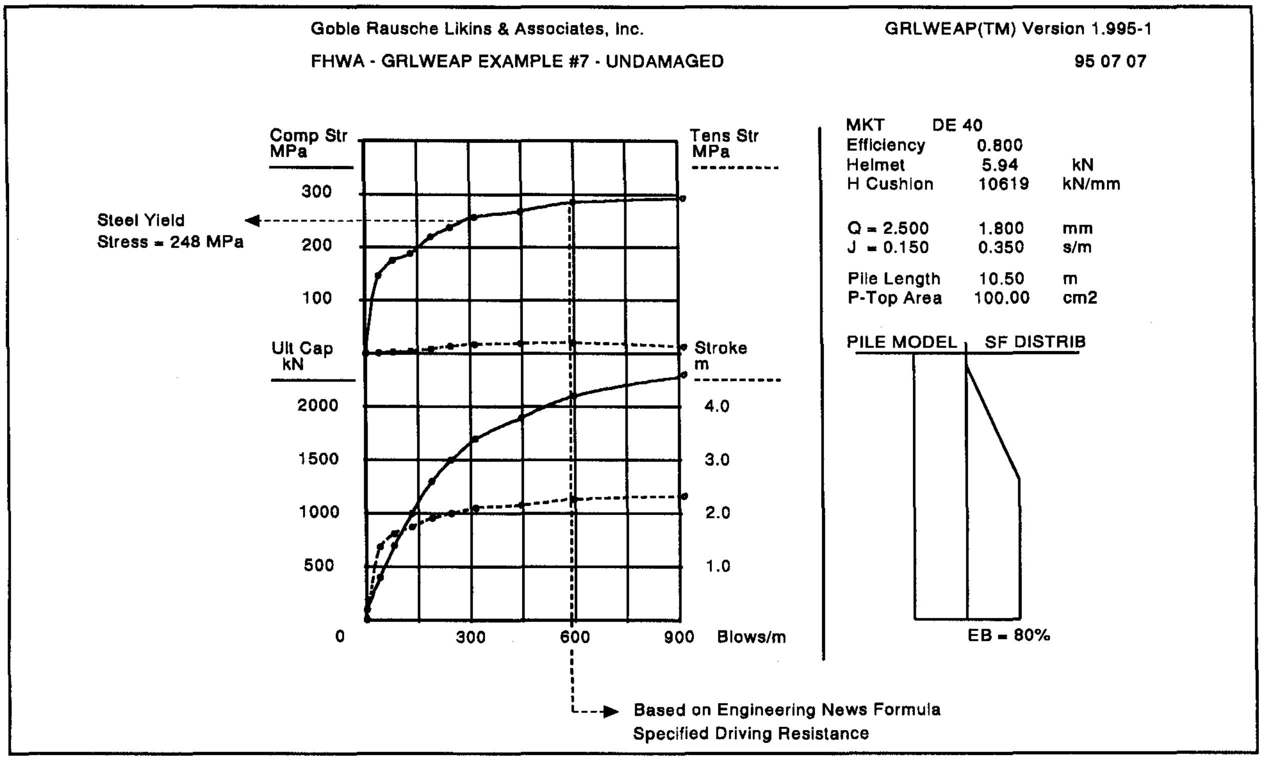


Figure 17.18 Example 7 Wave Equation Bearing Graph for Proposed Driving System

which several piles were driven, it was apparent that additional driving would not result in satisfactory pile performance. Consequently, the contractor was requested to pull several of the piles to check for possible damage. Upon extraction, it was noted that the piles were severely damaged. The flanges were separated and rolled up from the web. The damage occurred as the unprotected piles were driven through the miscellaneous rubble fill.

The effect of the damage on pile driveability can be evaluated with a wave equation analysis. Records indicate piles driven as hard as 800 blows/m did not support the 1690 kN ultimate capacity. Hence, this provides a reference point on the wave equation bearing graph on the driveability of a damaged pile. The bearing graph for the damaged pile was determined by adjusting the stiffness of the lower pile segment until the results agreed with the driving resistance and capacity observations. The resulting toe segment stiffness was roughly only 10% of that of an undamaged pile.

Figure 17.19 presents wave equation results for both an undamaged pile and a damaged pile. The results indicate that the ultimate load of 1690 kN could not be obtained for the damaged pile, regardless of the magnitude of the driving resistance. Essentially, the damaged pile section "cushioned" the hammer blow and attenuated the hammer energy. Once damaged, the soil resistance at the pile toe could not be overcome and, therefore the pile would not advance. This illustrates that driving stresses can also limit the driveability of a pile to the required ultimate capacity.

The pile damage potential on this project could have been greatly reduced if a wave equation had been performed during the design stage or had been specified for construction control. The wave equation bearing graph in Figure 17.18 illustrates that the ultimate capacity of 1690 kN could be obtained by the contractor's driving system at a driving resistance slightly greater than 300 blows/m. The compression driving stress at this driving resistance is slightly above the steel yield strength of 248 MPa. Hence the potential damage problem would have been clearly apparent at the time of the contractor's hammer submittal. Additional wave equation analyses of the contractor's driving system could have been performed to determine if driving stress levels could be reduced to acceptable levels by using reduced fuel settings and shorter hammer strokes. If driving stresses could not be controlled in this manner, approval of the proposed driving system should not have been obtained, and alternate hammers should have been evaluated.

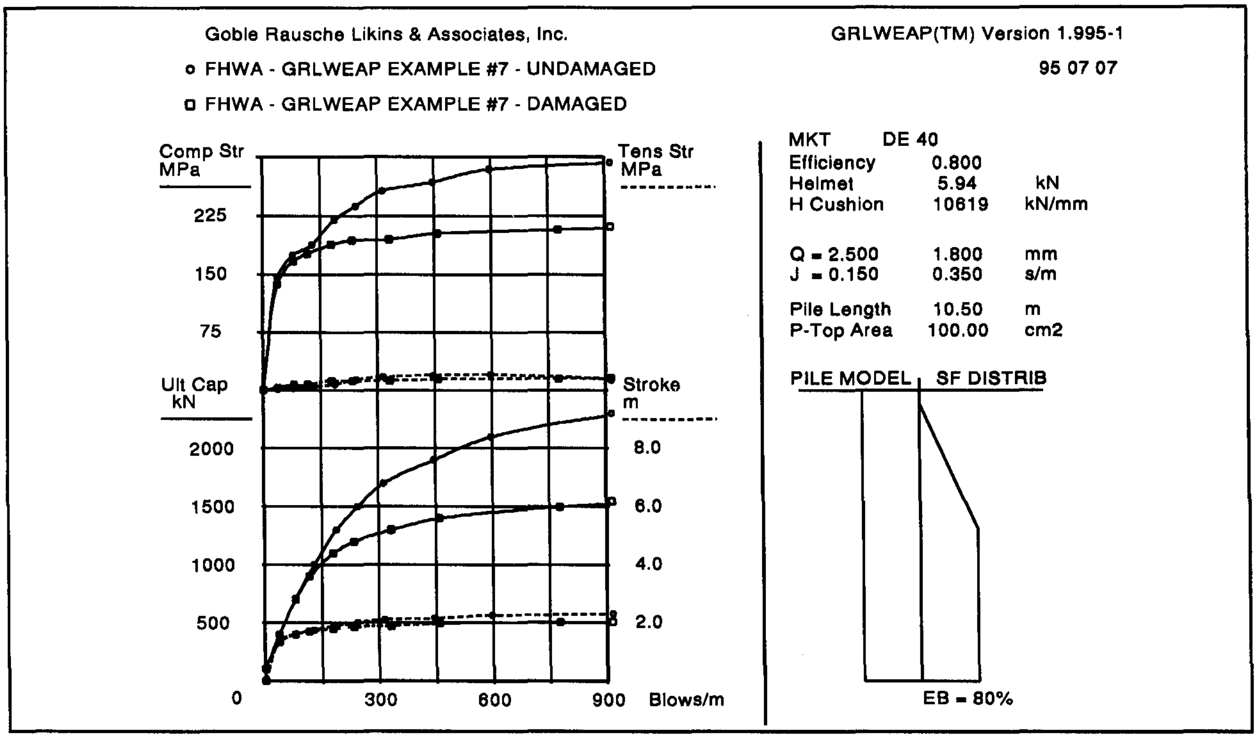


Figure 17.19 Example 7 Comparison of Wave Equation Bearing Graphs for Damaged and Undamaged Piles

17.5.8 Example 8 - Selection of Wall Thickness

This wave equation example demonstrates the evaluation of the required wall thickness for a pipe pile. Consider the soil and problem profile presented in Figure 17.20. Based upon static analyses and structural loading conditions, a 324 mm outside diameter closed end pipe pile with a design load of 665 kN is selected as the pile foundation type. Static analysis indicates the overlying unsuitable layers provide 140 kN of resistance. Hence the required ultimate pile capacity is 1470 kN. The calculated embedded pile length for this ultimate capacity is 14 m.

Since this is a design stage issue, the actual hammer and driving system configuration is unknown. Therefore, a typical hammer size and driving system configuration must be assumed with consideration of typical, locally available equipment as well as the calculated soil resistance at the time of driving. These factors led to the selection of a Berminghammer B-225 single acting diesel hammer with a rated energy of 39.7 kJ.

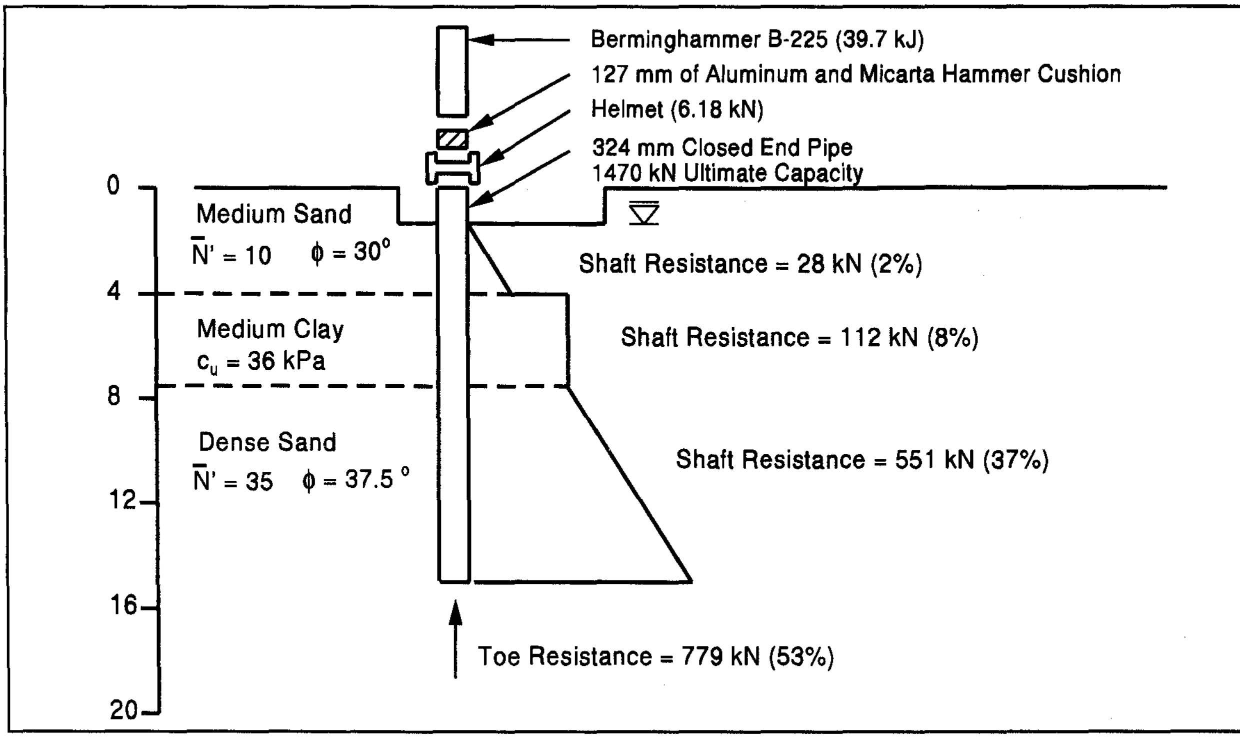


Figure 17.20 Example 8 Problem Profile

Wave equation analyses were performed for a 324 mm outside diameter pipe pile with wall thicknesses of 6.3, 7.1, 7.9 and 9.5 mm. Figures 17.21 and 17.22 present the results of these analyses. For the 6.3 mm wall thickness, the wave equation results indicate that a driving resistance of 615 blows/m will be required for the ultimate capacity of 1470 kN and that compression driving stresses approach 256 MPa. While this compression driving stress level is acceptable for Grade 3 pipe with a yield strength of 310 MPa, the 6.3 mm wall thickness pipe does not have suitable <u>driveability</u> for the project conditions. (As per Chapter 12, suitable driveability is a driving resistance between 30 and 120 blows per 0.25 meter or 120 and 480 blows/m.)

Wave equation results for the 7.1 mm wall thickness indicate that a driving resistance of 487 blows/m will be encountered for the ultimate capacity of 1470 kN and that compression driving stresses approach 225 MPa. While the driving stresses are again within acceptable limits for Grade 3 pipe, the driving resistance is still considered high and exceeds the 120 to 480 blow/m acceptance criteria. Differences between actual and assumed soil parameters (resistance distribution, quake, and damping) could easily increase the required driving resistance in the field and turn the high driving resistance into a refusal driving situation.

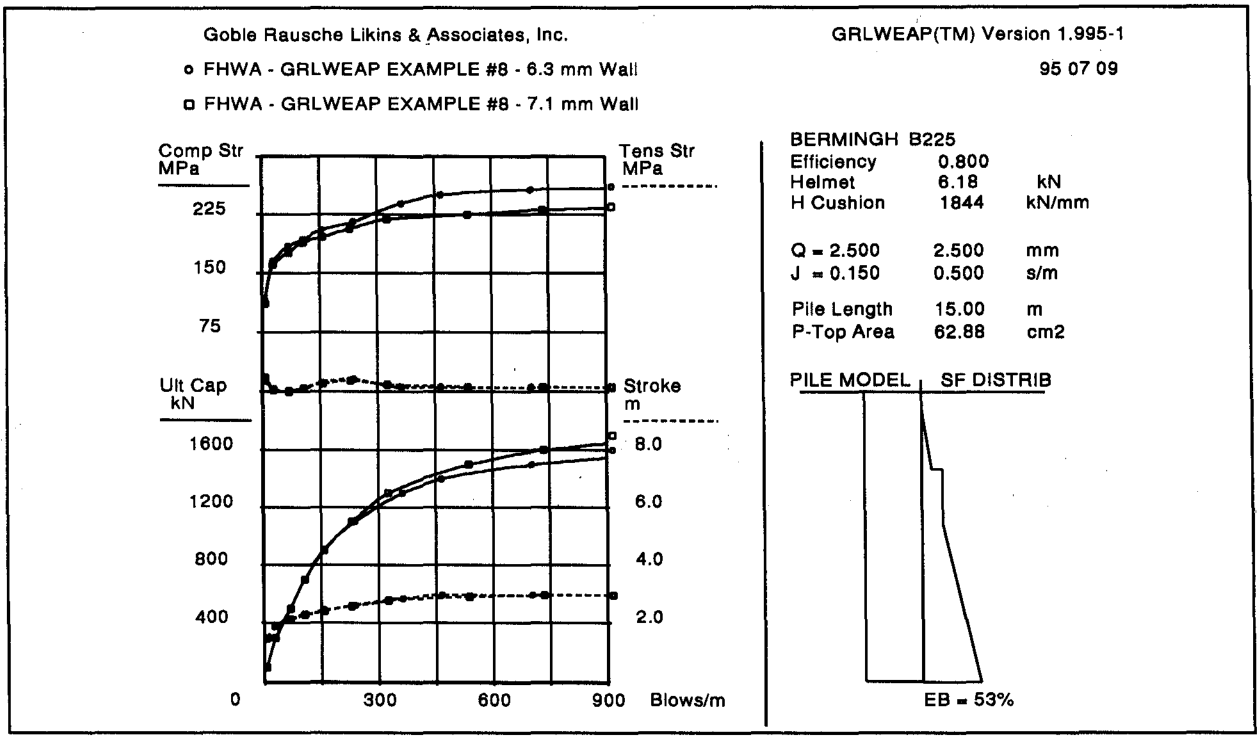


Figure 17.21 Example Bearing Graphs for 6.3 and 7.1 mm Wall Pipe Piles

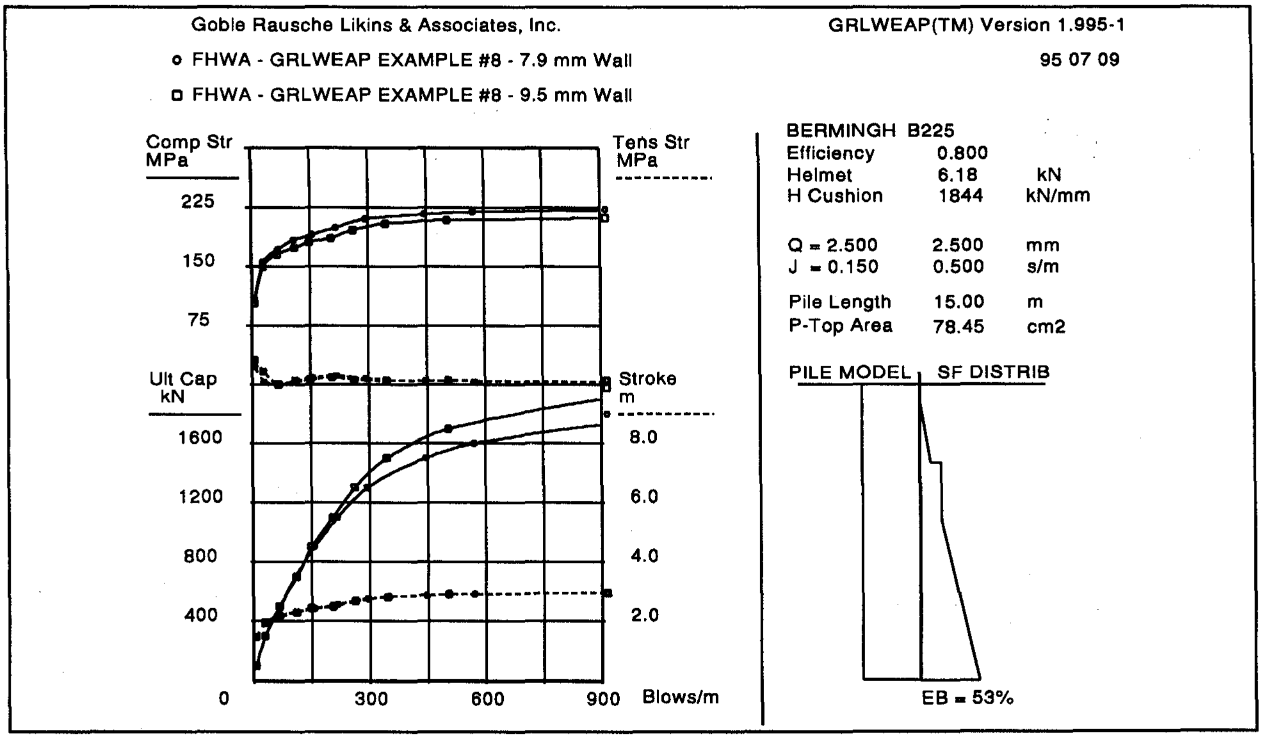


Figure 17.22 Example 8 Bearing Graph for 7.9 and 9.5 mm Wall Pipe Piles

For the 7.9 mm wall thickness, wave equation results indicate that a driving resistance of 412 blows/m is required for the ultimate capacity of 1470 kN and that compression driving stresses approach 217 MPa. Both the driving resistance and driving stresses are now within acceptable limits. Therefore, the 7.9 mm wall thickness pipe has the suitable driveability for the required capacity and is considered an acceptable foundation design. The 9.3 mm wall thickness pipe has even greater driveability and could also be chosen for reason of time savings during installation or other design considerations.

17.5.9 Example 9 - Evaluation of Vibratory Driving

This example will illustrate the use of a wave equation analysis for evaluating vibratory hammer installation of the sheet piles required for cofferdam construction in Example 5. The sheet piles of Example 5 have to be installed using a vibratory hammer. The contractor has an ICE 815 hammer available and intends to drive pairs of PZ27 sheet piles whose combined cross sectional area is 154 cm^2. These are Z-section sheets, each with a width of 460 mm, a depth of 300 mm, and a thickness of 10 mm. At the time of sheet pile installation, the soil within the cofferdam is not excavated and the piles are therefore driven from mudline to an estimated depth of 10 m. The sheet pile length is 15 m.

An evaluation of the static soil resistance by the effective stress method has been demonstrated earlier. For the non-excavated condition, first a 5 m thick layer of soft silt has to be penetrated by the sheet piles, followed by the extremely dense sand and the dense sand and gravel layers. The corresponding calculated soil resistance and the associated dynamic soil parameters are shown in Figure 17.23(a). Note that soil damping has been set to twice the normal values to model, in a very approximate manner, the effects of lock friction.

Depth m	Skin Frictn kPa	End Bearing kN	Skin Quake mm	Toe Quake mm	Skin Damping s/m	Toe Damping s/m
0.00	.00	.00	2.500	2.500	1.300	1.000
5.00	7.70	8.00	2.500	2.500	1.300	1.000
5.00	10.70	27.00	2.500	2.500	.330	1.000
9.00	29.90	77.00	2.500	2.500	.330	1.000
9.00	27.70	77.00	2.500	2.500	.300	1.000
15.00	50.70	140.00	2.500	2.500	.330	1.000

Figure 17.23(a) Example 9 Soil Resistance Information for Vibratory Sheet Pile Driving

Analyses are performed for pile penetration depths between 4 and 10 m at 1 m increments. For the driveability analysis, the statically calculated resistance values are directly used without any gain or loss factors. This might be in contrast to common experience which indicates that a large percentage of a soil's shaft resistance is lost due to soil vibration when driving with a vibratory hammer. The analysis therefore presents a worst case situation and would be particularly interesting to the contractor.

Figure 17.23(b) lists the hammer model, which consists of two masses and an elastomer connection modeled by a spring with 140 kN/mm spring stiffness. The product of the listed eccenter weight and the eccentric radius equals the hammer's rated moment. As per input, the frequency has been chosen at 16 Hz (960 RPM) even though the hammer is capable of running at 26 Hz. Also, an efficiency value of 0.8 and a start-up time of 0.1 second have been entered. In reality, the start-up time is probably much longer, as it includes the elapsed time between turning the hammer on and reaching full frequency in the hammer. However, for all except the first penetration analyzed, the start-up time is really non-existent as the hammer is run continuously. Furthermore, the start-up time has no significant effect on the results and is primarily important for reaching a numerically satisfactory solution. Finally, a 25 kN line pull (an upward directed crane force) has been entered to keep the hammer/pile system stable. This line pull effectively reduces the dead weight of the hammer-pile system, which plays a major role in advancing the pile. Probably, during harder driving, the operator will let the line slacken which will allow for a significant increase of the speed of pile penetration. Of course, the crane operator will not be able to maintain an exact line pull force and additional analyses should be run to check the effect of this force on the sheet pile penetration.

Vibratory ICE 815, 2 PZ 27 Sheet Piles

Hammer Model of: 815 Made by: ICE

No.	Weight kN	Stiffn kN/mm	CoR	C-Slk mm	Dampg kN/m/s
1	35.600				
2	41.720	140.0	1.000	3.0480	1425.6

Top Weight	(kN)	35.60	Bottom Weight + Clamp	(kN)	41.72
Connect. Stiffness	(kN/mm)	140.00	Connect. Dashpot	(kN/m/s)	1425.6
Eccenter Radius	(m)	.112	Eccenter Weight	(kN)	4.450
Line Pull	(kN)	25.00	Actual Target Frequ.	(1/s)	16.00
Efficiency		.8000	Rated Power	(kW)	375.0
Start-Up Time	(s)	.1000			

Figure 17.23(b) Example 9 Vibratory Hammer Model and Hammer Options

Figure 17.23(c) summarizes pile and soil model. For the first analyzed depth of 4 m, the static capacity of 12.1 kN is very small. This capacity was obtained after subtracting the weight of the sheet pile section extending above grade from the statically calculated pile capacity. Considering the hammer weight of nearly 80 kN minus line pull of 25 kN, the sheet pile will penetrate very rapidly at this depth as indicated in the final result table in Figure 17.23(d). After the pile penetrates into the sand layer, the required penetration time will increase, eventually reaching 46 seconds for 1 m at a penetration of 10 m. The calculated total time of penetration is 1.4 minutes, and although this result is subject to many uncertainties, it can be concluded that the hammer is easily capable of driving the sheet pile pairs to the design depth.

Depth 1 at 4.0 m; Dead Load 12.1 kN; Shaft/Toe G/L: 1.000/ 1.000

PILE PROFILE:

L b Top	Area	E-Mod	Spec Wt	Circumf	Strength	Wave Sp	EA/c
m	cm2	MPa	kN/m3	m	MPa	m/s	kN/m/s
.00	153.6	210000.	78.50	3.150	248.00	5123.	629.45
15.00	153.6	210000.	78.50	3.150	248.00	5123.	629.45

Wave Travel Time - 2L/c - = 5.856 ms

Pile and Soil Model for Rut = 33.2 kN

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Circmf	Area
	kN	kN/mm	mm	mm		kN	s/m	mm	m	m	cm2
1	1.205	3225.	3.000	.000	.85	.0	.000	2.50	1.00	3.2	153.6
2	1.205	3225.	.000	.000	1.00	.0	.000	2.50	2.00	3.2	153.6
3	1.205	3225.	.000	.000	1.00	.0	.000	2.50	3.00	3.2	153.6
12	1.205	3225.	.000	.000	1.00	1.8	1.300	2.50	12.00	3.2	153.6
13	1.205	3225.	.000	.000	1.00	5.3	1.300	2.50	13.00	3.2	153.6
14	1.205	3225.	.000	.000	1.00	8.9	1.300	2.50	14.00	3.2	153.6
15	1.205	3225.	.000	.000	1.00	12.5	1.300	2.50	15.00	3.2	153.6
Toe						4.7	1.000	2.50			

PILE, SOIL, ANALYSIS OPTIONS:

Uniform/Non-Uniform/2-Pile	0	Pile Segment Generation	Automatic
No. of Slacks/Splices	0	Pile Damping (%)	1
		Pile Damping Fact.(kN/m/s)	12.589
Soil Damping Option	Smith	Soil Damping Exponent	1.000
Soil Resistance Distr. No.	0		
Max No Analysis Iterations	0	Time Increment/Critical	160
Residual Stress Analysis	0	Output Option	25
Output Segment Generation	Automatic	Output Time Interval	10
Analysis Time-Input (ms)	0		

Figure 17.23(c) Example 9 Pile and Soil Model and Options

SUMMARY OVER DEPTHS

G/L at Shaft and Toe: 1.000 1.000

Depth	Rut	Frictn	End Bg	PenTime	max Str	min Str	Power
m	kN	kN	kN	s/m	MPa	MPa	kW
4.0	33.2	28.5	4.7	1.3	11.403	-5.946	44.5
5.0	57.8	51.1	6.7	3.7	13.216	-4.822	22.0
6.0	131.8	95.0	36.8	6.8	18.043	-6.560	30.5
7.0	201.8	151.9	49.9	9.2	22.229	-12.856	44.9
8.0	287.1	224.1	62.9	13.4	27.848	-21.055	64.9
9.0	387.4	311.6	75.8	24.3	35.571	-29.803	88.9
10.0	492.4	405.7	86.7	46.0	44.184	-37.742	110.2

Total Driving Time 1.40 minutes

Driving time for continuously running hammer; any waiting times not included

Figure 17.23(d) Example 9 Final Summary Table

17.6 ANALYSIS DECISIONS FOR WAVE EQUATION PROBLEMS

17.6.1 Selecting the Proper Approach

Even though the wave equation analysis is an invaluable tool for the pile design process, it should not be confused with a static geotechnical analysis. The wave equation does not determine the capacity of a pile based on soil boring data. The wave equation calculates a driving resistance for an assumed ultimate capacity, or conversely, it assigns an estimated ultimate capacity to a pile based on a field observed driving resistance. It is one thing to perform a wave equation bearing graph for a certain capacity and a totally different matter to actually realize that capacity at a certain depth. The greatest disappointments happen when pile lengths required during construction vary significantly from those computed during design. To avoid such disappointments, it is absolutely imperative that a static analysis, as described in Chapter 9, precede the wave equation analysis. The static analysis will yield an approximate pile penetration for a desired capacity or a capacity for a certain depth. The static analysis can also generate a plot of estimated pile capacity as a function of depth. It is important that the static analysis evaluates the soil resistance in the driving situation (*e.g.* remolded soil strengths, before excavation, before scour, before fill placement, *etc.*).

After the static analysis has been completed, a wave equation analysis can be performed leading either to a bearing graph or to driving resistances and stresses versus depth (driveability). Sometimes both analyses are performed. The bearing graph analysis is only valid within the proximity of the analyzed soil profile depending on the variability of the soil properties. The driveability analysis calculates driving resistances and stresses for a number of penetration depths, and therefore provides a more complete result. However, there is a very basic difference between these two approaches. The bearing graph approach allows the engineer to assess pile capacity given a driving resistance at a certain depth. The driveability analysis points out certain problems that might occur during driving. If the pile actually drives differently from the wave equation predictions, then a reanalysis with different soil resistance parameters would be needed to match the observed behavior.

Even though an accurate static analysis and a wave equation analysis have been performed with realistic soil parameters, the experienced foundation engineer would not be surprised if the driving resistance during pile installation were to differ substantially from the predicted one. Most likely the observed driving resistance would be lower than calculated. As an example, suppose that a 500 kN pile had to be driven into a clay. With a factor of safety of 2.5, the required ultimate capacity would be 1250 kN. The static soil analysis indicates that the pile has to be 25 m long for this ultimate capacity. There would be negligible toe resistance, and based upon remolded soil strength parameters, the soil may exhibit only 50% of its long term strength during driving. It is therefore only necessary to drive the pile to a capacity of 625 kN, which should be achieved at the 25 m depth. The expected end of installation driving resistance would then correspond to 625 kN. In a restrike test, say 7 days after installation, the 1250 kN capacity would be expected, and therefore a much higher driving resistance would be encountered than observed at the end of driving.

The above discussion points out one major reason for differences between analysis and reality. However, as with all mathematical simulations of complex situations, agreement of wave equation results with actual pile performance depends on the realism of the method itself, and on the accuracy of the model parameters. The accuracy of the wave equation analysis will be poor when either soil model or soil parameters inaccurately reflect the actual soil behavior, and when the driving system parameters do not represent the state of maintenance of hammer or cushions. The pile behavior is satisfactorily represented by the wave equation approach in the majority of cases. A review of potential wave equation error sources follows.

17.6.2 Hammer Data Input, External Combustion Hammers

The most important input quantity is the hammer efficiency. It is defined as that portion of the potential ram energy that is available in the form of kinetic ram energy immediately preceding the time of impact. Many sources of energy loss are usually lumped into this one number. If the hammer efficiency is set too high, then an optimistically low driving resistance would be predicted. This in turn could lead to overpredictions of ultimate pile capacity. If the efficiency is set very low, for conservative pile capacity assessments, then the stresses may be underpredicted, leading to possible pile failures during installation.

Hammer efficiency should be reduced for battered pile driving. The efficiency reduction depends on the hammer type and batter angle. For hammers with internal ram energy measurements, no reductions are required. Modern hydraulic hammers often allow for a continuously adjustable ram kinetic energy which is measured and displayed on the control panel. In this case the hammer efficiency does not have to cover friction losses of the descending ram, but only losses that occur during the impact (*e.g.* due to improper ram-pile alignment) and it may therefore be relatively high (say 0.95). For such hammers, the wave equation analysis can select the proper energy level for control of driving stresses and economical driving resistances by trying various energy (stroke) values which are lower than the rated value.

Similarly, a number of air/steam hammers can be fitted with equipment that allows for variable strokes. The wave equation analysis can then help to find that driving resistance at which the stroke can be safely increased to maximum. It is important, however, to realize that the reduced stroke is often exceeded, and that the maximum stroke not fully reached. Corresponding increases and decreases of efficiency for the low and high stroke may therefore be necessary.

17.6.3 Hammer Data Input, Diesel Hammers

The diesel hammer stroke increases when the soil resistance, and therefore driving resistance, increases. GRLWEAP simulates this behavior by trying a down stroke, and when the calculated up stroke is different, repeats the analysis with that new value for the down stroke. The accuracy of the resulting stroke is therefore dependent on the realism of the complete hammer-pile-soil model and should therefore be checked in the field by comparison with the actual stroke. The consequences of an inaccurate stroke could be varied. For example, an optimistic assumption of combustion pressure could lead to high stroke predictions and therefore to non-conservative predictions of ultimate pile capacity while stress estimates would be conservatively high (which may lead to a hammer rejection).

Stroke and energy transferred into the pile appear to be closely related, and large differences (say more than 10%) between stroke predictions and observations should be explained. Unfortunately, higher strokes do <u>not always</u> mean higher transferred energy values. When a hammer <u>preignites</u>, probably because of poor maintenance, then the gases combusting before impact slow the speed of the descending ram and cushion its impact. As a result, only a small part of the ram energy is transferred to the pile. A larger part of the ram energy remains in the hammer producing a high stroke. If, in this case, the combustion pressure would be calculated by matching the computed with the observed stroke under the assumption of a normally performing hammer, then the calculated transferred energy would be much higher than the measured one and calculated blow counts would be non-conservatively low. It is therefore recommended that hammer problems are corrected as soon as possible on the construction site. If this is not possible then several diesel stroke or pressure options should be tried when matching analysis with field observation and the most conservative results should be selected. Section 17.7.1.1 discusses the available diesel hammer stroke options in greater detail.

GRLWEAP's hammer data file contains reduced combustion pressures for those hammers which have stepwise adjustable fuel pumps. Note that decreasing combustion pressures may be associated with program input fuel pump settings that have increasing numbers. For example, Delmag hammers' fuel pump settings 4 (maximum), 3, 2, and 1 (minimum) correspond to GRLWEAP hammer setting inputs 1 (or 0) 2, 3, and 4.

17.6.4 Cushion Input

Cushions are subjected to destructive stresses during their service and therefore continuously change properties. Pile cushions experience a particularly pronounced increase in their stiffness because they are generally made of soft wood with its grain perpendicular to the load. Typically, the effectiveness of wood cushions in transferring energy increases until they start to burn. Then they quickly deteriorate; this happens after approximately 1500 blows. To be conservative, the harder cushion (increased elastic modulus, reduced thickness) should be used for driving stress evaluations and the less effective cushion (lower stiffness, lower coefficient of restitution) should be analyzed for pile capacity calculations. If accurate values are not known, parameter changes of 25% from nominal might be tried. Wood chips as a hammer cushion are totally unpredictable and therefore should not be allowed. This is particularly true when the wave equation is used for construction control.

In recent years, uncushioned hammers have been used with increasing frequency. For the wave equation analysis, since there is no cushion spring, the stiffness of the spring between hammer and helmet is derived from either ram or impact block (diesels). This stiffness is very high, much higher than the stiffnesses of most other components within the system, and for numerical reasons, may lead to inaccurate stress predictions. Analyses with different numbers of pile segments would show the sensitivity of the numerical solution. In general, the greater the number of pile segments, the more accurate the stress calculation.

17.6.5 Soil Parameter Selection

The greatest errors in ultimate capacity predictions are usually observed when the soil resistance has been improperly considered. A very common error is the confusion of design loads with the wave equation's ultimate capacity. Note that the wave equation capacity always must be divided by a factor of safety to yield the allowable design load. Factors of safety suggested by FHWA and AASHTO were discussed in Chapter 15.

Since the soil is disturbed at the end of driving, it often has a lower capacity at that time (occasionally also a higher one) than at a later time. For this reason, a restrike test should be conducted to assess the ultimate pile capacity after time dependent soil strength changes have occurred. However, restrike testing is not always easy. The hammer is often not warmed up and only slowly starts to deliver the expected energy while at the same time the bearing capacity of the soil deteriorates. Depending on the sensitivity of the soil, the driving resistance may be taken from the first 75 mm of pile penetration even though this may be conservative for some sensitive soils. For construction control, rather than restrike testing many piles, it is more reasonable to develop a site specific setup factor in a preconstruction test program. As long as the hammer is powerful enough to move the pile during restrike and mobilize the soil resistance, restrike tests with dynamic measurements are an excellent tool to calculate setup factors. For the production pile installation criterion, the required end of driving capacity is then the required ultimate capacity divided by the setup factor. Using the wave equation analysis and the reduced end of driving capacity, the required end of driving blow count is then calculated.

Although the proper consideration of static resistance at the time of driving or restriking is of major importance for accurate results, dynamic soil resistance parameters sometimes play an equally important role. Damping factors have been observed to vary with waiting times after driving. Thus, damping factors higher than recommended in the GRLWEAP Manual (say twice as high) may have to be chosen for analyses modeling restrike situations. Studies on this subject are still continuing. In any event, damping factors are not a constant for a given soil type. For soft soils, they may be much higher than recommended and on hard rock they may be much lower. Choosing a low damping factor may produce non-conservative capacity predictions.

Shaft quakes are usually satisfactory as recommended at 2.5 mm. However, larger toe quakes than the typically recommended pile diameter divided by 120 may have to be chosen, particularly when the soil is rather sensitive to dynamic effects. Only dynamic measurements can reveal a more accurate magnitude of soil quakes. However, short of such measurements, conservative assumptions sometimes have to be made to protect against unforeseen problems. Fortunately, toe quakes have a relatively insignificant effect on the wave equation results of piles having most of their resistance acting along the shaft. For end bearing piles however, particularly displacement piles, large toe quakes often develop during driving in saturated soils causing the toe resistance to build up only very slowly during the hammer blow. Thus, at the first instant of stress wave arrival at the pile toe, little resistance exists and tension stresses can develop. In the case of concrete piles, the tension stresses can produce pile damage. At the same time, large toe quakes dissipate an unusually large amount of energy and therefore cause high blow counts. Thus, more cushioning or lower hammer strokes may not be a possible alternative for stress reductions. Instead, in extreme cases, hammers with heavier rams and lower strokes had to be chosen to reduce the detrimental effects of large toe quakes (see also Example 6 in Section 17.5.6).

Stress predictions, particularly tension stresses, are also sensitive to the input of the <u>resistance distribution</u> and to the percentage of toe resistance. If the soil resistance distribution is based on a static analysis, then chances are that the shaft resistance is set too high because of the loss of shaft resistance during driving. It is therefore recommended that driveability analyses be performed with shaft resistances reduced by estimated setup factors which will adjust the statically calculated capacity to the conditions occurring during driving.

<u>Residual stress wave equation analyses</u> are superior to normal analyses in basic concept and probably also in results. Unfortunately, not enough correlation work has been performed to empirically determine dynamic soil constants (quakes and dampings) that should be used with residual stress analyses. Another reason for its slow acceptance is the slower analysis performance. However, for long slender piles with significant shaft resistance components, residual stress analyses should be performed (maybe in addition to standard analyses) to assess potentially damaging stress conditions and the possibility of ultimate capacities which could be much higher than indicated by the standard wave equation analysis. Note that residual stress analyses may not be meaningful to represent early restrike situations where energies increase from blow to blow while, in sensitive soils, capacities successively decrease. The residual stress analysis assumes that hammer energy and pile capacity are constant under several hammer blows.

17.6.6 Comparison With Dynamic Measurements

Often the first impression is that wave equation predicted stresses and capacity values agree quite well with results from field dynamic measurements described in Chapter 18. However, there are additional observations and measurements that should be compared, such as stroke, bounce chamber pressure, and transferred energy. Often transferred energy values are somewhat lower than calculated, and adjustment of hammer efficiency alone may improve energy agreement but produce problems with driving stress and capacity agreement. Thus instead of adjusting hammer efficiency, the coefficients of restitution may have to be lowered. Sometimes matching of measured values can be very frustrating and difficult, and the task should be done with reason. Matching stresses and transferred energies within 10% of the observed or measured quantities may be accurate enough. **The wave equation maximum stresses in the final summary table can be anywhere along the length of the pile and may therefore not occur at the same location where the field measured maxima occur. When comparing GRLWEAP and field measurement results, it is therefore important to check the driving stresses in the extreme tables for the pile segment that corresponds to the measurement location.**

In summary, the following procedure is suggested for matching wave equation predictions with field measurements:

a. All adjustments are done until the quantities to be matched agree within 10%. It is to be realized that CAPWAP and GRLWEAP work with different models and input quantities and therefore cannot agree perfectly.

b. Perform wave equation modeling as accurately as possible for the system which measurements were taken. Use observed stroke, CAPWAP bearing capacity and associated soil parameters, and cushion properties as per standard recommended values.

c. For matching of transferred energy, vary hammer efficiency by increasing it to at most 0.95 and decreasing it to no less than 50% of the standard recommended hammer efficiency for that hammer type. If efficiency changes are insufficient to produce agreement between wave equation calculation and field measurement results to within 10%, adjust cushion coefficients of restitution. The cushion coefficients of restitution should not be increased to values above 0.98 nor decreased to values less than 50% of the standard recommended coefficient of restitution for that cushion material.

d. For matching the measured force, adjust cushion stiffness (pile cushion if present otherwise hammer cushion). This process may then require readjusting hammer efficiency and coefficient of restitution for energy match as per step c. Additional iterations through steps c and d should be made until transferred energy and force are within 10%.

e. Compare blow counts. Change the shaft and toe damping and the toe quake simultaneously and proportionately to achieve agreement between measured and computed blow counts.

17.7 WAVE EQUATION INPUT PARAMETERS

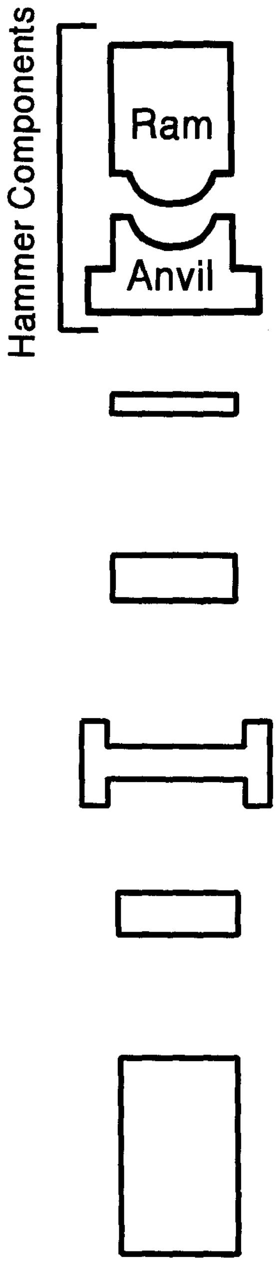
As described in the previous sections, the input for a wave equation analysis consists of information about the soil, pile, hammer, cushions, helmet, splices, and any other devices which participate in the transfer of energy from hammer to soil. This input information is usually gathered from contract plans, the contractor's completed Pile and Driving Equipment Data Form (Figure 17.24), soil boring, and a static pile capacity analysis. Helpful information can also be found in the tables of the GRLWEAP Users Manual (1996) which, at least in part, is included in the "Help" display of GRLWEAP's input section. These tables are correct only for ideal situations, but may yield valuable data before a specific driving system has been identified. In general, contractors tend to assemble equipment from a variety of sources, not all of them of a standard type. It is therefore important to check and confirm what equipment the contractor has actually included in the driving system on the job.

The following sections explain the most important input quantities for the data input process in the GRLWEAP program. For a more detailed explanation of input quantities, reference is made to the program's Users Manual.

For a simple bearing graph analysis, only three of the fourteen possible GRLWEAP data input pages must be used. These three pages are shown in Figures 17.25 through Figure 17.27. A cursor can be moved from one variable to another. For the variable activated by the cursor, a short help is given at the bottom of the input page. Additional help is available if an "H" is displayed at the lower right hand corner (see Input Page 1 and Input Page 2 in Figures 17.25 and 17.26). The following input descriptions summarize what type of input is requested without describing in detail the full implications of choosing certain options.

Contract No.: ____________________ Structure Name and/or No.: ____________________
Project: ____________________

Pile Driving Contractor or Subcontractor: ____________________
County: ____________________
(Piles driven by)



Hammer

Manufacturer: ____________ Model No.: ____________
Hammer Type: ____________ Serial No.: ____________
Manufacturers Maximum Rated Energy: ____________ (Joules)
Stroke at Maximum Rated Energy: ____________ (meters)
Range in Operating Energy: ________ to ________ (Joules)
Range in Operating Stroke: ________ to ________ (meters)
Ram Weight: ____________ (kg)
Modifications: ____________________

Striker Plate

Weight: ____________ (N) Diameter: ____________ (mm)
Thickness: ____________ (mm)

Hammer Cushion

Material #1
Name: ____________
Area: ____________ (cm^2)
Thickness/Plate: ____________ (mm)
No. of Plates: ____________

Material #2
(for Composite Cushion)
Name: ____________
Area: ____________ (cm^2)
Thickness/Plate: ____________ (mm)
No. of Plates: ____________

Total Thickness of Hammer Cushion: ____________

Helmet (Drive Head)

Weight: ____________ (kN)

Pile Cushion

Material: ____________________
Area: ____________ (cm^2) Thickness/Sheet: ____________ (mm)
No. of Sheets: ____________
Total Thickness of Pile Cushion: ____________ (mm)

Pile

Pile Type: ____________________
Wall Thickness: ____________ (mm) Taper: ____________
Cross Sectional Area: ____________ (cm^2) Weight/Meter: ____________

Ordered Length: ____________ (m)
Design Load: ____________ (kN)
Ultimate Pile Capacity: ____________ (kN)

Description of Splice: ____________________

Driving Shoe/Closure Plate Description: ____________________

Submitted By: ____________ Date: ____________
Telephone No.: ____________ Fax No.: ____________

Figure 17.24 Pile and Driving Equipment Data Form

17.7.1 GRLWEAP Input - Page 1

The first page requires title input (important for problem identification), important hammer options, pile and soil resistance input, and hammer cushion and helmet information.

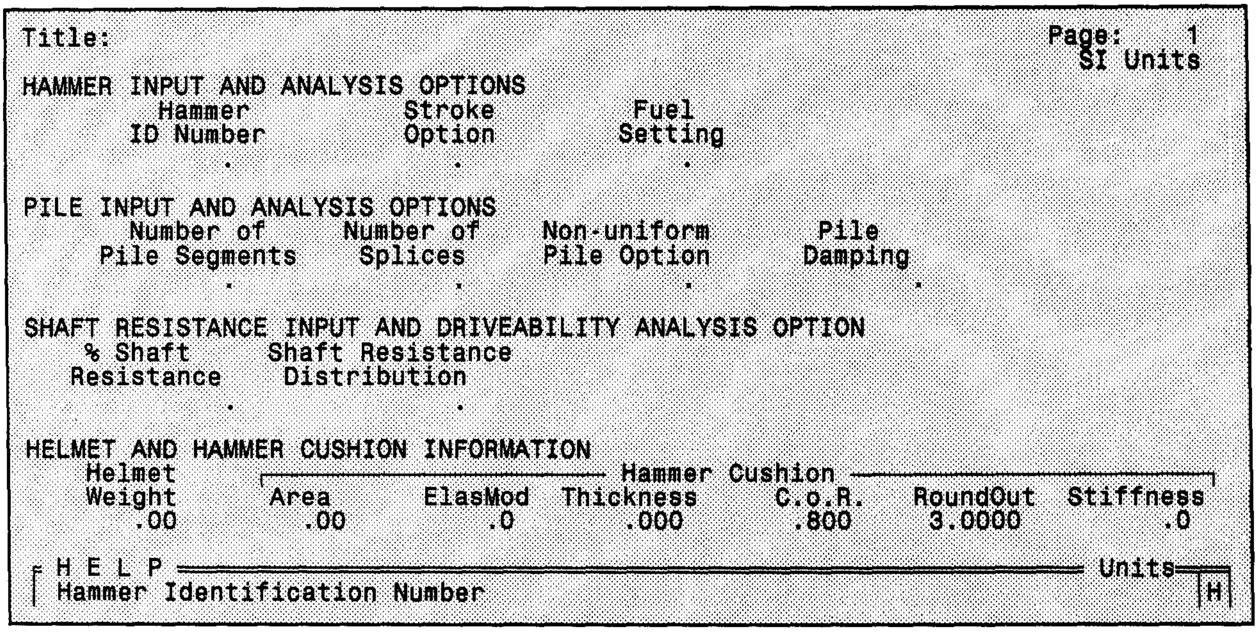


Figure 17.25 Input Page 1: Title, Options, Hammer Cushion

17.7.1.1 Hammer Input and Analysis Options

Hammer ID Number - GRLWEAP contains a hammer data file with approximately 500 different hammer models. The user only has to pick a number from the hammer listing given in the Help section of the program. Note that the hammer data in the file assumes that the hammer has been well maintained and not been significantly modified.

Stroke Option - For any diesel hammer, the stroke is a function of pile size and soil resistance. The stroke option lets the user decide whether a fixed stroke (option 1 or -1) is to be analyzed or whether the program should calculate the diesel hammer stroke (option 0). Also, an entry 2, or -2 will produce the so-called inspectors chart for a fixed ultimate capacity and an automatically varied stroke.

The positive stroke options 1 and 2 simply analyze one hammer blow and ignore whether or not the calculated rebound stroke matches the analyzed down stroke. The negative options -1 and -2 will repeat the analyses with adjusted combustion pressures until the upward stroke matches the analyzed downward stroke. Thus, if a high stroke (relative to the soil resistance) has to be analyzed then, at first, the calculated upward stroke will be small compared to the down stroke. Increasing the combustion pressure in the analysis provides the hammer model the energy necessary to maintain a high stroke in the presence of a low soil resistance. As pointed out in Section 17.6.3, analyzing a hammer with a high combustion pressure, even though the high stroke was the result of preignition, may lead to high calculated transferred energies and therefore non-conservative capacity predictions. On the other hand, if the observed hammer stroke is relatively low and if friction (which should be modeled with a lower hammer efficiency) has been eliminated as a reason for the low stroke, then a reduced combustion pressure is a very reasonable analysis option.

Fuel Setting - A number of diesel hammers have stepwise adjustable fuel pumps which allow for the injection of measured, variable amounts of fuel into the combustion chamber. This option allows the user the choice of such a hammer setting. For hammers with continuously variable fuel pumps, the program's reduced fuel settings do not correspond to a specific fuel setting that can be selected in the field but rather an arbitrary value between the hammer's maximum and minimum available settings.

17.7.1.2 Pile Input and Analysis Options

Number of Pile Segments - Usually the number of pile segments is left to the program to calculate. The user may choose a larger or smaller value to make the analysis more accurate or faster, respectively.

Number of Splices - Some piles are spliced with devices that allow for some slippage during extension or compression. A welded splice does not allow for slippage and therefore is modeled like a uniform pile section and not counted as splice.

Non-uniform Pile Option - Simply enter 0, 1, or 2 for specifying a uniform pile, a non-uniform pile, or two parallel piles (for example, a mandrel driven pile), respectively.

Pile Damping Option - Depending on the pile material, this option is entered as 1, 3, or 5 for steel, concrete or timber, respectively, and corresponds to a percentage of pile structural damping.

17.7.1.3 Shaft Resistance Input and Driveability Analysis Options

% Shaft Resistance - This very important option has two functions. If set to zero it generates a driveability analysis. Otherwise it causes a bearing graph, or inspector's chart, to be produced and represents the percentage of ultimate resistance acting along the pile shaft.

Shaft Resistance Distribution - After the percentage of shaft resistance and, therefore, the total shaft resistance has been determined for an ultimate capacity value, this shaft resistance must be distributed along the embedded portion of the pile. Often a triangular or a rectangular distribution is sufficiently accurate and can be selected here with a simple number input. Alternatively, the user may input a more complex distribution on another input page.

17.7.1.4 Helmet and Hammer Cushion Information

As pointed out earlier, the following information may either be retrieved from the User's Manual, the program's help section or the contractor's completed Pile and Driving Equipment Data Form (Figure 17.24). Since contractors seldom use standard equipment on construction sites, the latter is the preferable data source.

Helmet Weight should be the combined weight of the helmet, hammer cushion, striker plate, inserts, and all other components located between hammer and pile (kN).

Area of the hammer cushion perpendicular to the load (cm^2).

ElasMod is the elastic modulus of the hammer cushion material (MPa).

Thickness of the hammer cushion. For sandwiched cushions, this is the thickness of the entire cushion stack and the striker plate is not included (mm).

C.O.R. is the Coefficient of Restitution of the hammer cushion material.

RoundOut (compressive slack) deformation of the hammer cushion (mm). This quantity is a small distance of cushion compression over which the stiffness of the hammer cushion is thought to increase from 0 to its nominal value. Usually the user leaves this quantity at the preprogrammed default value.

Stiffness of the hammer cushion (kN/m). Use of this input will override previous inputs for area, elastic modulus and thickness.

17.7.2 GRLWEAP Input - Page 2

```
Title:                                                              Page:    2
                                                                      SI Units
PILE CUSHION INFORMATION
                    Elastic                        Coeff.of       Round
      Area          Modulus     Thickness        Restitution       Out      Stiffness
       .00               .0          .000               .500      3.0000           .0

PILE INFORMATION
  Total     X-Sectn     Elastic     Specific    Circum-   Strength/  Coeff.of    Round
  Length     Area       Modulus     Weight      ference     Yield    Restitutn    Out
     .00        .00   210000.0      78.500        .000       .000      .8500    3.0000

HAMMER OVERRIDE VALUES
                                   Reaction   ComDelay   Comb Exp    Stroke
   Stroke      Effcy    Pressure    Weight    Ign Vol     Coeff    Conv Crit    Unused
     .00        .000          .0      .000       .000       .00        .00        .00

SOIL PARAMETERS
     Quake                 Damping                       Toe No. 2
   Skin       Toe       Skin       Toe       Quake    Damping   Fraction     Depth
  2.500      2.500      .000       .000      .000       .000       .000        .00

 H E L P                                                                   Units
 Area of the Pile Cushion                                                   cm²  |H|
```

Figure 17.26 Input Page 2: Pile Cushion, Pile, Hammer Modifications, Soil

17.7.2.1 Pile Cushion Information

When a pile cushion is used, usually for concrete piles, input is required for the pile cushion area, elastic modulus, thickness, coefficient of restitution, round out, and stiffness, as previously described for the hammer cushion.

17.7.2.2 Pile Information

Total Length is the total pile length in the leads (m). For example if plans require a 15 m long pile but the contractor is driving 18 m long piles, then the analysis length should be 18 m. If pile sections are spliced together to form a long pile then an analysis before and after splicing may be of interest. In that case, "Total Length" may be the length of the short first section before splicing or the combined length after splicing.

X-Sectn Area is the pile cross sectional area at the pile head (cm^2).

Elastic Modulus is the elastic modulus of the pile material at the pile head (MPa).

Specific Weight is the weight per unit volume of the pile material at the pile head (kN/m^3).

Circumference of pile head for the calculation of capacity from unit shaft resistance for driveability analyses (m).

Strength/Yield used for piles with more than one material for an evaluation of the critical but not necessarily the maximum stresses (MPa). For example, in a concrete pile with a steel H-pile tip, the program uses the strength/yield information to include the critical concrete stresses rather than the much higher but possibly non-critical steel stresses in the final table.

Coeff. of Restitutn of the pile head - helmet interface. The manual provides experience values.

Round Out (compressive slack) deformation of the pile head - helmet interface (mm). Again the program provides experience values for standard cases.

For piles with non-uniform cross sections, additional information would also be needed.

17.7.2.3 Hammer Override Values

Hammer overrides allows one or more values of the hammer data file to be changed for a particular analysis. The most commonly used overrides are discussed below.

Stroke (m) and **Effcy** (efficiency) are probably the most important hammer override values. Under certain circumstances the user may want to analyze a stroke which is different from the rated stroke or from the automatically calculated one. For hammers with read-out of energy (this is sometimes available in modern hydraulic hammers), stroke should be entered as a proportionally reduced value when energy is not at the maximum rated value. The user should also seriously consider whether or not the data file efficiency (which is the same for all hammer makes of the same type) should be adjusted to reflect actual field conditions.

Pressure (kPa) is important for diesel hammers when calculated and observed hammer stroke differ. A new pressure value may then be tried for better agreement.

Reaction Weight, ComDel Ign Vol, Delay, Comb Exp. Coeff, and Stroke Conv Crit are quantities associated with specific diesel hammer functions and are not routinely input unless an unusual hammer performance has to be modeled (*e.g.*, the combustion delay - ComDel Ign Vol - is the quantity that allows for modeling of preignition in diesel hammers with liquid fuel injection).

17.7.2.4 Soil Parameters

Both the Users Manual and the program's Help section provide tables with very basic suggestions for the dynamic soil resistance parameters.

Quake Skin is the soil quake along the pile shaft usually chosen as 2.5 mm.

Quake Toe is the soil quake at the pile toe. Often chosen as b/120 (mm) where b is the effective pile diameter or width of the pile toe.

Damping Skin is the soil damping constant along the shaft. A Smith shaft damping constant of 0.65 s/m is commonly chosen for cohesive soils and 0.16 s/m for non-cohesive soils. Although several damping models are available in most wave equation programs, the Smith approach is generally preferred.

Damping Toe is the soil damping constant at the toe. Most commonly, Smith toe damping constants of 0.50 s/m are chosen regardless of soil type.

Toe No. 2 input quantities are reserved for those situations when piles have more than one pile toe interface, as for an H-pile section cast into a concrete pile. Then end bearing acts both against the concrete bottom and the steel toe. Toe No. 2 would be the concrete bottom in this example.

17.7.3 GRLWEAP Input - Page 3

17.7.3.1 Ultimate Capacities

For bearing graphs, up to 10 ultimate resistance values (kN) may be analyzed in one analysis. For the inspector's graph, only one capacity value will be analyzed with varying strokes.

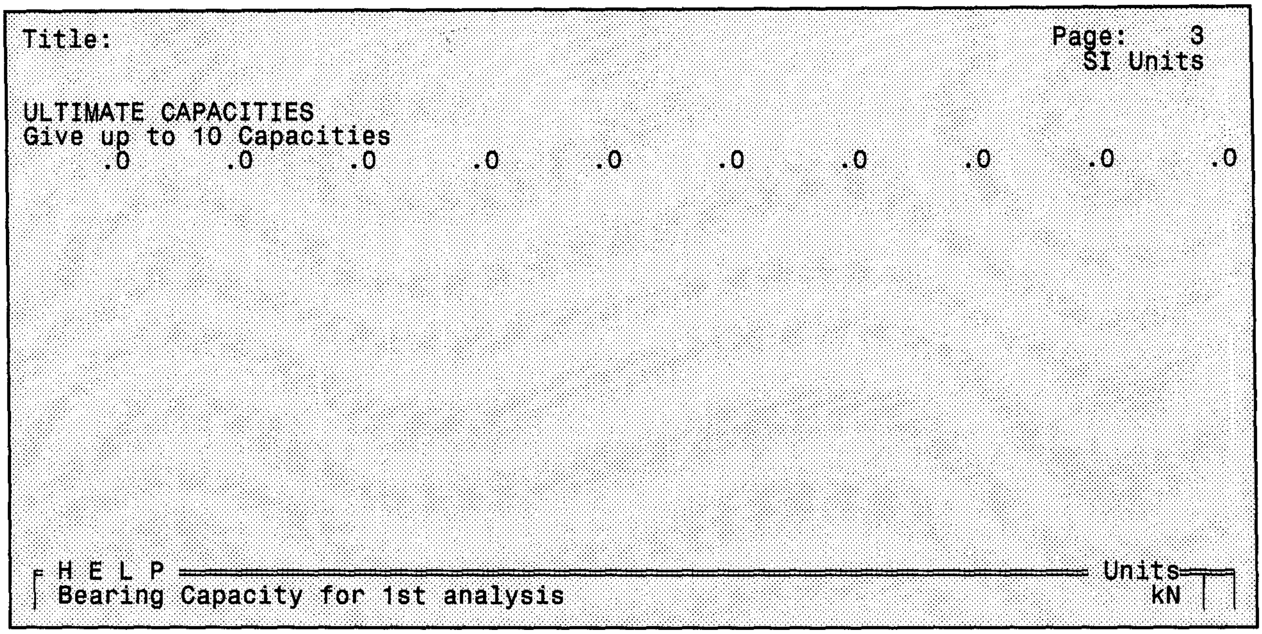


Figure 17.27 Input Page 3: Ultimate Capacities

For a simple bearing graph, these three input pages complete the required program input. When more complex problems are analyzed, additional input pages are required. For a discussion of more complex problems, the interested reader should consult the GRLWEAP Manual (1996).

17.8 GRLWEAP OUTPUT

The printed GRLWEAP output begins with a listing of file names used for input and a listing of the input file. There follows a disclaimer statement which points out some of the uncertainties associated with wave equation analyses. The user is urged to check that the correct data file was used and consider the disclaimer when drawing conclusions from analysis results.

The first page of output, shown in Figure 17.28, lists the hammer and drive system components used in the analysis. Hence hammer model, hammer stroke and efficiency, helmet weight, as well as hammer and pile cushion properties including thickness, area, elastic modulus and coefficient of restitution, are but a few of the input details printed on this page of output.

```
                  Hammer Model of: D 12      Made by: DELMAG

         No.      Weight     Stiffn         CoR      C-Slk      Dampg
                    kN        kN/mm                    mm      kN/m/s
           1       4.079
           2       4.079    15989.4       1.000     3.0480
           3       4.079    15989.4       1.000     3.0480
   Imp Block       3.604     9924.2        .900     3.0400
      Helmet       9.560     6950.0        .800     3.0480        81.3

HAMMER OPTIONS:
Hammer File ID No.                  3  Hammer Type                        1
Stroke Option                       0  Stroke Convergence Crit.        .020
Fuel Pump Setting                   1  Hammer Damping                     2

HAMMER DATA:
Ram Weight            (kN)      12.24  Ram Length            (mm)   2652.01
Maximum Stroke         (m)       2.62  Actual Stroke          (m)      1.63
                                       Efficiency                      .800

MaximumPressure      (kPa)    9711.70  Actual Pressure      (kPa)   9711.70
Compression Exponent            1.350  Expansion Exponent             1.250
Ram Diameter          (cm)     299.97  Minimum Stroke         (m)      1.63

Combustion Delay       (s)     .00200  Ignition Duration      (s)    .00200

        The Hammer Data Includes Estimated (NON-MEASURED) Quantities
        ------------------------------------------------------------

HAMMER CUSHION                         PILE CUSHION
Cross Sect. Area      (cm2)   1829.03  Cross Sect. Area      (cm2)      .00
Elastic-Modulus       (MPa)    1930.3  Elastic-Modulus       (MPa)       .0
Thickness              (mm)     50.80  Thickness              (mm)      .00
Stiffness           (kN/mm)    6950.0  Stiffness           (kN/mm)       .0
```

Figure 17.28 Hammer Model, Driving System and Hammer Option Output

The second page of output, presented in Figure 17.29, summarizes the pile and soil model used in the analysis. A brief summary of the pile profile is provided at the top of the page, and includes the pile length, area, modulus of elasticity, specific weight, circumference, material strength, wave speed, and pile impedance. A detailed summary of the pile and soil model follows beneath the pile profile. The detailed pile model includes the number of pile segments, their weight and stiffness, any compression (C-Slk) or tension (T-Slk) slacks with associated coefficient of restitution (CoR). The listing also shows segment bottom depth (LbTop), and the averages of both segment circumference and cross sectional area.

```
 PILE PROFILE:
  L b Top      Area     E-Mod  Spec Wt  Circumf  Strength  Wave Sp    EA/c
        m       cm2       MPa    kN/m3        m       MPa      m/s  kN/m/s
      .00     100.0  209820.     78.80    1.000      1.00    5111.  410.54
    12.20     100.0  209820.     78.80    1.000      1.00    5111.  410.54

               Wave Travel Time - 2L/c - =   4.774 ms


              Pile and Soil Model for Rut =     250.0 kN
No. Weight  Stiffn C-Slk T-Slk  CoR  Soil-S  Soil-D Quake  LbTop Circmf   Area
        kN   kN/mm    mm    mm           kN     s/m    mm      m      m    cm2
  1  1.202   1376. 3.048  .000  .80      .4    .164  2.54   1.52    1.0  100.0
  2  1.202   1376.  .000  .000 1.00     1.2    .164  2.54   3.05    1.0  100.0
  3  1.202   1376.  .000  .000 1.00     2.0    .164  2.54   4.57    1.0  100.0
  4  1.202   1376.  .000  .000 1.00     2.7    .164  2.54   6.10    1.0  100.0
  5  1.202   1376.  .000  .000 1.00     3.5    .164  2.54   7.63    1.0  100.0
  6  1.202   1376.  .000  .000 1.00     4.3    .164  2.54   9.15    1.0  100.0
  7  1.202   1376.  .000  .000 1.00     5.1    .164  2.54  10.67    1.0  100.0
  8  1.202   1376.  .000  .000 1.00     5.9    .164  2.54  12.20    1.0  100.0
Toe                                   225.0    .492  2.54

 PILE, SOIL, ANALYSIS OPTIONS:
 Uniform/Non-Uniform/2-Pile          0  Pile Segment Generation    Automatic
 No. of Slacks/Splices               0  Pile Damping            (%)        1
                                        Pile Damping Fact.(kN/m/s)     8.211
 % Skin Friction                    10  % End Bearing                     90
 Soil Damping Option             Smith  Soil Damping Exponent          1.000
 Soil Resistance Distr. No.          1

 Max No Analysis Iterations          0  Time Increment/Critical          160
 Residual Stress Analysis            0  Output Option                     10
 Output Segment Generation   Automatic  Output Time Interval               1
 Analysis Time-Input   (ms)          0
```

Figure 17.29 Pile, Soil Model and Analysis Options

The soil model summarized includes the soil static soil resistance distribution (Soil-S), the soil damping parameters (Soil-D) along the shaft and at the pile toe as well as the soil quakes along the shaft and at the pile toe. Additional pile and soil modeling options, including the percent shaft and toe resistance, are summarized below the detailed model.

Beginning on the third page of output, as shown in Figure 17.30, an extrema table is printed for each pile segment number. This extrema output is printed for each analyzed ultimate capacity and includes:

min F and max F	minimum and maximum pile forces (kN).
min Str and max Str	minimum and maximum pile driving stresses (MPa).
max V, max D, and max Et	maximum velocity (m/s), displacement (mm) and transfer energy (kJ), respectively.

```
            Rut=   250.0, Rtoe =    225.0    (kN), Time Inc. = .101 ms
 No.  min F, t   max F, t  min Str, t max Str, t   max V, t  max D, t   max Et
      (kN)       (kN)       (MPa)      (MPa)       (m/s)      (mm)        (kJ)
  1     0.,  0  1107.,  2     .0,  0  110.7,  2     2.5,  2  34.5, 28   14.48
  2    -6., 50  1118.,  2    -.6, 50  111.8,  2     2.5,  7  34.4, 28   14.45
  3   -10., 50  1125.,  3   -1.0, 50  112.5,  3     2.6,  6  34.3, 28   14.38
  4   -11., 50  1130.,  3   -1.1, 50  113.0,  3     2.8,  6  34.2, 28   14.28
  5   -11., 46  1132.,  3   -1.1, 46  113.2,  3     2.9,  6  34.1, 28   14.14
  6   -12., 46  1124.,  4   -1.2, 46  112.4,  4     2.9,  5  33.9, 28   13.98
  7   -11., 46  1060.,  4   -1.1, 46  106.0,  4     3.0,  5  33.8, 28   13.77
  8    -7., 46   874.,  4    -.7, 46   87.4,  4     3.6,  5  33.6, 28   13.65

  (Eq) Strokes Analyzed and Last Return (m):
    1.63  1.33  1.41  1.39
```

Figure 17.30 Extrema Table Output

The "t" values following the extreme values are times in milliseconds relative to hammer impact. For the analysis of diesel hammers, the iteration on hammer stroke is indicated beneath the extrema table information.

For bearing graph analyses, GRLWEAP prints a summary table for all input ultimate capacities after the extrema table listing for the last ultimate capacity analysis. The summary table is illustrated in Figure 17.31, and includes the ultimate capacity, R_{ut}, and the corresponding driving resistance, hammer stroke, tension and compression stresses, the maximum transferred energy, ENTHRU, and the hammer operating speed, Bl Rt, for diesels only. The indicators "i,t" are the pile segment number at the location and at the time when the extreme stress values occur, respectively.

A review of the "printed output" can be accomplished on the computer screen before printing. This review is extremely important as it can point out inadvertent omissions or erroneous input data. The reviewer should carefully check ram weight, stroke, efficiency, cushion stiffness, pile masses, stiffnesses, soil parameters, *etc.* Furthermore, any error messages or warnings issued by the program should be checked for relevance to the results.

Rut (kN)	Bl Ct (bpm)	Stroke down	(m) up	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)	Bl Rt (b/min)
250.0	32.2	1.41	1.39	-1.18(	6, 46)	113.18(	5, 3)	14.5	55.3
500.0	76.6	1.63	1.64	-3.18(	5, 32)	131.10(	4, 3)	12.9	51.0
750.0	124.5	1.81	1.80	-5.68(	5, 26)	148.15(	8, 4)	12.8	48.6
1250.0	272.2	2.12	2.11	-16.46(	5, 16)	200.98(	8, 4)	13.8	44.9
1500.0	422.9	2.28	2.27	-24.81(	5, 16)	221.10(	8, 4)	14.5	43.4
1750.0	763.6	2.42	2.42	-29.52(	5, 15)	237.24(	8, 4)	15.1	42.1
2000.0	1829.8	2.55	2.56	-33.53(	5, 15)	251.59(	1, 7)	15.7	41.0

Figure 17.31 GRLWEAP Final Summary for Bearing Graph Analyses

17.9 PLOTTING OF GRLWEAP RESULTS

The summary table results are usually presented in the form of a bearing graph relating the ultimate capacity to driving resistance. Compression and tension stresses versus driving resistance are also plotted. A typical GRLWEAP bearing graph was presented in Figure 17.4 as part of Example 1.

The wave equation bearing graph should be provided to the resident construction engineer, pile inspector, and the contractor.

TABLE 17-1 SUGGESTED USE OF THE WAVE EQUATION TO SOLVE FIELD PROBLEMS (CONTINUED)	
Problem	Solution
Diesel hammer stroke (bounce chamber pressure) higher than calculated.	The field observed stroke exceeds the wave equation calculated stroke by more than 10%. Compare calculated and observed blow counts. If observed are higher, soil resistance is probably higher than anticipated. If blow counts are comparable, reanalyze with higher combustion pressure to match observed stroke <u>and</u> assure that preignition is not a problem, *e.g.*, by measurements.
Diesel hammer stroke (bounce chamber pressure) lower than calculated.	The field observed stroke is less than 90% of the stroke calculated by the wave equation. Check that ram friction is not a problem (ram surface should have well lubricated appearance). Compare calculated and observed blow count. If observed one is lower, soil resistance is probably lower than anticipated. If blow counts are comparable, reanalyze with lower combustion pressure to match observed hammer stroke.

TABLE 17-2 WAVE EQUATION ANALYSIS PROBLEMS	
Problem	Solution
Cannot find hammer in data file.	See if there is a hammer of same type, similar ram weight and energy rating and modify its data.
Cannot find an acceptable hammer to drive pile within driving stress and driving resistance limits.	Both calculated stresses and blow counts are too high. Increase pile impedance or material strength or redesign for lower capacities. Alternatively, check whether soil has potential for setup. If soil is fine grained or known to exhibit setup gains after driving then end of driving capacity may be chosen lower than required. Capacity should be confirmed by restrike testing or static load testing.
Diesel hammer analysis with low or zero transferred energies.	Probably soil resistance too low for hammer to run. Try higher capacities.
Unknown hammer energy setting.	Perform analyses until cushion thickness/hammer energy setting combination is found that yields acceptable stresses with minimum cushion thickness. Specify that this thickness be used in the field and its effectiveness verified by measurements.
Cannot find a suggested set of driving system data.	Contact contractor, equipment manufacturer, or use data for similar systems.
Unknown pile cushion thickness.	Perform analyses until cushion thickness/hammer energy setting combination is found that yields acceptable stresses with minimum cushion thickness. Specify that this thickness be used in the field and its effectiveness verified by measurements.
Calculated pile cushion thickness is uneconomical.	In order to limit stresses, an unusually thick pile cushion was needed for pile protection. Try to analyze with reduced energy settings. For tension stress problems, energy settings often can be increased after pile reaches sufficient soil resistance.

TABLE 17-2 WAVE EQUATION ANALYSIS PROBLEMS (CONTINUED)	
Problem	Solution
Calculated driving times unrealistically high or low.	The calculation of driving times is very sensitive, particularly at high blow counts. Use extreme caution when using these results for cost estimation. Also, no interruption times are included and the estimate is only applicable to non-refusal driving.
Wave equation calculated energy and/or forces difficult to match with field measurements.	In general, it is often difficult to make all measured quantities agree with their calculated equivalents. A 10% agreement should be sufficient. Parameters to be varied include hammer efficiency, coefficients of restitution, hammer and/or pile cushion stiffnesses.

REFERENCES

AASHTO, American Association of State Highway and Transportation Officials, (1992). Interim Specifications, Bridges, 1993 and 1994. Standard Specifications for Highway Bridges, ISBN 1-56051-014-5, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

Blendy, M.M. (1979). Rational Approach to Pile Foundations. Symposium on Deep Foundations, ASCE National Convention.

Cheney, R.S. and Chassie, R.G. (1993). Soils and Foundations Workshop Manual. Second Edition, Report No. HI-88-009, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering, Washington, D.C., 353-362.

Goble, G.G. and Rausche, F. (1976). Wave Equation Analysis of Pile Driving - WEAP Program, U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development, Washington, D.C., Volumes I-IV.

Goble, G.G. and Rausche, F. (1986). Wave Equation Analysis of Pile Driving - WEAP86 Program, U.S. Department of Transportation, Federal Highway Administration, Implementation Division, McLean, Volumes I-IV.

Goble, Rausche, Likins and Associates, Inc. (1996). GRLWEAP Users Manual. 4535 Emery Industrial Parkway, Cleveland, OH 44128.

Hirsch, T.J., Carr, L. and Lowery, L.L. (1976). Pile Driving Analysis. TTI Program, U.S. Department of Transportation, Federal Highway Administration, Offices of Research and Development, IP-76-13, Washington, D.C., Volumes I-IV.

Soares, M., de Mello, J. and de Matos, S. (1984). Pile Driveability Studies, Pile Driving Measurements. Proceedings of the Second International Conference on the Application of Stress-Wave Theory to Piles, Stockholm, 64-71.

Smith, E.A.L. (1960). Pile Driving Analysis by the Wave Equation. American Society of Civil Engineers, Journal of the Soil Mechanics and Foundations Division, 86(4), 35-61.

U.S. Army Corps of Engineers, Department of the Army (1993) Engineering Manual EM 1110-2-2906, reprinted by American Society of Civil Engineers in Design of Pile Foundations, 345 East 47th Street, New York, NY 10017-2398.

STUDENT EXERCISE #11 - WAVE EQUATION HAMMER APPROVAL

A contractor owns two hammers that he may use on a bridge construction project but is unsure which hammer will actually be available for the project. Therefore, he has submitted Pile Driving and Equipment Data forms for both hammers to the engineer for approval. The pile foundation design requires 25 meter long, 356 mm diameter, closed end pipe piles to be driven for an ultimate pile capacity of 2670 kN. The pipe piles have a wall thickness of 12.7 mm and are to comply with ASTM A-252, Grade 3 steel. Therefore, the piles have a minimum yield strength of 310 MPa.

The first driving system consists of a Vulcan 510 single acting air hammer with a manufacturer's rated hammer energy of 67.8 kJ. The Vulcan 510 hammer will have an aluminum and micarta hammer cushion. The second driving system consists of an IHC S-70 double acting hydraulic hammer which has a manufacturer's rated energy of 70.0 kJ. The contractor proposes to operate this hammer at an equivalent stroke of 1.9 meters or roughly 92% of the maximum energy. The IHC hammer does not utilize a hammer cushion. The results of the wave equation analyses for the two proposed driving systems are attached.

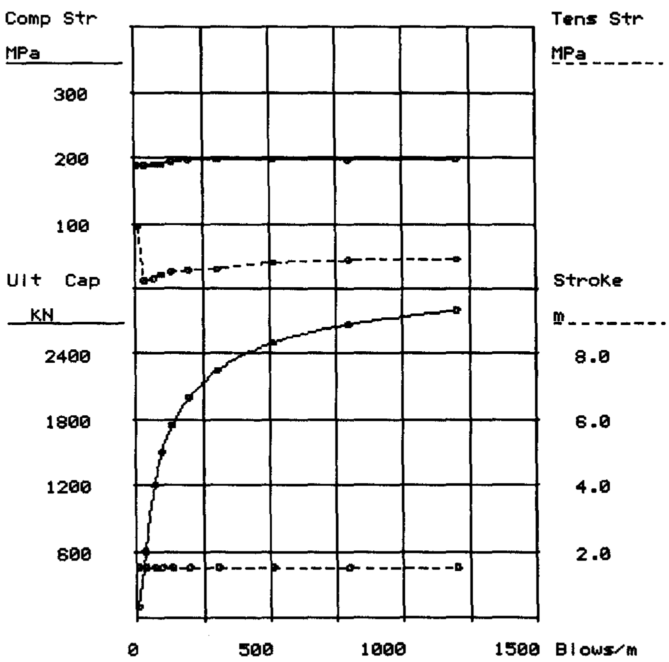
Based on the submitted hammer information and wave equation results, should both, or either of these hammers be approved?

Note: Recommended driving resistances for hammer approval are presented on Page 12-12.

Recommended driving stress limits for steel pipe piles are presented on Page 11-5.

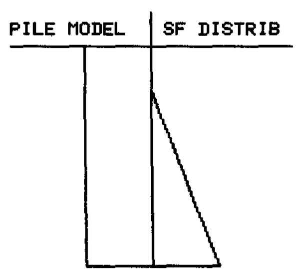
HAMMER APPROVAL - VUL 510 SUBMITTAL 95/09/22
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.994-1

Rut (kN)	Bl Ct (bpm)	Stroke(eq.) (m)	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)
100.0	7.5	1.52	-94.84	(5, 12)	188.83	(7, 4)	38.1
600.0	33.1	1.52	-12.54	(12, 47)	189.12	(8, 5)	41.9
1200.0	67.8	1.52	-15.71	(12, 33)	189.70	(9, 5)	41.5
1500.0	95.8	1.52	-21.81	(13, 31)	190.04	(8, 5)	40.6
1750.0	134.5	1.52	-25.95	(15, 45)	193.46	(25, 8)	39.9
2000.0	199.3	1.52	-28.52	(14, 30)	197.31	(25, 8)	39.7
2250.0	306.7	1.52	-30.44	(10, 41)	198.28	(25, 8)	39.7
2500.0	511.9	1.52	-40.46	(11, 40)	197.83	(25, 8)	39.6
2670.0	796.8	1.52	-44.19	(11, 40)	197.03	(25, 8)	39.6
2800.0	1202.6	1.52	-46.29	(11, 39)	198.47	(2, 13)	39.6



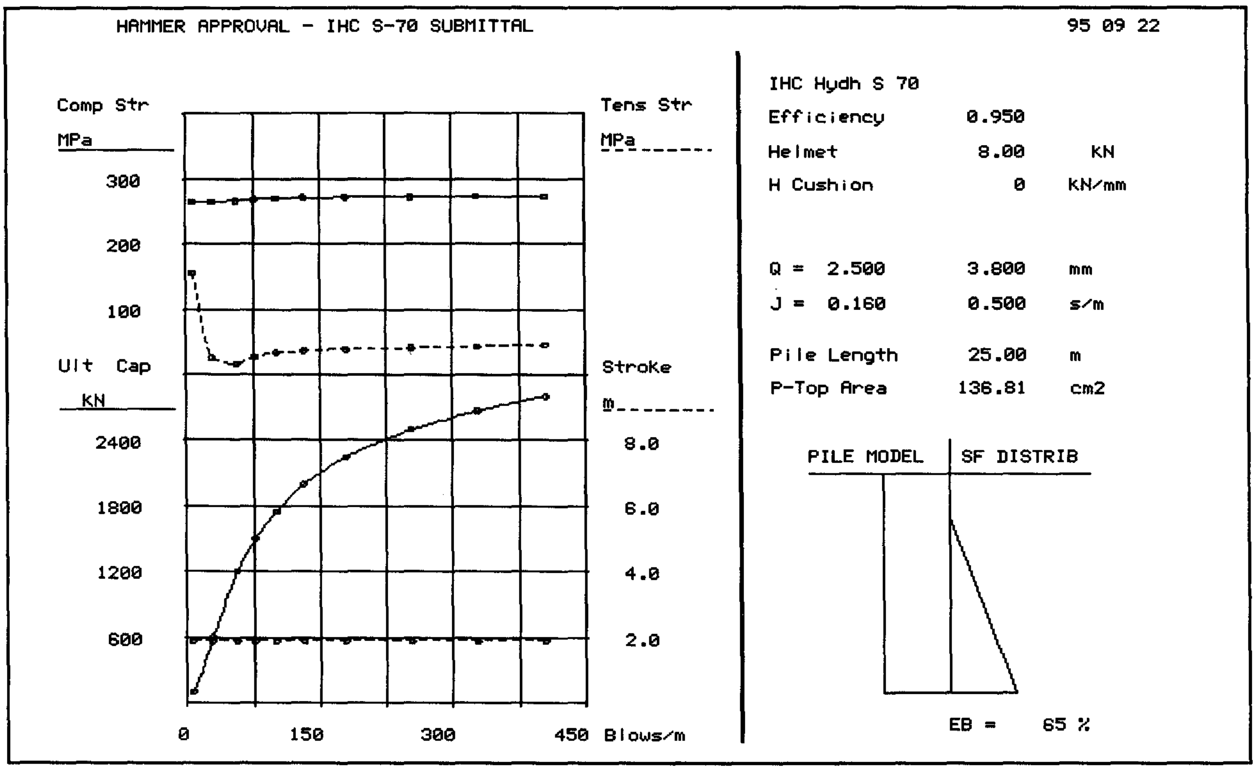
VULCAN VUL 510

Efficiency	0.670	
Helmet	6.14	kN
H Cushion	1316	kN/mm
Q = 2.500	3.800	mm
J = 0.160	0.500	s/m
Pile Length	25.00	m
P-Top Area	136.81	cm2



HAMMER APPROVAL - IHC S-70 SUBMITTAL 95/09/22
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.994-1

Rut (kN)	Bl Ct (bpm)	Stroke(eq.) (m)	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)
100.0	6.5	1.90	-155.26(	6, 11)	265.77(	3, 3)	51.1
600.0	29.2	1.90	-25.68(	6, 47)	265.77(	3, 3)	55.0
1200.0	56.7	1.90	-17.12(	13, 32)	267.25(	4, 3)	54.9
1500.0	77.1	1.90	-27.35(	13, 29)	269.03(	7, 3)	54.0
1750.0	100.8	1.90	-33.58(	13, 28)	270.15(	7, 3)	53.5
2000.0	132.1	1.90	-37.35(	14, 28)	271.23(	7, 3)	53.2
2250.0	179.7	1.90	-39.84(	14, 28)	271.79(	7, 3)	53.2
2500.0	253.7	1.90	-41.38(	14, 27)	272.41(	8, 4)	53.1
2670.0	328.1	1.90	-43.87(	14, 26)	273.02(	7, 3)	53.1
2800.0	405.1	1.90	-46.17(	14, 26)	273.23(	7, 3)	53.1



STUDENT EXERCISE #12 - WAVE EQUATION INSPECTORS CHART

A contractor has chosen a Kobe K-35 for foundation installation of HP 360 x 174 H-piles. The H-piles are to be driven to a limestone bedrock for an ultimate pile capacity of 3250 kN. The H-piles are to be A-36 steel.

For hammer approval, a standard wave equation bearing graph analysis was performed. The results from this analysis are the next page and indicate that both the driving resistance (Chapter 12) and driving stresses (Chapter 11) are within specification limits for the ultimate capacity of 3250 kN. The standard bearing graph indicates a driving resistance of 255 blows per meter at a hammer stroke of 2.40 m should result in the required ultimate pile capacity.

A constant capacity wave equation analysis or inspectors chart was then performed to assist field personnel in the determining the required driving resistance at other field observed hammer strokes. The results of this constant capacity analysis for Pier 2 piles is presented on page 17-69. The analysis results have been furnished to the inspector in expanded form as presented on page 17-70 and should be used to answer the following questions.

1. Pile #1 has a field observed hammer stroke is 2.20 m and a driving resistance of 275 blows/m. Does this pile have the required ultimate capacity?

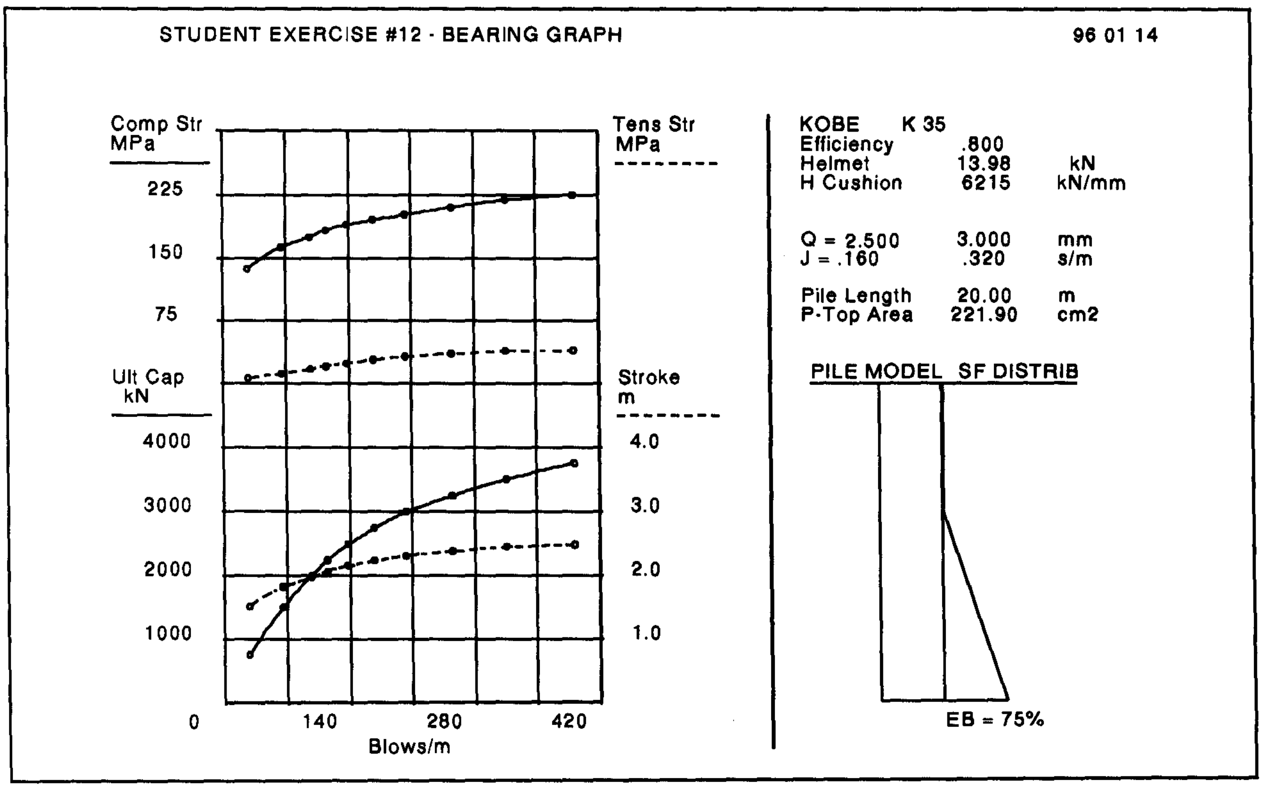
   Any additional action required by the inspector?

2. Pile #2 has a field observed hammer stroke of 2.85 m and a driving resistance of 195 blows/m. Does this pile have the required ultimate capacity?

   Any additional action required by the inspector?

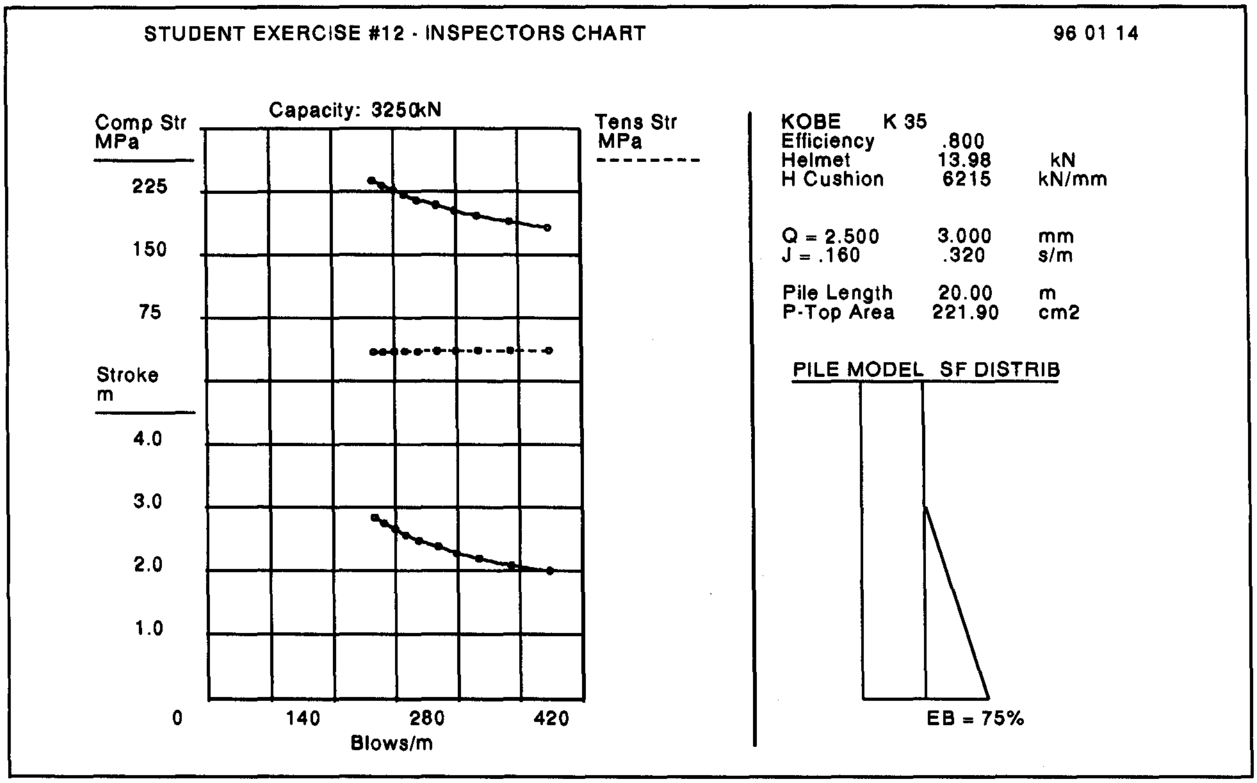
STUDENT EXERCISE #12 - BEARING GRAPH 96/01/14
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.995-1

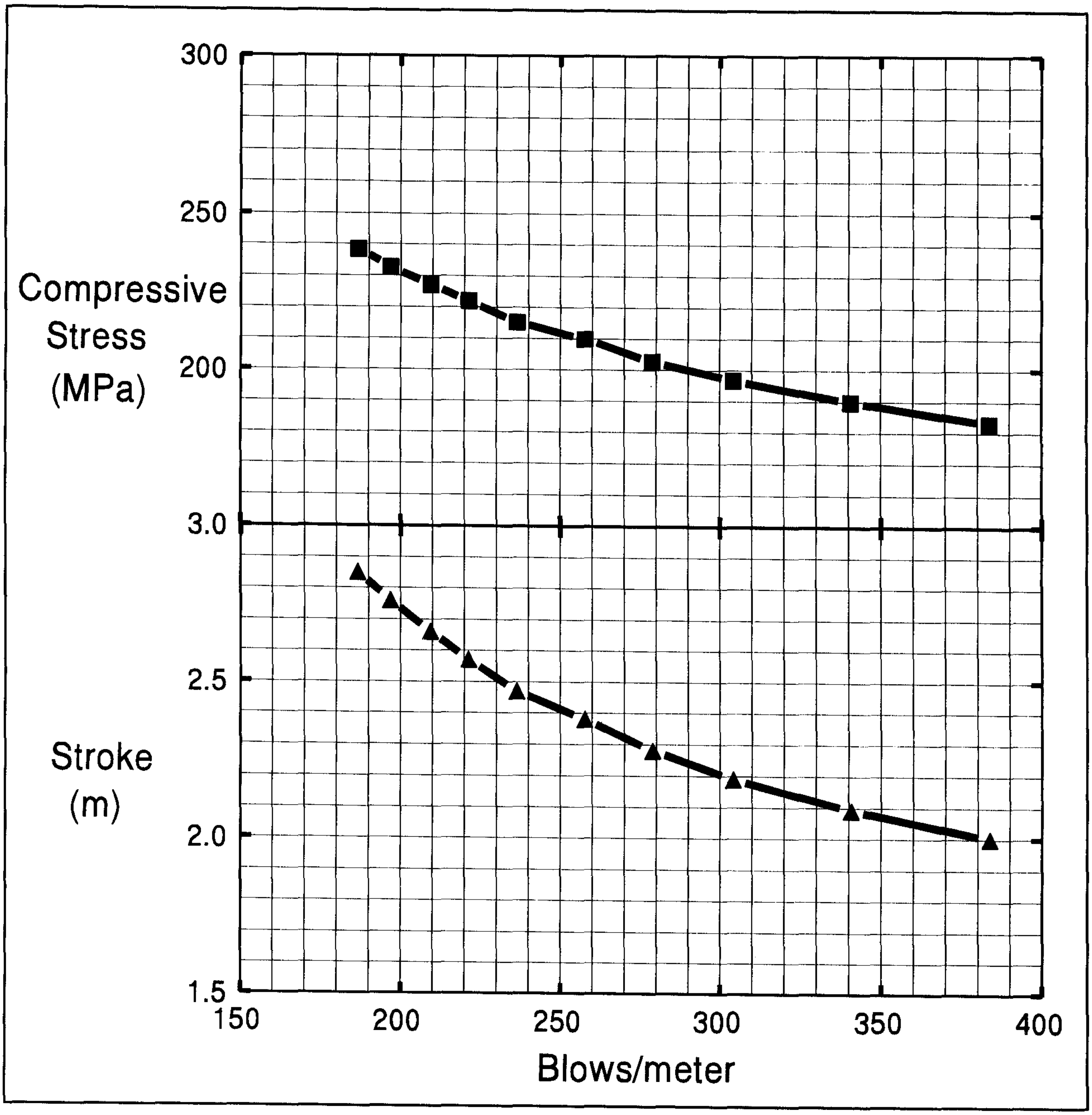
Rut (kN)	Bl Ct (bpm)	Stroke down	(m) up	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)	Bl Rt (b/min)
750.0	27.5	1.52	1.54	-7.39	(9, 46)	137.40	(4, 3)	44.7	52.7
1500.0	65.2	1.82	1.83	-12.30	(10, 30)	163.13	(10, 4)	41.7	48.2
2000.0	96.8	1.97	1.99	-17.55	(11, 27)	175.50	(10, 4)	41.9	46.2
2250.0	114.8	2.08	2.07	-21.69	(12, 26)	183.43	(10, 4)	43.2	45.2
2500.0	138.5	2.16	2.16	-24.44	(12, 24)	189.75	(11, 4)	44.1	44.3
2750.0	167.9	2.24	2.24	-28.96	(11, 23)	195.82	(11, 4)	45.1	43.6
3000.0	203.5	2.32	2.32	-32.91	(11, 23)	201.83	(10, 4)	46.2	42.8
3250.0	255.1	2.39	2.40	-36.08	(11, 22)	210.55	(20, 6)	47.0	42.2
3500.0	315.8	2.46	2.46	-39.09	(11, 22)	219.45	(20, 6)	48.2	41.6
3750.0	392.0	2.49	2.51	-40.49	(10, 22)	225.05	(20, 6)	48.9	41.3



STUDENT EXERCISE #12 - INSPECTORS CHART 96/01/14
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.995-1

Rut (kN)	Bl Ct (bpm)	Stroke down	(m) up	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)	Bl Rt (b/min)
3250.0	383.7	2.00	2.40	-37.17	(11, 22)	182.91	(20, 6)	39.2	44.0
3250.0	340.7	2.09	2.40	-36.94	(11, 22)	189.80	(20, 6)	41.1	43.5
3250.0	304.2	2.19	2.39	-36.76	(11, 22)	196.77	(20, 6)	43.1	43.1
3250.0	278.9	2.28	2.39	-36.44	(11, 22)	202.57	(20, 6)	45.0	42.6
3250.0	257.7	2.38	2.39	-36.13	(11, 22)	209.78	(20, 6)	46.8	42.2
3250.0	236.4	2.47	2.39	-35.92	(11, 22)	215.22	(20, 6)	48.8	41.8
3250.0	221.3	2.57	2.39	-35.61	(11, 22)	221.89	(20, 6)	50.6	41.4
3250.0	209.4	2.66	2.40	-35.26	(11, 22)	227.07	(20, 6)	52.3	41.0
3250.0	196.8	2.76	2.40	-35.01	(11, 23)	232.65	(20, 6)	54.2	40.7
3250.0	186.6	2.85	2.40	-34.69	(11, 23)	238.40	(20, 6)	56.0	40.3




300
250
Compressive
Stress
(MPa)
200
3.0
2.5
Stroke
(m)
2.0
1.5
150
200
250
300
350
400
Blows/meter

18. DYNAMIC PILE TESTING AND ANALYSIS

Dynamic test methods use measurements of strain and acceleration taken near the pile head as a pile is driven or restruck with a pile driving hammer. These dynamic measurements can be used to evaluate the performance of the pile driving system, calculate pile installation stresses, determine pile integrity, and estimate static pile capacity.

Dynamic test results can be further evaluated using signal matching techniques to determine the relative soil resistance distribution on the pile, as well as representative dynamic soil properties for use in wave equation analyses. This chapter provides a brief discussion of the equipment and methods of analysis associated with dynamic measurements.

18.1 BACKGROUND

Work on the development of the dynamic pile testing techniques that have become known as the Case Method started with a Master thesis project at Case Institute of Technology. This work was done by Eiber (1958) at the suggestion and under the direction of Professor H.R. Nara. In this first project, a laboratory study was performed in which a rod was driven into dry sand. The Ohio Department of Transportation (ODOT) and the Federal Highway Administration subsequently funded a project with HPR funds at Case Institute of Technology beginning in 1964. This project was directed by Professors R.H. Scanlan and G.G. Goble. At the end of the first two year phase, Professor Scanlan moved to Princeton University. The research work at Case Institute of Technology under the direction of Professor Goble continued to be funded by ODOT and FHWA, as well as several other public and private organizations until 1976.

Four principal directions were explored during the 12 year period that the funded research project was active. There was a continuous effort to develop improved transducers for the measurement of force and acceleration during pile driving. Field equipment for recording and data processing was also continually improved. Model piles were driven and tested both statically and dynamically at sites in Ohio. Full scale piles driven and statically tested by ODOT, and later other DOT's, were also tested dynamically to obtain capacity correlations. Finally, analysis method improvements were

c. CAPWAP analysis can provide refined estimates of static capacity, assessment of soil resistance distribution, and soil quake and damping parameters for wave equation input.

18.2.2 Hammer and Driving System Performance

a. Calculation of energy transferred to the pile for comparison with the manufacturer's rated energy and wave equation predictions which indicate hammer and drive system performance. Energy transfer can also be used to determine effects of changes in hammer cushion or pile cushion materials on pile driving resistance.

b. Determination of drive system performance under different operating pressures, strokes or batters, or changes in hammer maintenance by comparative testing of hammers or of a single hammer over an extended period of use.

c. Identification of hammer performance problems, such as preignition problems with diesel hammers or preadmission in air/steam hammers.

d. Determination of whether soil behavior or hammer performance is responsible for changes in observed driving resistances.

18.2.3 Driving Stresses and Pile Integrity

a. Calculation of compression and tension driving stresses. In cases with driving stress problems, this information can be helpful when evaluating adjustments to pile installation procedures. Calculated stresses can also be compared to specified driving stress limits.

b. Determination of the extent and location of pile structural damage, Rausche and Goble (1979). Thus, costly extraction may not be necessary to confirm or quantify damage suspected from driving records.

c. CAPWAP analysis for stress distribution throughout pile.

18.3 DYNAMIC TESTING EQUIPMENT

A typical dynamic testing system consists of a minimum of two strain transducers and two accelerometers bolted to diametrically opposite sides of the pile to monitor strain and acceleration and account for nonuniform hammer impacts and pile bending. The reusable strain transducers and accelerometers are generally attached two to three diameters below the pile head. Almost any driven pile type (concrete, steel pipe, H, Monotube, timber, *etc.*) can be tested with the pile preparation for each pile type slightly varying.

Figures 18.1 and 18.2 illustrate the typical pile preparation procedures required for dynamic testing. In Figure 18.1, a prestressed concrete pile is being prepared for gage attachment by drilling and then installing concrete anchors. In Figure 18.2, the concrete pile to be tested during driving has been positioned in the leads for driving. A member of the pile crew climbs the leads and then bolts the gages to the pile at this time. Piles to be tested during restrike can be instrumented at any convenient location and the climbing of the leads is usually not necessary. Pile preparation and gage attachment typically requires 10 to 20 minutes per pile tested. After the gages are attached, the driving or restrike process continues following usual procedures. Most restrike tests are only 20 blows or less.

A close up view of a strain transducer and an accelerometer bolted to a steel pipe pile is shown in Figure 18-3. The individual cables from each gage are combined into a single main cable which in turn relays the signals from each hammer blow to the data acquisition system on the ground. The data acquisition system, such as the Pile Driving Analyzer shown in Figure 18-4, conditions and converts the strain and acceleration signals to force and velocity records versus time. The force is computed from the measured strain, ϵ, times the product of the pile elastic modulus, E, and cross sectional area, A, or: $F(t) = EA\epsilon(t)$. The velocity is obtained by integrating the measured acceleration, record, a, or: $V(t) = \int a(t)\,dt$.

Older dynamic testing systems required multiple components for processing, recording, and display of dynamic test signals. In newer dynamic testing systems, these components have been combined into one PC computer based system. During driving, the Pile Driving Analyzer performs integrations and all other required computations to analyze the dynamic records for transferred energy, driving stresses, structural integrity, and pile capacity. Numerical results for each blow for up to nine dynamic quantities are

Figure 18.1 Pile Preparation for Dynamic Testing

Figure 18.2 Pile Positioned for Driving and Gage Attachment

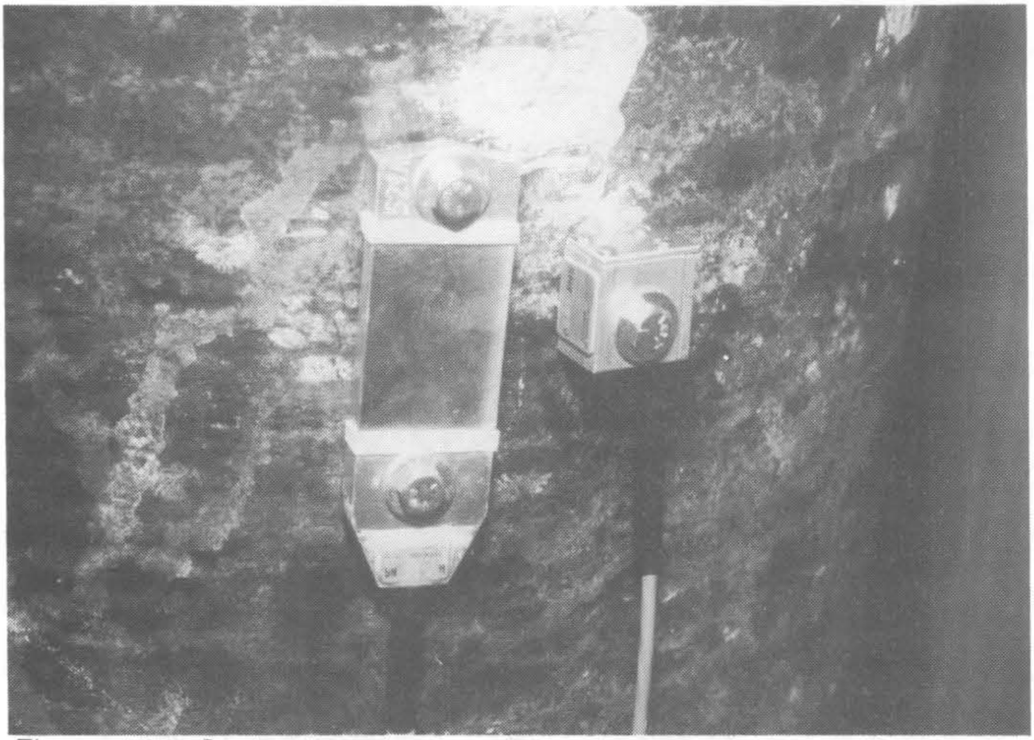


Figure 18.3 Strain Transducer and Accelerometer Bolted to Pipe Pile

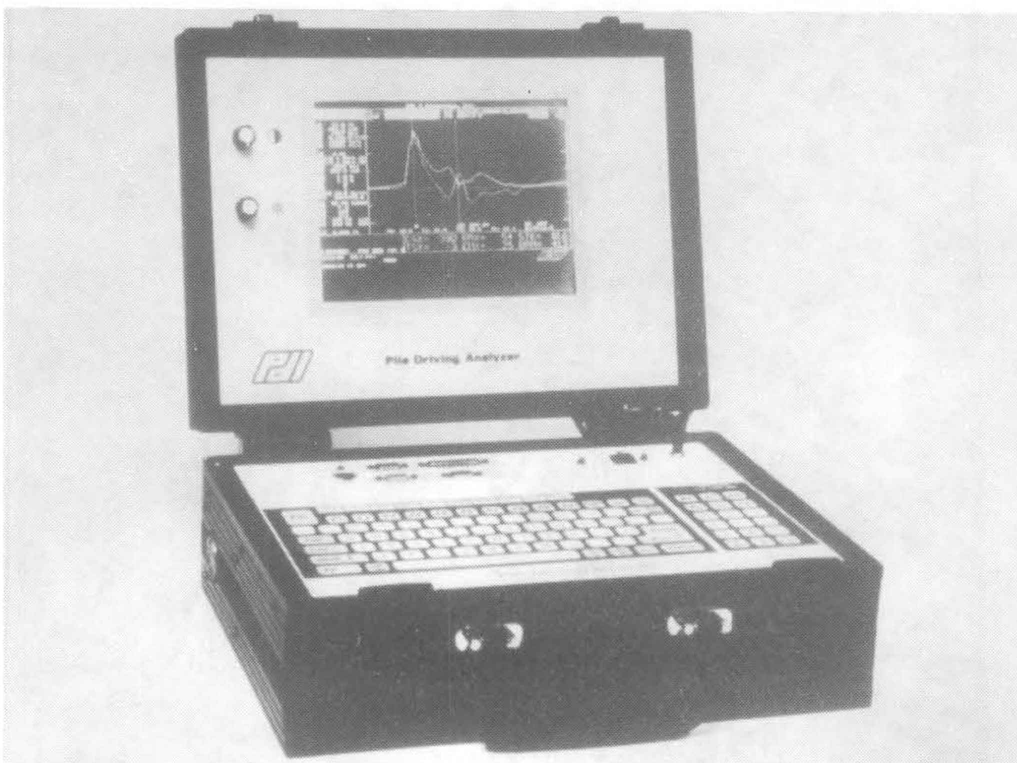


Figure 18.4 Pile Driving Analyzer (courtesy of Pile Dynamics, Inc.)

electronically stored in a file which can be later used to produce graphical and numeric summary outputs. In this system, force and velocity records are also viewed on a graphic LCD computer screen during pile driving to evaluate data quality, soil resistance distribution, and pile integrity. Complete force and velocity versus time records from each gage are also digitally stored for later reprocessing and data analysis by CAPWAP.

Data quality is automatically evaluated by the Pile Driving Analyzer and if any problem is detected, then a warning is given to the test engineer. Other precautionary advice is also displayed to assist the engineer in collecting data. The capabilities discussed in the remainder of this chapter are those included in these newer systems.

Additional information on the equipment requirements for dynamic testing are detailed in ASTM D-4945, Standard Test Method for High Strain Dynamic Testing of Piles and in AASHTO T-298-33, Standard Method of Test for High Strain Dynamic Testing of Piles.

18.4 BASIC WAVE MECHANICS

This section is intended to summarize wave mechanics principles applicable to pile driving. Through this general overview, an understanding of how dynamic testing functions and how test results can be qualitatively interpreted can be obtained.

When a uniform elastic rod of cross sectional area, A, elastic modulus, E, and wave speed, C, is struck by a mass, then a force, F, is generated at the impact surface of the rod. This force compresses the adjacent part of the rod. Since the adjacent material is compressed, it also experiences an acceleration and attains a particle velocity, V. As long as there are no resistance effects on the uniform rod, the force in the rod will be equal to the particle velocity times the rod impedance, EA/C.

Figure 18.5(a) illustrates a uniform rod of length, L, with no resistance effects, that is struck at one end by a mass. Force and velocity (particle velocity) waves will be created in the rod, as shown in Figure 18.5(b). These waves will then travel down the rod at the material wave speed, C. At time L/C, the waves will arrive at the end of the rod, as shown in Figures 18.5(c) and 18.5(d). Since there are no resistance effects acting on the rod, a free end condition exists, and a tensile wave reflection occurs, which doubles the pile velocity at the free end and the net force becomes zero. The wave then travels up the rod with force of the same magnitude as the initial input, except in tension, and the velocity of the same magnitude and same sign.

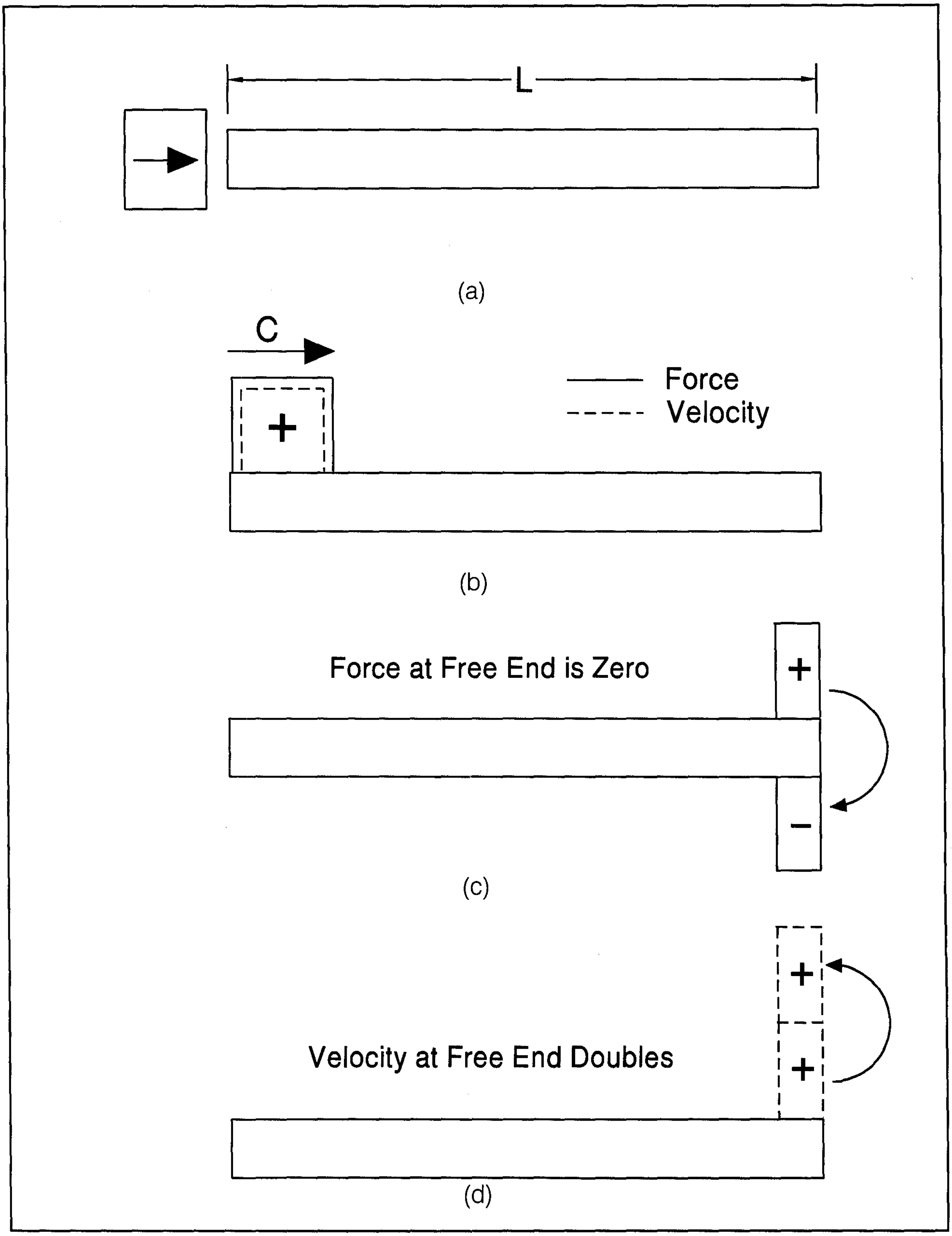


Figure 18.5 Free End Wave Mechanics

Consider now that the rod is a pile with no resistance effects, and that force and velocity measurements are made near the pile head. Typical force and velocity measurements versus time for this "free end" condition are presented in Figure 18.6. The toe response in the records occurs at time 2L/C. This is the time required for the waves to travel to the pile toe and back to the measurement location, divided by the wave speed. Since there are no resistance effects acting on the pile shaft, the force and velocity records are equal until the reflection from the free end condition arrives at the measurement location. At time 2L/C, the force wave goes to zero and the velocity wave doubles in magnitude. Note the repetitive pattern in the records at 2L/C intervals generated as the waves continue to travel down and up the pile. This illustration is typical of an easy driving situation where the pile "runs" under the hammer blow.

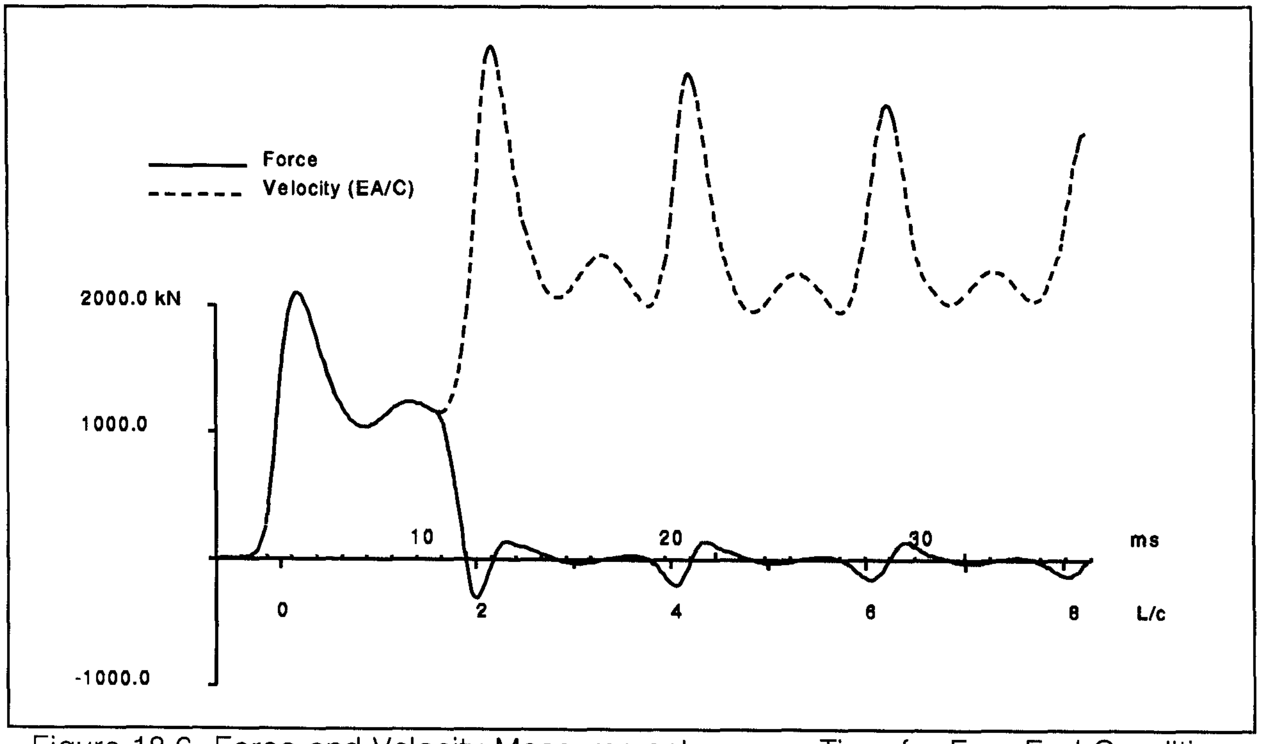


Figure 18.6 Force and Velocity Measurements versus Time for Free End Condition

Figure 18.7(a) illustrates a uniform rod of length, L, that is struck by a mass. Again there are no resistance effects along the rod length, but the pile end is fixed, *i.e.*, it is prevented by some mechanism from moving in such a manner that the particle velocity must be zero at that point. The mass impact will impart force and velocity waves in the rod as shown in Figure 18.7(b). These waves will again travel down the rod at the

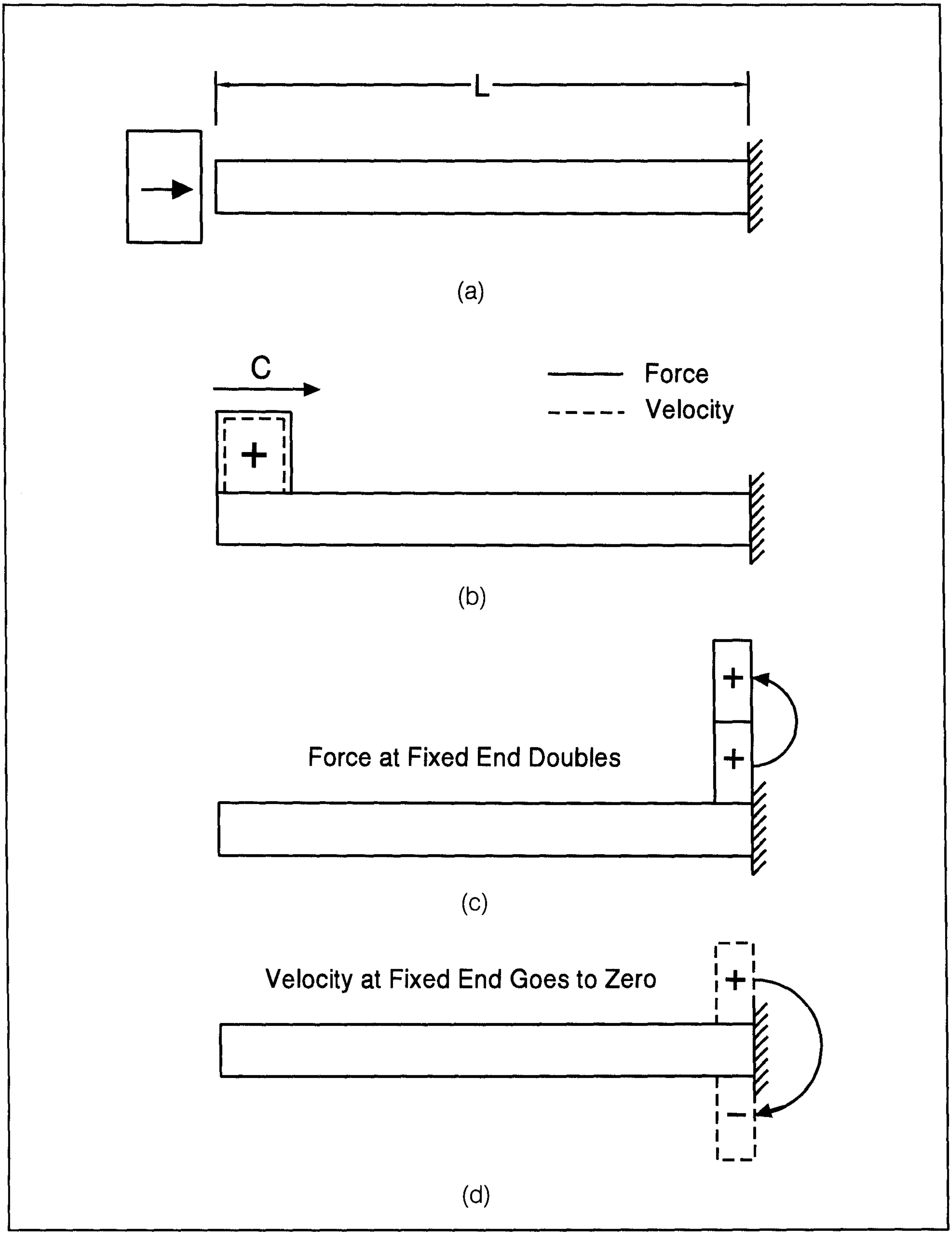


Figure 18.7 Fixed End Wave Mechanics

material wave speed, C. At time L/C, the waves will arrive at the end of the rod as shown in Figures 18.7(c) and 18.7(d). There the fixed end condition will cause a compression wave reflection and therefore the force at the fixed end doubles in magnitude and the pile velocity becomes zero. A compression wave then travels up the rod.

Consider now that the rod is a pile with a fixed end condition and that force and velocity measurements are again made near the pile head. The force and velocity measurements versus time for this condition are presented in Figure 18.8. Since there are no resistance effects acting on the pile shaft, the force and velocity records are equal until the reflection from the fixed end condition arrives at the measurement location. At time 2L/C, the force wave increases in magnitude and the velocity wave goes to zero. This illustration is typical of a hard driving situation where the pile is driven to rock.

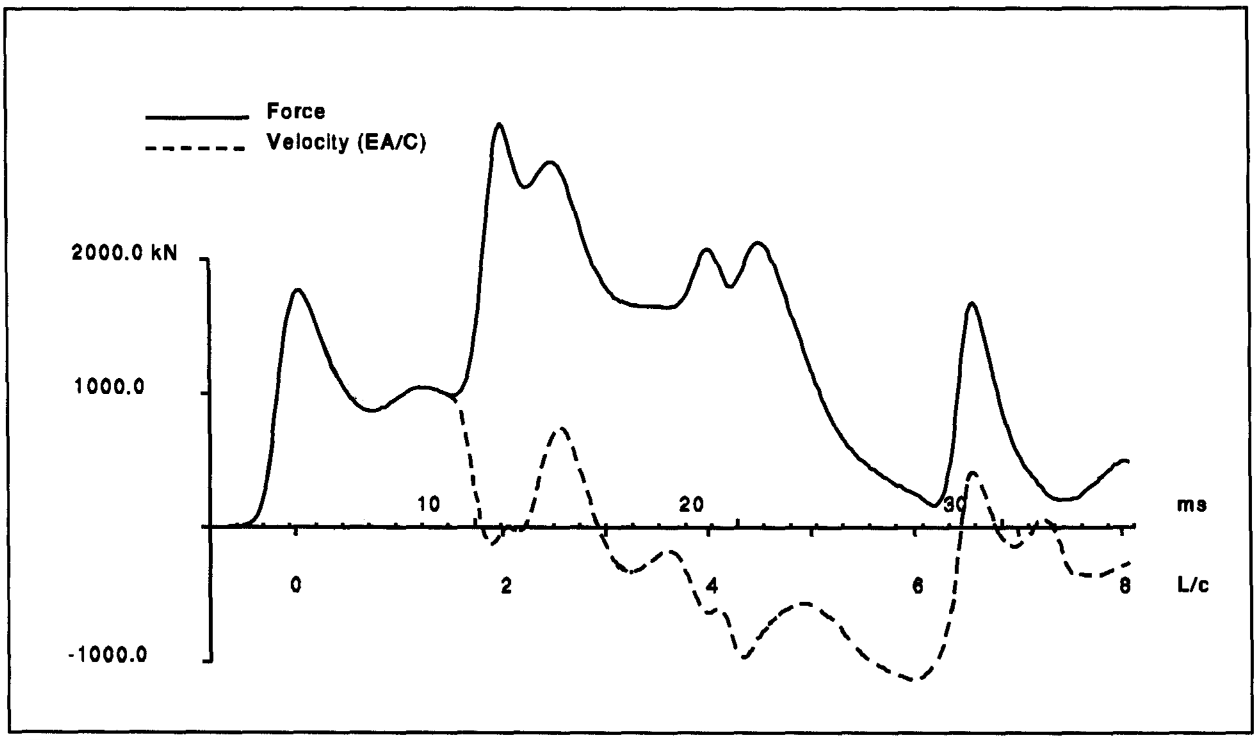


Figure 18.8 Force and Velocity Measurements versus Time for Fixed End Condition

As discussed above, the force and velocity records versus time are equal or proportional at impact and remain proportional thereafter until affected by soil resistance or cross sectional changes. Reflections from either effect will arrive at the measurement location at time 2X/C where X is the distance to the soil resistance or cross section change. Both soil resistance effects and cross sectional increases will cause an increase in the force record and a proportional decrease in the velocity record. Conversely, cross sectional reductions, such as those caused by pile damage, will cause a decrease in the force record and an increase in the velocity record.

The concept of soil resistance effects on force and velocity records can be further understood by reviewing the theoretical soil resistance example presented in Figure 18.9. In this case, the soil resistance on a pile consists only of a small resistance located at a depth, A, below the measurement location, and a larger soil resistance at depth B. No other resistance effects act on the pile, so a free end condition is present at the pile toe. The force and velocity records versus time for this example will be proportional until time 2A/C, when the reflection from the small soil resistance effect arrives at the measurement location. This soil resistance reflection will then cause a small increase in the force record and a small decrease in the velocity record.

No additional soil resistance effects act on the pile between times 2A/C and 2B/C. Therefore, the force and velocity records will remain parallel over this time interval with no additional separation. At time 2B/C, the reflection from the large soil resistance effect will arrive at the measurement location. This large soil resistance reflection will then cause a large increase in the force record and a large decrease in the velocity record. No additional soil resistance effects act on the pile between times 2B/C and 2L/C. Therefore, the force and velocity records will again remain parallel over this time interval with no additional separation between the records.

At time 2L/C, the reflection from the pile toe will arrive at the measurement location. Since no resistance is present at the pile toe, a free end condition exists and a tensile wave will be reflected. Hence, an increase in the velocity record and a decrease in the force record will occur.

These basic interpretation concepts of force and velocity records versus time can be used to qualitatively evaluate the soil resistance effects on a pile. In Figure 18.10(a), minimal separation occurs between the force and velocity records between time 0, or the time of impact, and time 2L/C. In addition, a large increase in the velocity record and corresponding decrease in the force record occurs at time 2L/C. Hence, this record indicates minimal shaft and toe resistance on the pile.

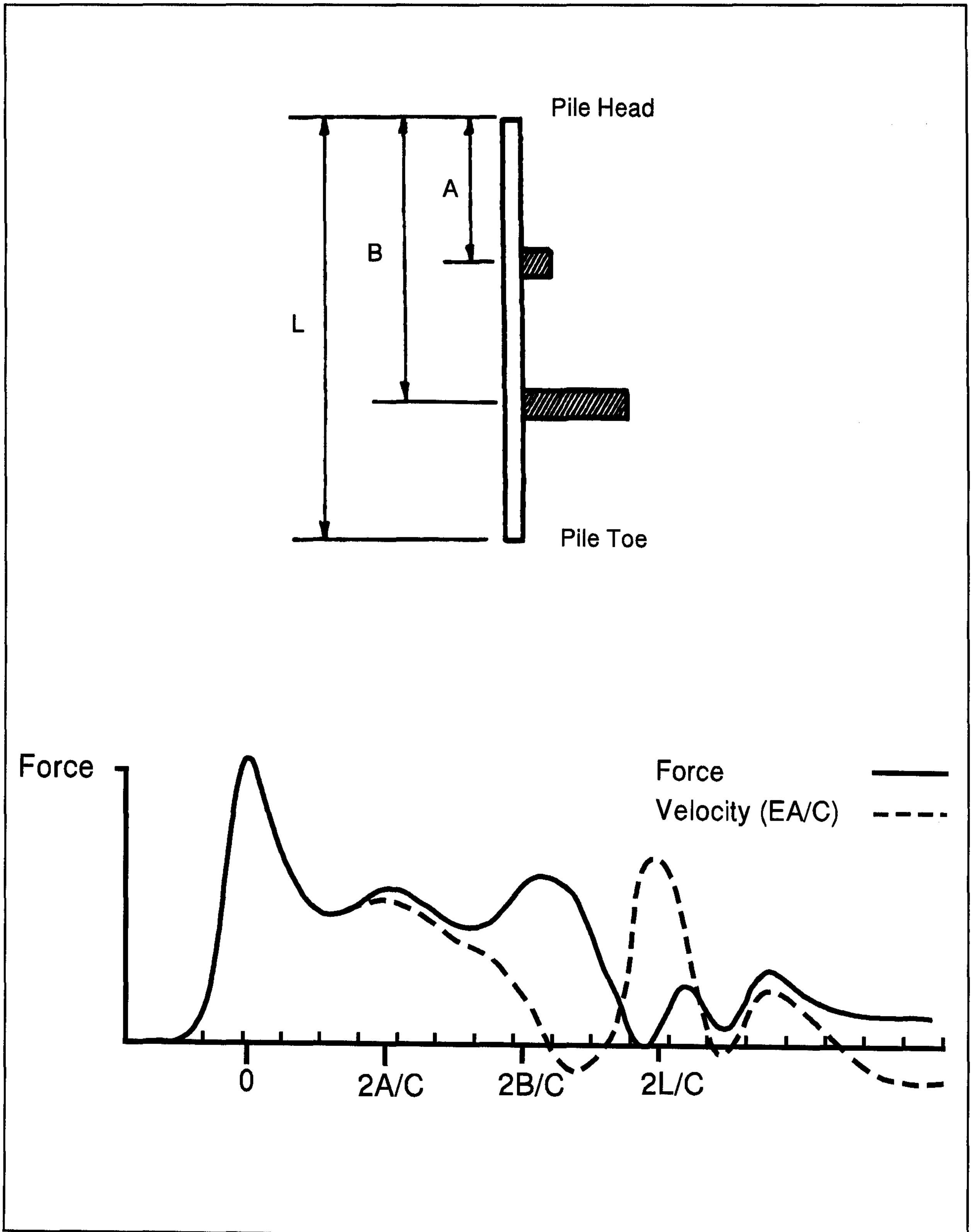


Figure 18.9 Soil Resistance Effects on Force and Velocity Records (after Hannigan, 1990)

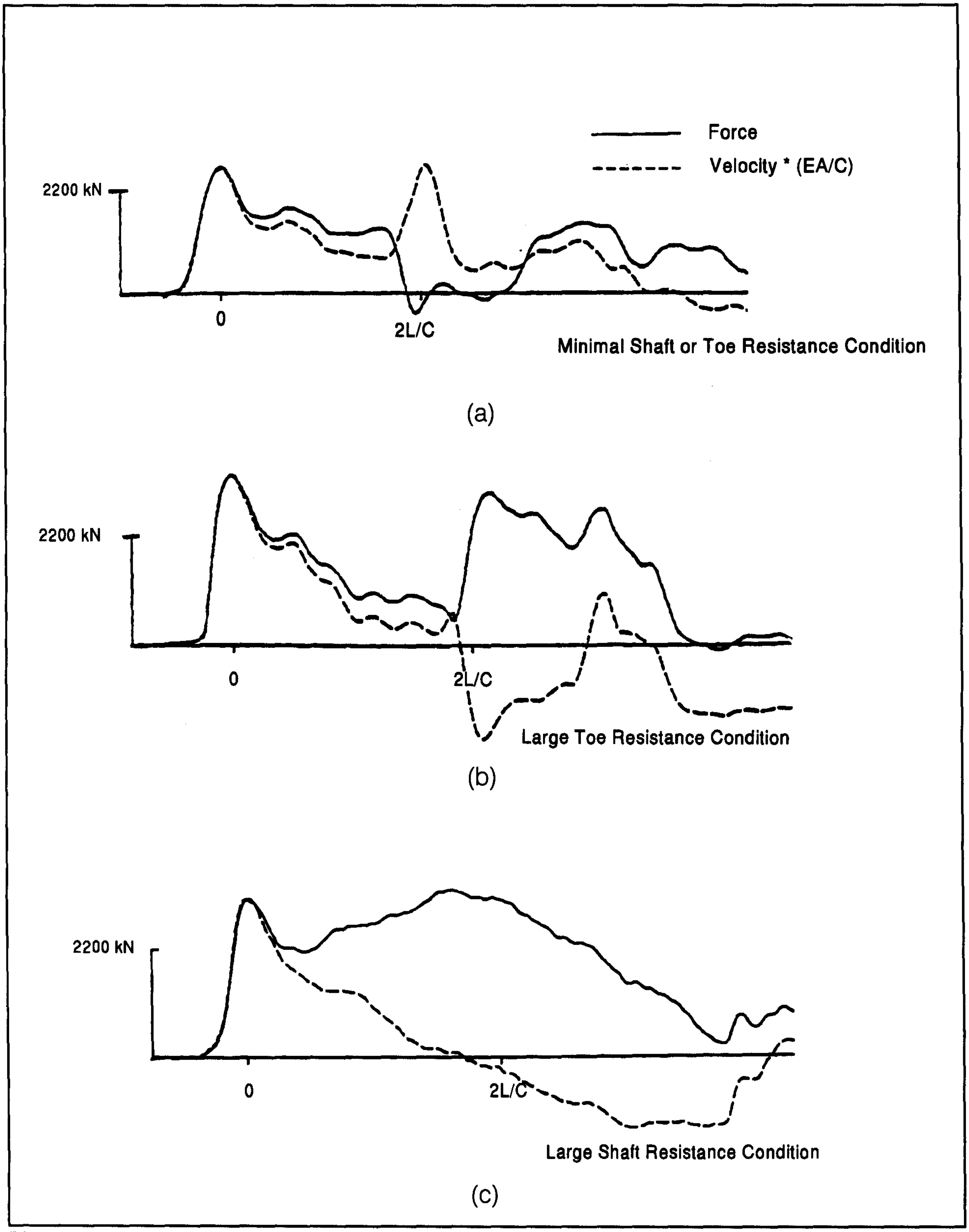


Figure 18.10 Typical Force and Velocity Records for Various Soil Resistance Conditions (after Hannigan, 1990)

In Figure 18.10(b), minimal separation again occurs between the force and velocity records between time 0 and time 2L/C. However in this example, a large increase in the force record and corresponding decrease in the velocity record occurs at time 2L/C. Therefore, this force and velocity record indicates minimal shaft and a large toe resistance on the pile.

In Figure 18.10(c), a large separation between the force and velocity records occurs between time 0 and time 2L/C. This force and velocity record indicates a large shaft resistance on the pile.

18.5 DYNAMIC TESTING METHODOLOGY

As introduced in Section 18.1, two methods have developed for analyzing dynamic measurement data, the Case Method and CAPWAP. In the field, the Pile Driving Analyzer uses the Case Method equations for estimates of static pile capacity, calculation of driving stresses and pile integrity, as well as computation of transferred hammer energy. The CAPWAP analysis method is a more rigorous numerical analysis procedure that uses dynamic records of force and velocity along with wave equation type pile and soil modeling to calculate static pile capacity, the relative soil resistance distribution, and dynamic soil properties of quake and damping. Static pile capacity evaluation from these two methods will be described in greater detail in subsequent sections. For additional details of the dynamic analysis procedures, references are provided at the end of this chapter.

18.5.1 Case Method Capacity

Research conducted at Case Western Reserve University in Cleveland, Ohio, resulted in a method which uses electronic measurements taken during pile driving to predict static pile capacity. Assuming the pile is linearly elastic and has constant cross section, the total static and dynamic resistance on a pile during driving, RTL, can be expressed using the following equation, which was derived from a closed form solution to the one dimensional wave propagation theory:

$$RTL = 1/2[F(t_1) + F(t_2)] + 1/2[V(t_1) - V(t_2)]\ EA/C$$

Where: F = Force measured at gage location.
V = Velocity measured at gage location.
t_1 = Time of initial impact.
t_2 = Time of reflection of initial impact from pile toe (t_1 + 2L/C).
E = Pile modulus of elasticity.
C = Wave speed of pile material.
A = Pile area at gage location.
L = Pile length below gage location.

To obtain the static pile capacity, the dynamic resistance (damping) must be subtracted from the above equation. Goble *et al.* (1975) found that the dynamic resistance component could be approximated as a linear function of a damping factor times the pile toe velocity, and that the pile toe velocity could be estimated from dynamic measurements at the pile head. This led to the standard Case Method capacity equation, RSP, expressed below:

$$RSP = RTL - J[V(t_1) EA/C + F(t_1) - RTL]$$

Where: J = Dimensionless damping factor based on soil type near the pile toe.

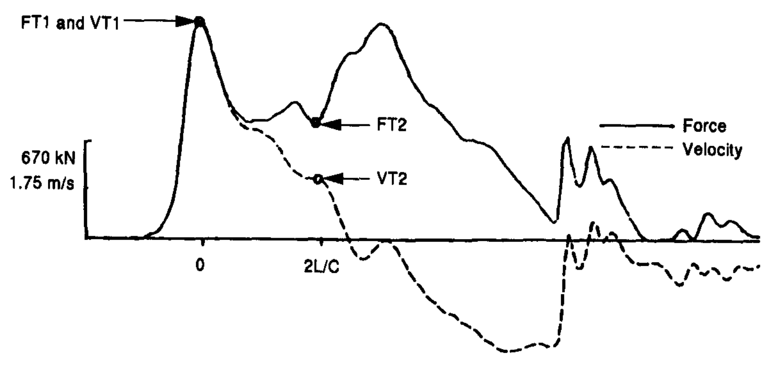
Typical damping factors versus soil type at the pile toe were determined by finding the range in the Case damping factor, J, for a soil type that provided a correlation of the RSP static capacity within 20% of the static load test failure load, determined using the Davisson (1972) offset limit method. The original range in Case damping factor versus soil type from this correlation study, Goble *et al.* (1975), as well as typical ranges in Case damping factor for the RSP equation based on subsequent experience, Pile Dynamics, Inc. (1996), are presented in Table 18-1. While use of these values with the RSP equation may provide good initial capacity estimates, site specific damping correlations should be developed based upon static load test results or CAPWAP analysis. It should also be noted that Case damping is a non-dimensional damping factor and is not the same as the Smith damping discussed in Chapter 17 for wave equation analysis.

TABLE 18-1 SUMMARY OF CASE DAMPING FACTORS FOR RSP EQUATION		
Soil Type at Pile Toe	Original Case Damping Correlation Range Goble *et al.* (1975)	Updated Case Damping Ranges Pile Dynamics (1996)
Clean Sand	0.05 to 0.20	0.10 to 0.15
Silty Sand, Sand Silt	0.15 to 0.30	0.15 to 0.25
Silt	0.20 to 0.45	0.25 to 0.40
Silty Clay, Clayey Silt	0.40 to 0.70	0.40 to 0.70
Clay	0.60 to 1.10	0.70 or higher

The RSP or standard Case Method equation is best used to evaluate the capacity of low displacement piles, and piles with large shaft resistances. For piles with large toe resistances and for displacement piles driven in soils with large toe quakes, the toe resistance is often delayed in time. This condition can be identified from the force and velocity records. In these instances, the standard Case Method equation may indicate a relatively low pile capacity and the maximum Case Method equation, RMX, should be used. The maximum Case Method equation searches for the t_1 time in the force and velocity records which results in the maximum capacity. An example of this technique is presented in Figure 18.11. When using the maximum Case Method equation, experience has shown that the Case damping factor should be at least 0.4, and on the order of 0.2 higher than that used for the standard Case Method capacity equation, RSP.

The RMX and RSP Case Method equations are the two most commonly used solutions for field evaluation of pile capacity. Additional automatic Case Method solutions are available that do not require selection of a Case damping factor. These automatic methods, referred to as RAU and RA2, search for the time when the pile toe velocity is zero and hence damping is minimal. The RAU method may be applicable for piles with minimal shaft resistance and the RA2 method may be applicable to piles with toe resistance plus moderate shaft resistance. It is recommended that these automatic methods be used as supplemental indicators of pile capacity where appropriate with the more traditional standard or maximum Case Method equations primarily used to evaluate pile capacity.

STANDARD CASE METHOD, RSP



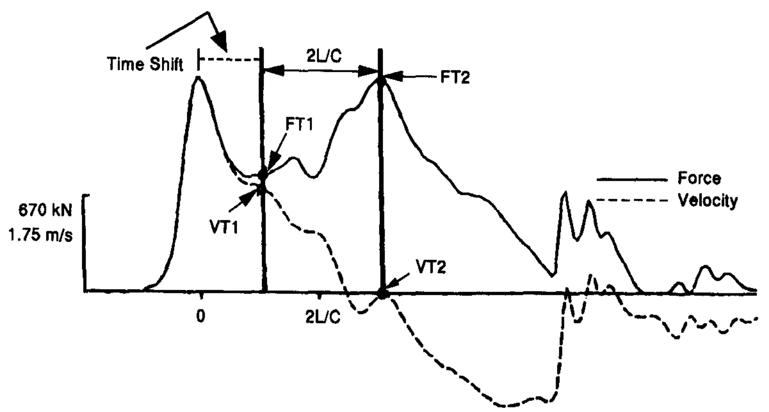
TOTAL RESISTANCE

RTL = 1/2 (FT1 + FT2) + 1/2 (VT1 - VT2) (EA/C)
= 1/2 (1486 + 819) + 1/2 (3.93 -1.07) 381
= 1153 + 545 = 1698 kN.

STATIC RESISTANCE

RSP = RTL - J[VT1 (EA/C) + FT1 - RTL]
= 1698 - 0.4 [3.93 (381) + 1486 - 1698]
= 1698 - 514 = 1184 kN.

MAXIMUM CASE METHOD, RMX



TOTAL RESISTANCE

RTL = 1/2 (FT1 + FT2) + 1/2 (VT1 - VT2) (EA/C)
= 1/2 (819 + 1486) + 1/2 (1.92 - 0.0) 381
= 1153 + 366 = 1519 kN.

STATIC RESISTANCE

RMX = RTL - J[VT1 (EA/C) + FT1 - RTL]
= 1519 - 0.7 [1.92 (381) + 819 - 1519]
= 1519 - 22 = 1497 kN.

Figure 18.11 Standard, RSP and Maximum, RMX, Case Method Capacity Estimates

18.5.2 Energy Transfer

The energy transferred to the pile head can be computed from the strain and acceleration measurements. As described in Section 18.3, the acceleration signal is integrated to obtain velocity and the strain measurement is converted to force. Transferred energy is equal to the work done which can be computed from the integral of the force and velocity records over time as given below:

$$E_p(t) = \int_0^t F(t)\,V(t)\,dt$$

Where:
- E_p = The energy at the gage location expressed as a function of time.
- F = The force at the gage location expressed as a function of time.
- V = The velocity at the gage location expressed as a function of time.

This procedure is illustrated in Figure 18.12. The maximum energy transferred to the pile head corresponds to the maximum value of $E_p(t)$ and can be used to evaluate the performance of the hammer and driving system as described in Section 18.7.

18.5.3 Driving Stresses and Integrity

The Pile Driving Analyzer calculates the compression stress at the gage location using the measured strain and pile modulus of elasticity. However, the maximum compression stress in the pile may be greater than the compression stress calculated at the gage location, such as in the case of a pile driven through soft soils to rock. In these cases CAPWAP or wave equation analysis may be used to evaluate the maximum compression stress in the pile. Computed tension stresses are based upon the superposition of the upward and downward traveling force waves calculated by the Pile Driving Analyzer.

The basic concepts of wave mechanics were presented in Section 18.4. Convergence between the force and velocity records prior to the toe response at time 2L/C indicates an impedance (EA/C) reduction in the pile. For uniform cross section piles an impedance reduction is therefore pile damage. The degree of convergence between the force and velocity records is termed BTA, which can be used to evaluate pile damage following the guidelines presented in Rausche and Goble, (1979). These guidelines are provided in Table 18-2. Piles with BTA values below 80% correspond to damaged or broken piles.

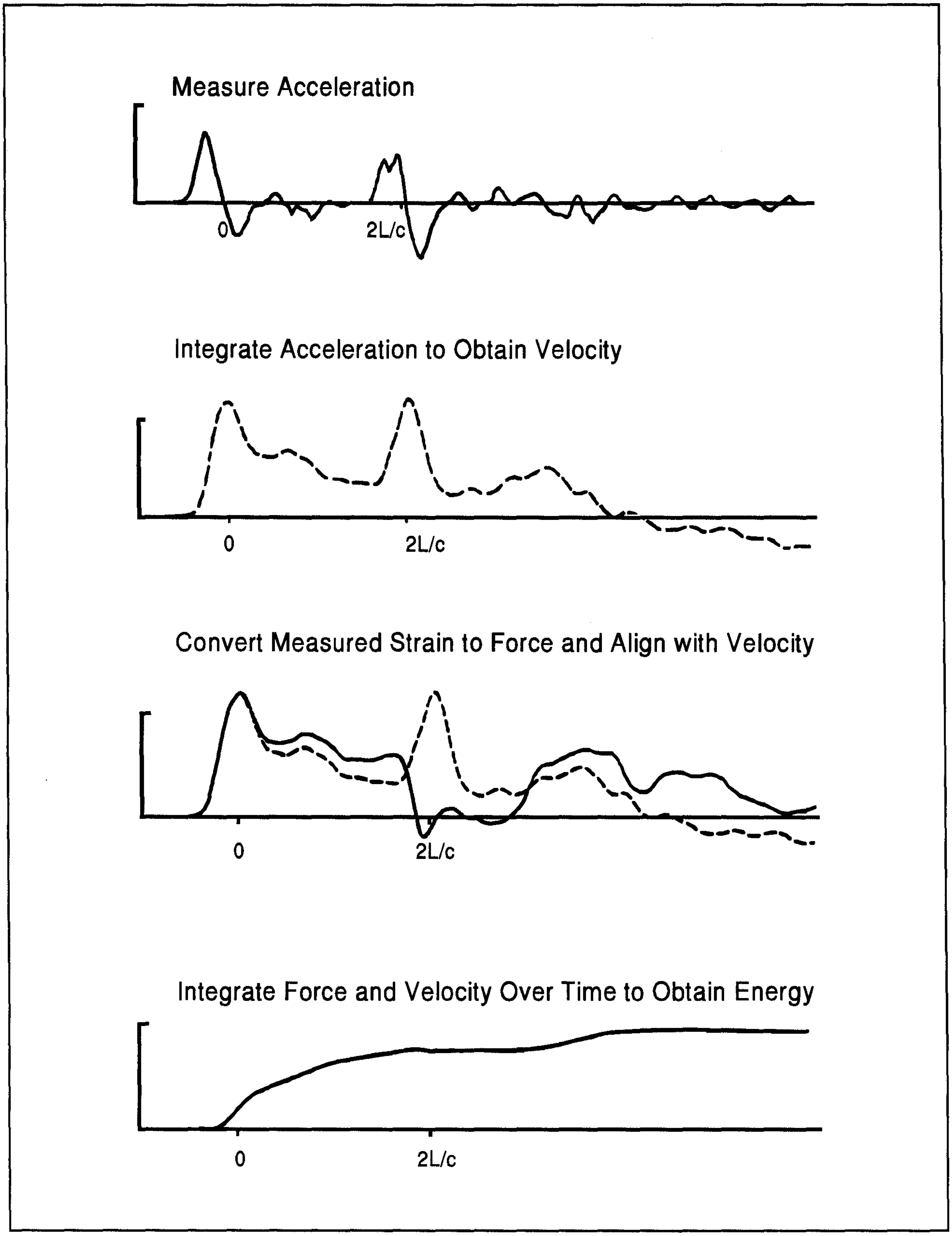


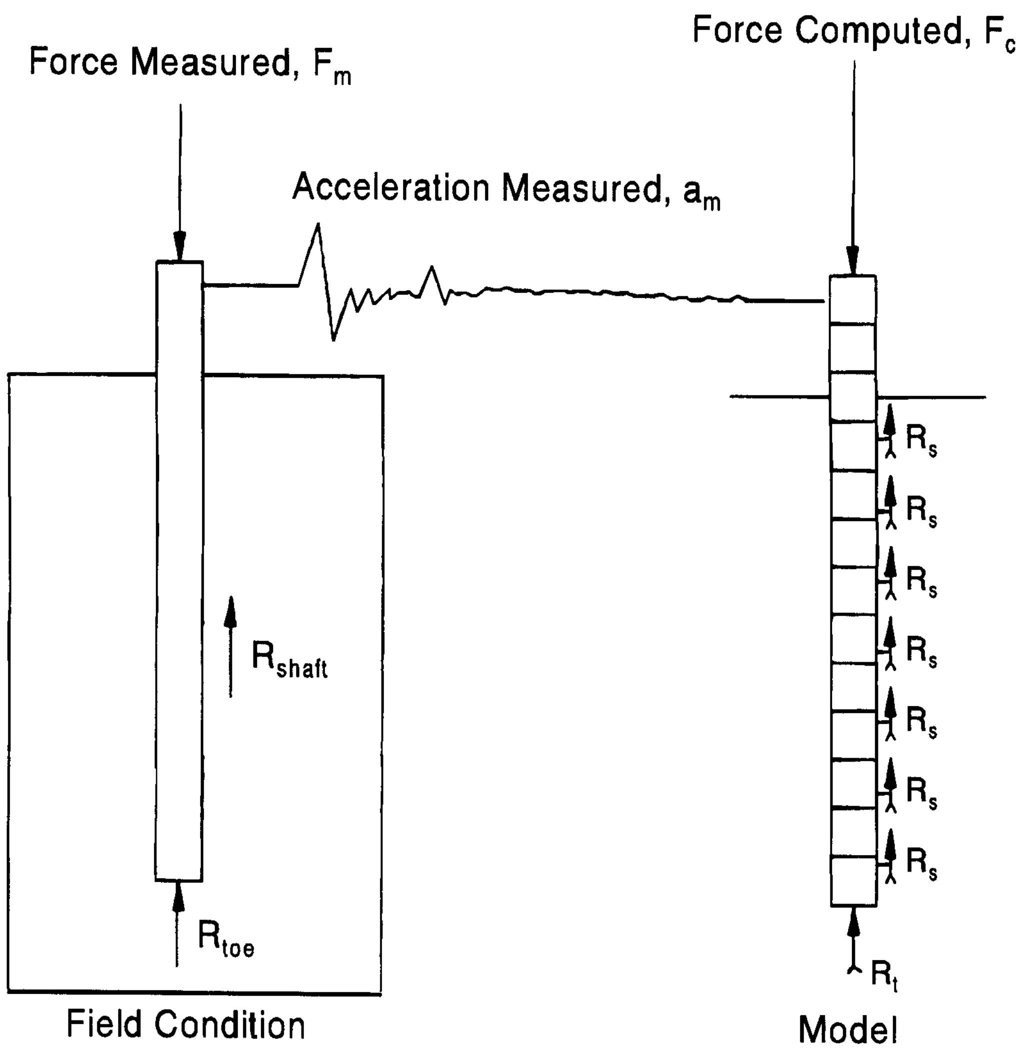
Figure 18.12 Energy Transfer Computation (after Hannigan, 1990)

TABLE 18-2 PILE DAMAGE GUIDELINES (Rausche and Goble, 1979)	
BTA	Severity of Damage
1.0	Undamaged
0.8 - 1.0	Slightly Damaged
0.6 - 0.8	Damaged
Below 0.6	Broken

18.5.4 The CAPWAP Method (CAse Pile Wave Analysis Program)

CAPWAP is a computer program for a more rigorous evaluation of static pile capacity, the relative soil resistance distribution, and soil quake and damping characteristics. A CAPWAP analysis is performed on an individual hammer blow that is usually selected from the end of driving or beginning of restrike. As such, a CAPWAP analysis refines the Case Method dynamic test results at a particular penetration depth or time. CAPWAP uses wave equation type pile and soil models; the Pile Driving Analyzer measured force and velocity records are used as the head boundary condition, replacing the hammer model.

In the CAPWAP method depicted in Figure 18.13, the pile is modeled by a series of continuous pile segments and the soil resistance modeled by elasto-plastic springs (static resistance) and dashpots (dynamic resistance). The force and acceleration data from the Pile Driving Analyzer are used to quantify pile force and pile motion, which are two of the three unknowns. The remaining unknown is the boundary conditions, which are defined by the soil model. First, reasonable estimates of the soil resistance distribution and quake and damping parameters are made. Then, the measured acceleration is used to set the pile model in motion. The program then computes the equilibrium pile head force, which can be compared to the Pile Driving Analyzer determined force. Initially, the computed and measured pile head forces will not agree with each other. Adjustments are made to the soil model assumptions and the calculation process repeated.



1. Measure F_m, a_m
2. Compute $F_c = F_c(a_m, R_s, R_t)$
3. Compare $F_m \leftrightarrow F_c$
4. Correct R_s, R_t
5. Iterate (go to 2)

Figure 18.13 Schematic of CAPWAP Analysis Method

In the CAPWAP matching process, the ability to match the measured and computed waves at various times is controlled by different factors. Figure 18.14 illustrates the factors that most influence match quality in a particular zone. The assumed shaft resistance distribution has the dominant influence on match quality beginning with the rise of the record at time t_r before impact and continuing for a time duration of 2L/C thereafter. This is identified as Zone 1 in Figure 18.14.

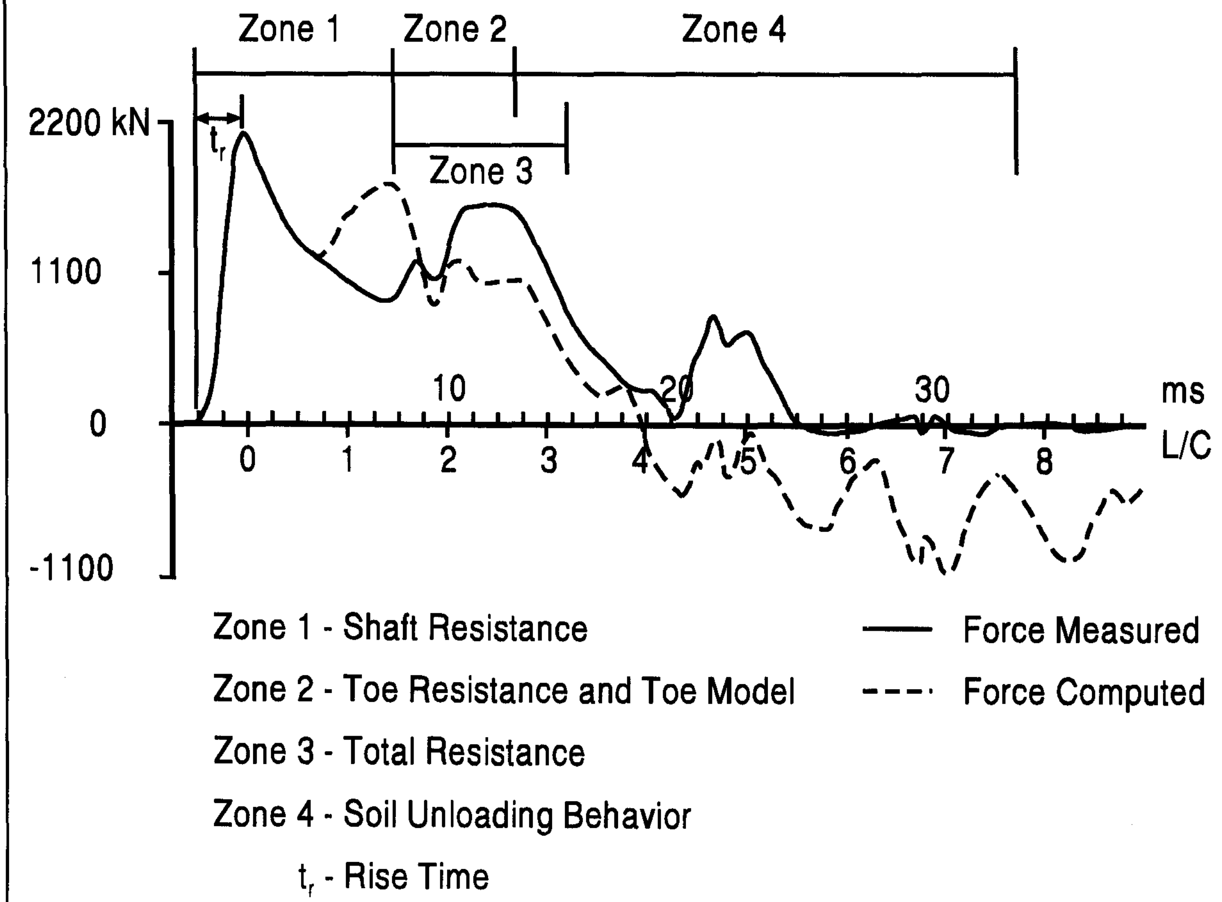


Figure 18.14 Factors Most Influencing CAPWAP Force Wave Matching (after Hannigan, 1990)

In Zone 2, the toe resistance and toe model (toe damping, toe quake and toe gap) most influence the wave match. Zone 2 begins where Zone 1 ends and continues for a time duration equal to the rise time, t_r plus 3 ms. During Zone 3, which begins where Zone 1 ends and continues for a time duration of the rise time t_r plus 5 ms, the overall capacity controls the match quality. A good wave match in Zone 3 is essential for accurate capacity assessments. Zone 4 begins at the end of Zone 2 and continues for a duration of about 20 ms. The unloading behavior of the soil most influences match quality in this zone.

With each analysis, the program evaluates the match quality by summing the absolute values of the relative differences between the measured and computed waves. The program computes a match quality number for each analysis that is the sum of the individual match quality numbers for each of these four zones. An illustration of the CAPWAP iteration process is presented in Figure 18.15.

Through this trial and error iteration adjustment process to the soil model as illustrated in Figure 18.13, the soil model is refined until no further agreement can be obtained between the measured and computed pile head forces. The resulting soil model is then considered the best estimate of the static pile capacity, the soil resistance distribution, and the soil quake and damping characteristics. An example of the final CAPWAP result summary is presented in 18.16. A summary of the stress distribution throughout the pile is also obtained as illustrated in Figure 18.17. Lastly, CAPWAP includes a simulated static load-set graph based on the CAPWAP calculated static resistance parameters and the elastic compression characteristics of the pile.

CAPWAP is a proprietary computer program of Goble, Rausche, Likins and Associates, Inc. and the program software is available from the developer. Alternatively, analysis of dynamic test data can be obtained from the developer or other consulting engineers who have acquired program licenses.

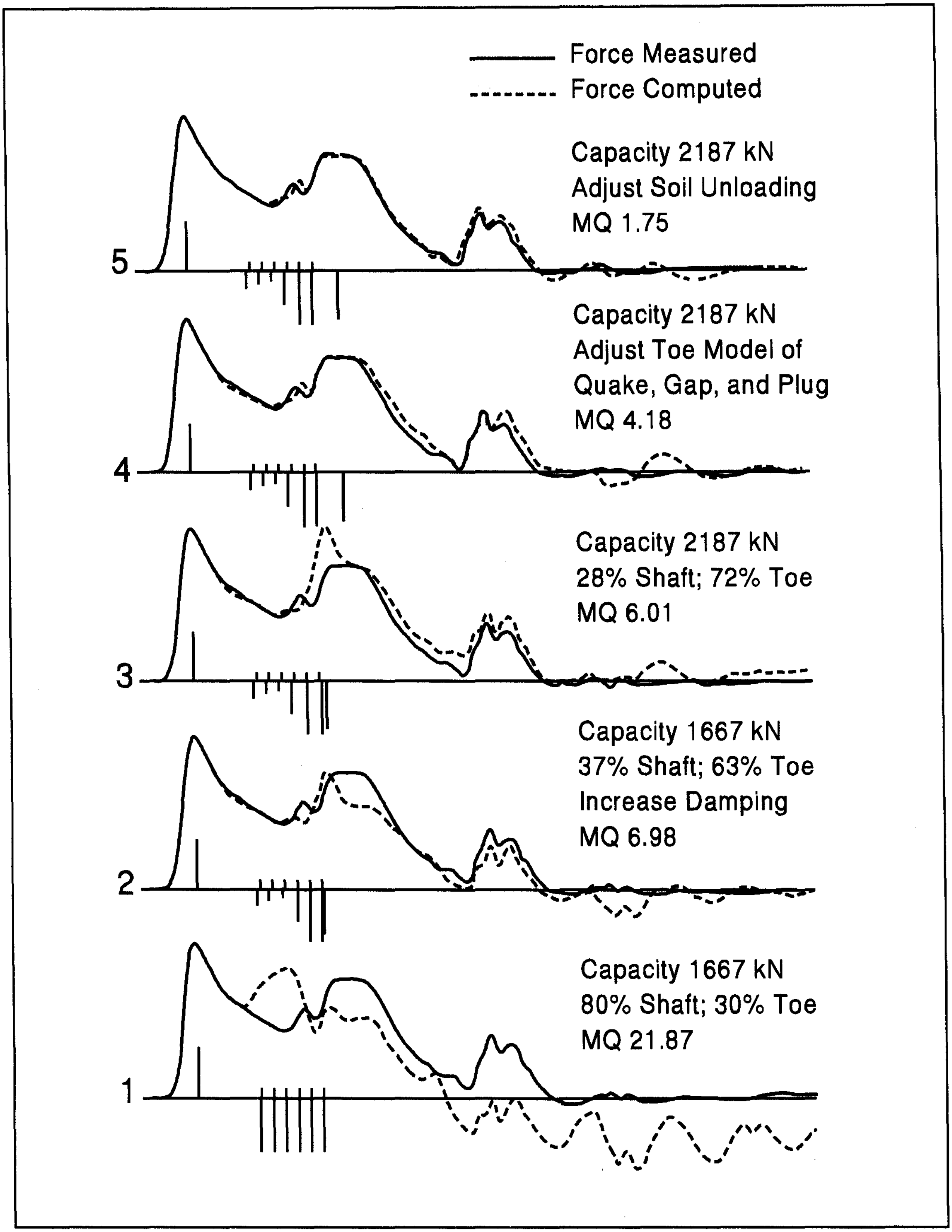


Figure 18.15 CAPWAP Iteration Matching Process (after Hannigan, 1990)

Goble Rausche Likins & Associates, Inc. 09-Nov-95

PEACH FREEWAY BRIDGE
Pile: PIER-2L Blow: 528
Collected: 01-Oct-92 CAPWAP(R) Ver. 1.994-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 2187.0; along Shaft 612.1; at Toe 1575.0 kN

Soil Sgmnt No.	Depth Below Gages m	Depth Below Grade m	Ru kN	Force in Pile at Ru kN	Sum of Ru kN	Unit Resist. w. Respect to Depth kN/m	Unit Resist. w. Respect to Area kN/m2	Smith Damping Factor s/m	Quake mm
				2187.0					
1	10.2	2.3	44.0	2143.0	44.0	21.58	15.20	.550	2.300
2	12.2	4.3	28.0	2115.0	72.0	13.71	9.65	.550	2.300
3	14.3	6.4	21.0	2094.0	93.0	10.29	7.25	.550	2.300
4	16.3	8.4	119.0	1975.0	212.0	58.35	41.09	.550	2.300
5	18.4	10.5	202.0	1773.0	414.0	99.03	69.74	.550	2.300
6	20.4	12.5	198.0	1575.0	612.1	97.08	68.36	.550	2.300
Average Skin Values			102.0			48.96	35.21	.550	2.300
Toe			1575.0				17499.63	.290	3.600

Soil Model Parameters/Extensions	Skin	Toe
Case Damping Factor	.488	.662

Figure 18.16 CAPWAP Final Results Table

Goble Rausche Likins & Associates, Inc. 09-Nov-95

PEACH FREEWAY BRIDGE
Pile: PIER-2L Blow: 528
Collected: 01-Oct-92 CAPWAP(R) Ver. 1.994-1

EXTREMA TABLE

Pile Sgmnt No.	Depth Below Gages m	max. Force kN	min. Force kN	max. Comp. Stress kN/cm2	max. Tension Stress kN/cm2	max. Trnsfd. Energy kN- m	max. Veloc. m/s	max. Displ. cm
1	1.0	3174.0	-72.5	18.848	-.430	32.31	4.4	1.737
2	2.0	3195.0	.0	18.973	.000	31.13	4.3	1.680
4	4.1	3214.2	.0	19.086	.000	29.83	4.3	1.560
6	6.1	3235.5	.0	19.213	.000	28.49	4.2	1.440
8	8.2	3273.7	.0	19.440	.000	27.20	4.2	1.320
10	10.2	3342.5	-60.9	19.848	-.362	25.92	4.1	1.200
12	12.2	3257.1	-83.6	19.342	-.497	23.56	4.0	1.080
14	14.3	3252.0	-140.1	19.311	-.832	21.75	3.9	.960
16	16.3	3379.0	-126.5	20.065	-.751	20.15	3.6	.850
18	18.4	3182.0	-49.7	18.895	-.295	17.11	3.4	.730
19	19.4	2718.8	.0	16.145	.000	14.02	3.3	.680
20	20.4	3005.1	.0	17.845	.000	11.97	3.0	.640
Absolute	16.3			20.065		(T=	24.7 ms)	
	14.3				-.832	(T=	41.2 ms)	

Figure 18.17 CAPWAP Stress Distribution Profile

18.6 USAGE OF DYNAMIC TESTING METHODS

Dynamic testing is specified in many ways, depending upon the information desired or purpose of the testing. For example, a number of test piles driven at preselected locations may be specified. In this application, the test piles are usually driven in advance of, or at the start of, production driving so that the information obtained can be used to establish driving criteria and/or pile order lengths for each substructure unit. Alternatively, or in addition to a test pile program, testing of production piles on a regular interval may be specified. Production pile testing is usually performed for quality assurance checks on hammer performance, driving stress compliance, pile integrity, and ultimate capacity. Lastly, dynamic testing can be used on projects where it was not specified to troubleshoot problems that arise during construction.

The number of piles that should be dynamically tested on the project depends upon the project size, variability of the subsurface conditions, the availability of static load test information, and the reasons for performing the dynamic tests. A higher percentage of piles should be tested, for example, where there are difficult subsurface conditions with an increased risk of pile damage, or where time dependent soil strength changes are being relied upon for a significant portion of the ultimate pile capacity.

On small projects, a minimum of two dynamic tests is recommended. On larger projects and small projects with anticipated installation difficulties or significant time dependent capacity issues, a greater number of piles should be tested. Dynamically testing one or two piles per substructure location is not unusual in these situations. Regardless of the project size, specifications should allow the engineer to adjust the number and locations of dynamically tested piles based on design or construction issues that arise.

Restrike dynamic tests should be performed whenever pile capacity is being evaluated by dynamic test methods. Restrikes are commonly specified 24 hours after initial driving. However, in fine grained soils, longer time periods are generally required for the full time dependent capacity changes to occur. Therefore, longer restrike times should be specified in these soil conditions whenever possible. On small projects, long restrike durations can present significant construction sequencing problems. Even so, at least one longer term restrike should be performed in these cases. The longer term restrike should be specified 2 to 6 days after the initial 24 hour restrike, depending upon the soil type. A warmed up hammer (from driving or restriking a non-test pile) should be used whenever restrike tests are performed.

When dynamic testing is performed by a consultant, the requirements for CAPWAP analyses should be specifically addressed in the dynamic testing specification. On larger projects, CAPWAP analyses are typically performed on 20 to 40% of the dynamic test data obtained from both initial driving and restrike dynamic tests. This percentage typically increases on smaller projects with only a few test piles, or on projects with highly variable subsurface conditions.

It is often contractually convenient to specify that the general contractor retain the services of the dynamic testing firm. However, this can create potential problems since the contractor is then responsible for the agency's quality assurance program. Some agencies have contracted directly with the dynamic testing firm to avoid this potential conflict and many large public owners have purchased the equipment and perform the tests with their own staff.

18.7 PRESENTATION AND INTERPRETATION OF DYNAMIC TESTING RESULTS

The results of dynamic pile tests should be summarized in a formal report that is sent to both the construction engineer and foundation designer. The construction engineer should understand the information available from the dynamic testing and its role in the project construction. As discussed in Chapter 9, numerous factors are considered in a pile foundation design. Therefore, the foundation designer should interpret the dynamic test results since many other factors; (downdrag, scour, uplift, lateral loading, settlement, *etc.*) may be involved in the overall design and construction requirements.

Construction personnel are often presented with dynamic testing results with minimal guidance on how to interpret or use the information. Therefore, it may be helpful to both construction personnel and foundation designers to familiarize themselves with the typical screen display and information available during a dynamic test. Figure 18.18 presents a typical Pile Driving Analyzer display for a 356 mm square prestressed concrete pile driven with a diesel hammer having a maximum rated energy of 89.6 kJ.

The main Pile Driving Analyzer input quantities are displayed in the upper left corner of the screen and include the pile length below gages, LE; the pile cross sectional area at the gages, AR; the pile elastic modulus, EM; the unit weight of the pile material, SP; the pile wave speed, WS; as well as the Case damping factor, JC. The lower left corner includes input quantities for display scales and transducer calibrations and is generally of little interest except to the test engineer. Construction personnel reviewing field results should, however, note the units indicator, UN, in this area of the screen. The force units are noted to be in "kN * 10" or kilonewtons times 10. This means any forces (but not stresses), capacity, or energy results displayed in the numerical results area must be multiplied by 10.

The screen is dominated by the graphical display of force (solid line) and velocity (dashed line) records versus time. This display will change for each hammer blow. The first vertical line represents time t_1 in the Case Method calculations and corresponds to the time of impact as the waves pass the gage location near the pile head. The second vertical line represents time t_2 in the Case Method calculations and corresponds to the time when the input waves have traveled to the pile toe and returned to the gage location or time 2L/C.

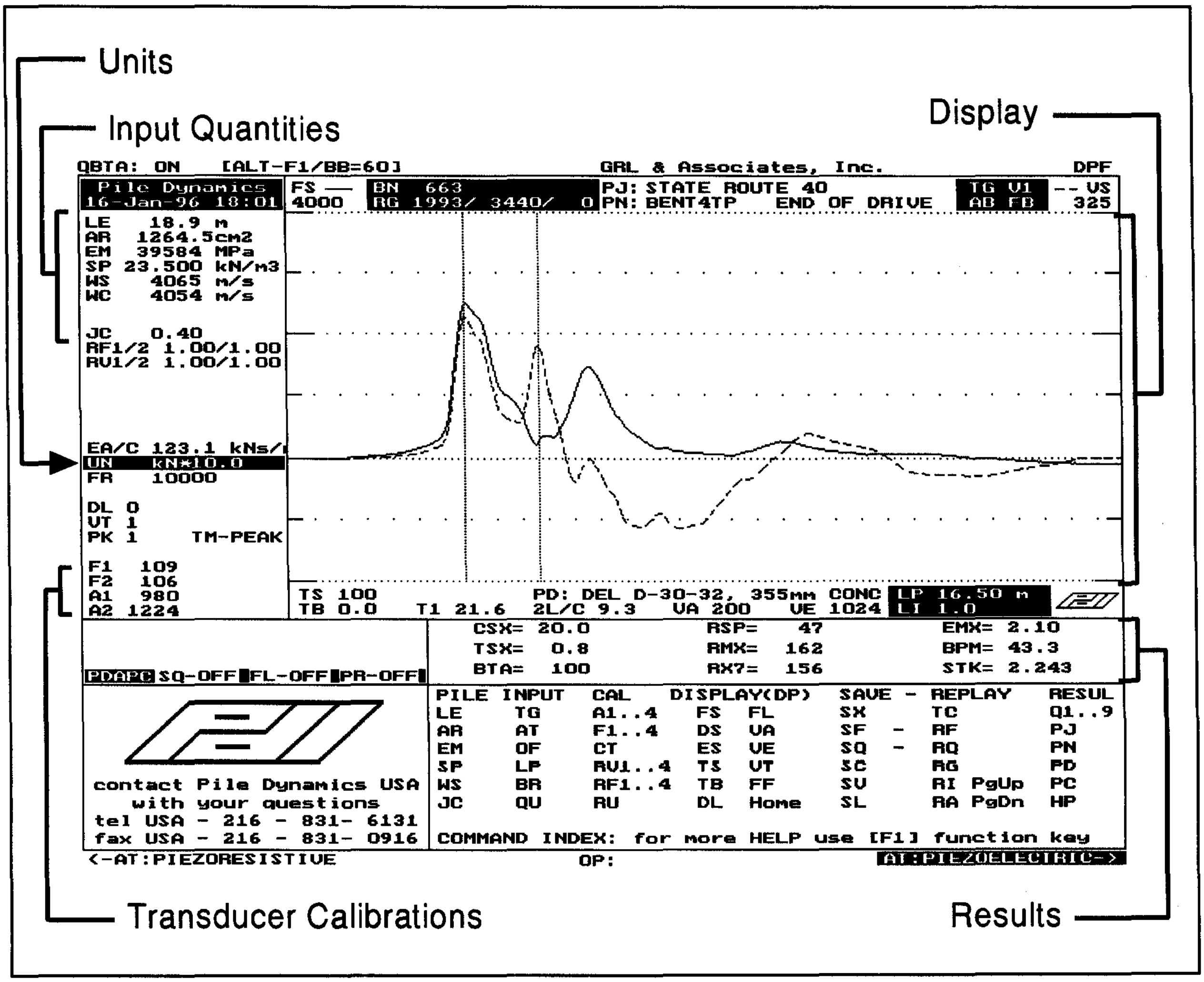


Figure 18.18 Typical Dynamic Test System Screen Display

An experienced test engineer can visually interpret these signals for data quality, soil resistance distribution and pile integrity. As discussed earlier, soil resistance forces cause a relative increase in the force wave and a corresponding relative decrease in the velocity wave. Therefore on a pile with a uniform cross section, the separation between the force and velocity records between times t_1 and t_2 indicates the shaft resistance. The magnitude of separation is also indicative of the magnitude of the soil resistance at that depth. Toe resistance is indicated by the separation between these records near and after time t_2.

The Pile Driving Analyzer searches for convergence between the force and velocity records beginning at the time of the sharp rise in the records prior to time t_1 and continuing for a time interval of 2L/C thereafter. If convergence between the force and velocity records occurs prior to the rise in the velocity record preceding time t_2, a cross sectional reduction or pile damage is indicated. The degree of convergence between the force and velocity records is expressed by the BTA integrity value as a percentage of the approximate reduced cross sectional area.

Numerical results from Case Method computations are identified by three letter codes displayed below the graphical records. In the example given in Figure 18.18, the first column of results provides information on the driving stresses and pile integrity. The compression stress at the pile head, CSX, is 20.0 MPa and the calculated tension stress, TSX, is 0.8 MPa. These calculated stress levels are below the recommended driving stress limits for a prestressed concrete pile given in Chapter 11. Pile integrity, BTA, is calculated as 100%, indicating that no damage is present.

The middle column of results includes computations for the standard Case Method capacity, RSP, and maximum Case Method capacity, RMX, both calculated with the input Case damping factor, JC, of 0.4. These results are 470 and 1620 kN respectively, when adjusted by the units multiplier. As noted earlier, a damping factor at least 0.2 higher is usually used with the maximum Case Method as compared to the standard Case Method. Therefore, the capacity using the RMX equation with a damping factor of 0.7 labeled RX7 was calculated and indicated a capacity of 1560 kN. From the force and velocity records in the example, the experienced test engineer would note that the resistance is delayed in time, based upon the separation between the force and velocity records occurring after time t_2. Therefore, the maximum Case Method equation should be used for capacity evaluation, and from the capacity results noted above, a Case Method capacity of 1560 kN would be chosen.

The final column of numerical results includes the transferred hammer energy, EMX, which is 21 kJ; the hammer operating speed in blows per minute, BPM, which is 43.3; and the calculated hammer stroke for the single acting diesel hammer of 2.24 meters.

Depending upon the hammer-pile combination, average transferred energies as a percentage of the rated energy range from about 25% for a diesel hammer on a concrete pile to 50% for an air hammer on a steel pile, Rausche *et al.* (1985a). Hence, the transferred energy of 21 kJ is 23% of the rated energy and is therefore slightly below average. The performance of a hammer and driving system can be evaluated from a driving system's rated transfer efficiency, which is defined as the energy transferred to the pile head divided by the manufacturer's rated hammer energy. Figure 18.19 presents transfer efficiencies for selected hammer and pile type combinations expressed as a percentile. In this graph, the average transfer efficiency for a given hammer-pile combination can be found by noting where that graph intersects the 50 percentile. Histograms of the transfer efficiencies for each of these hammer and pile types are also presented in Figure 18.20. The histograms may be useful in assessing drive system performance as they provided the distribution and standard deviation of drive system performance for a given hammer-pile combination at the end of drive condition.

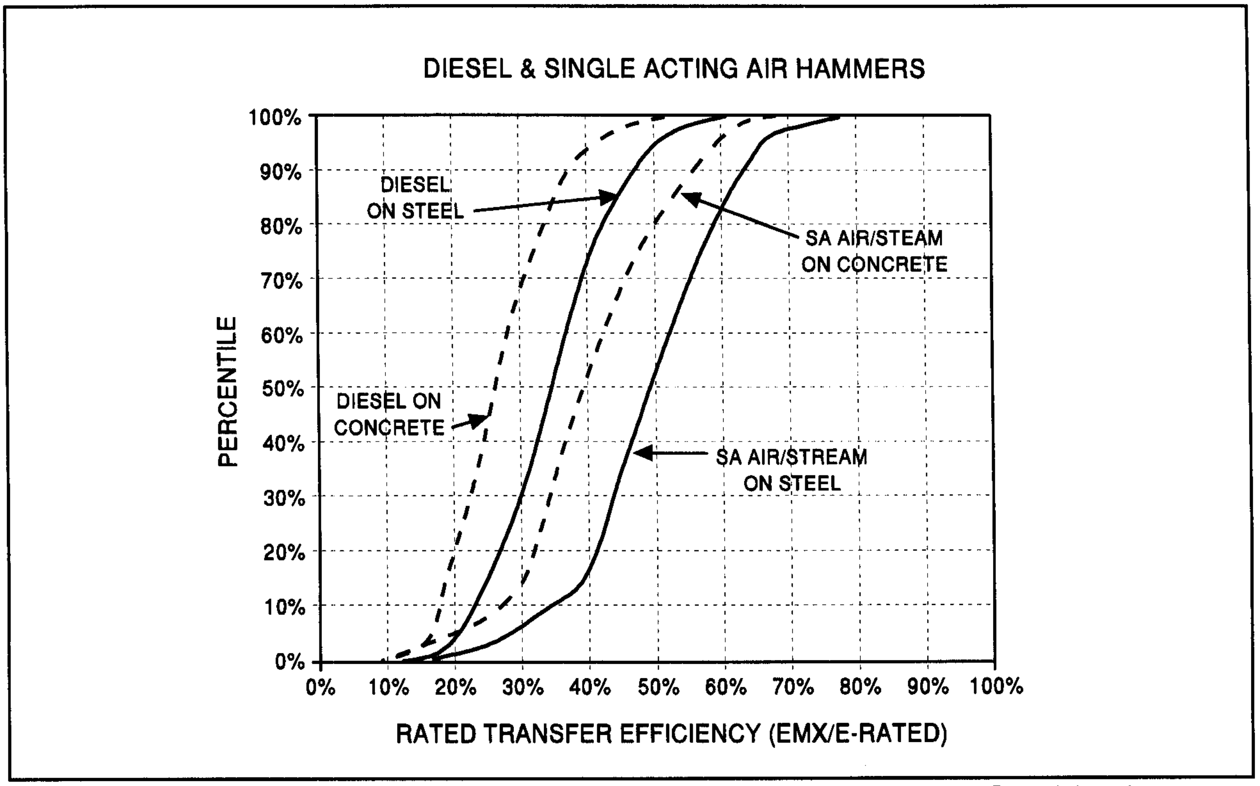
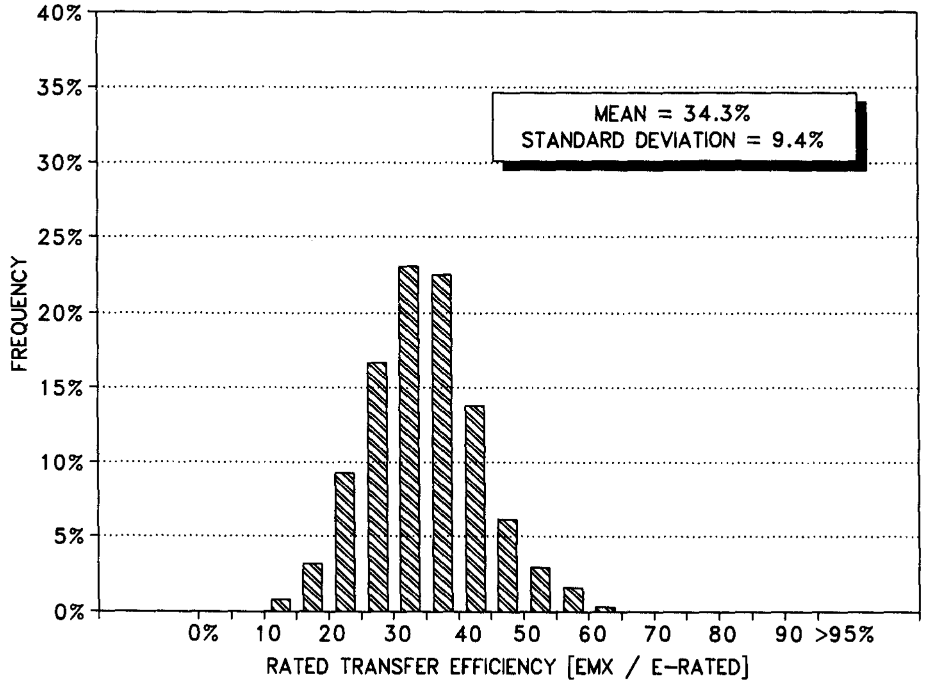
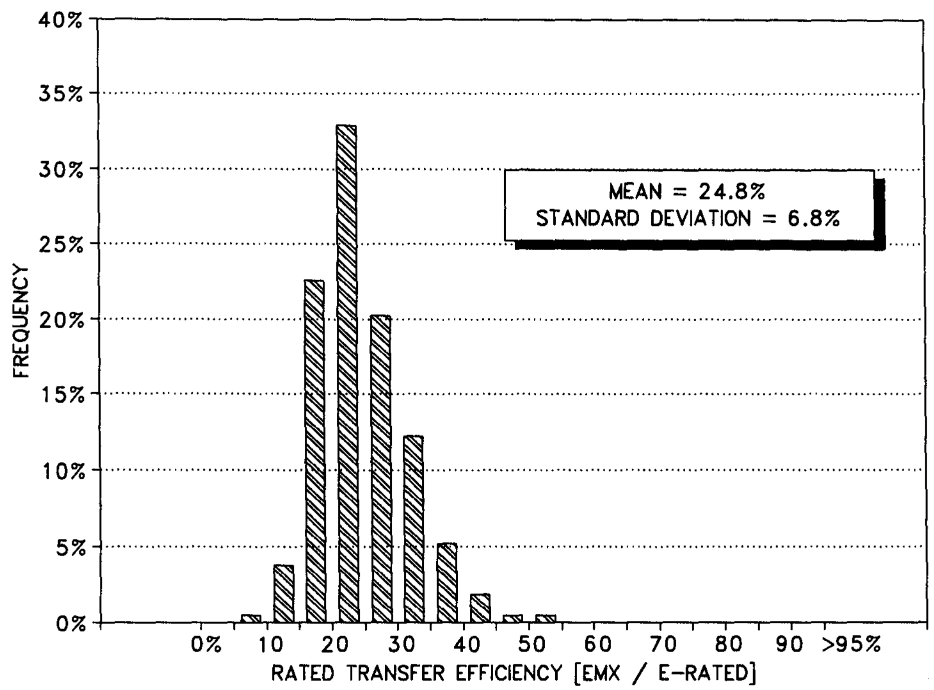


Figure 18.19 Transfer Efficiencies for Select Hammer and Pile Combinations



Diesel on Steel



Diesel on Concrete/Timber

Figure 18.20(a) Histograms of Transfer Efficiency for Diesel Hammers

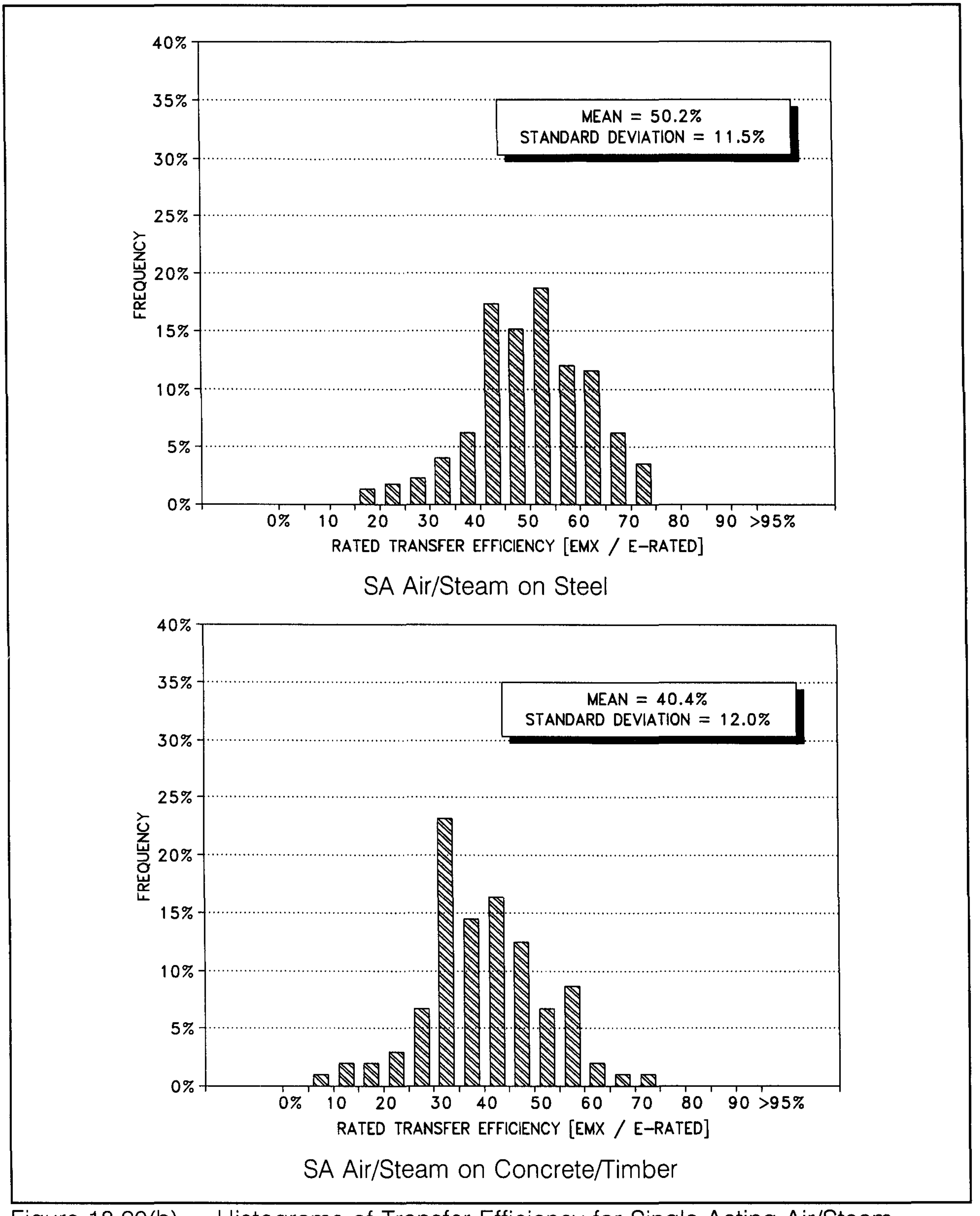


Figure 18.20(b) Histograms of Transfer Efficiency for Single Acting Air/Steam Hammers

In the field, construction personnel should check that the calculated driving stresses, CSX and TSX, are maintained within specification limits. Drive system performance indicated by the transferred energy, EMX, should be within a reasonable range of that predicted by wave equation analysis or recorded on previous tests at the site. If significant variations in energy are noted, the reasons for the discrepancy should be evaluated. The recorded hammer speed should be compared to the manufacturer's specifications. Capacity estimates should be compared with the required ultimate pile capacity. In soils with time dependent changes in capacity, this comparison should be based on restrike tests and not end-of-initial driving results.

A force and velocity record for a 406 mm x 13 mm wall closed end pipe pile is presented in Figure 18.21. As can be seen from the input properties, the pipe pile is 29.1 meters long below gages. A visual interpretation of the signal would indicate the pile has developed moderate shaft resistance over the lower portion of the pile with the majority of the pile capacity due to toe resistance. Note that an intermediate vertical line labeled D has also appeared between the two vertical lines corresponding to the pile head, t_1, and pile toe, t_2. Convergence between the force and velocity records before time 2L/C, as noted by the D line, indicates a pile impedance reduction or damage. A warning box has also appeared on the screen asking the test engineer if damage is occurring. For the example shown, damage was occurring at a depth of 14.9 meters below gages due to a welding problem at the pile splice.

In Figure 18.22, a force and velocity record for a HP 360 x 132 H-pile is presented. This record is typical of a pile driven to rock. Note the strong separation in the force and velocity records at time 2L/C (second vertical line). The compression stress at the gage location, CSX, is 211 MPa. This is within the recommended driving stress limit of 223 MPa for A-36 steel given in Chapter 11. The Pile Driving Analyzer can also compute an estimate of the compression stress at the pile toe, CSB. This quantity may be helpful in driving stress control for piles to rock. For the record shown, CSB is calculated to be 232 MPa which is above the recommended driving stress limit. Therefore, a slight reduction in hammer stroke at final driving may be necessary. The CSB quantity is an approximate value. A better assessment of the compression stresses at the pile toe could be gained from CAPWAP or wave equation analyses.

Additional insight into the pile and soil behavior during driving can be obtained by comparing the dynamic test numerical results versus pile penetration depth and corresponding driving resistance. Dynamic testing systems typically assign a sequential

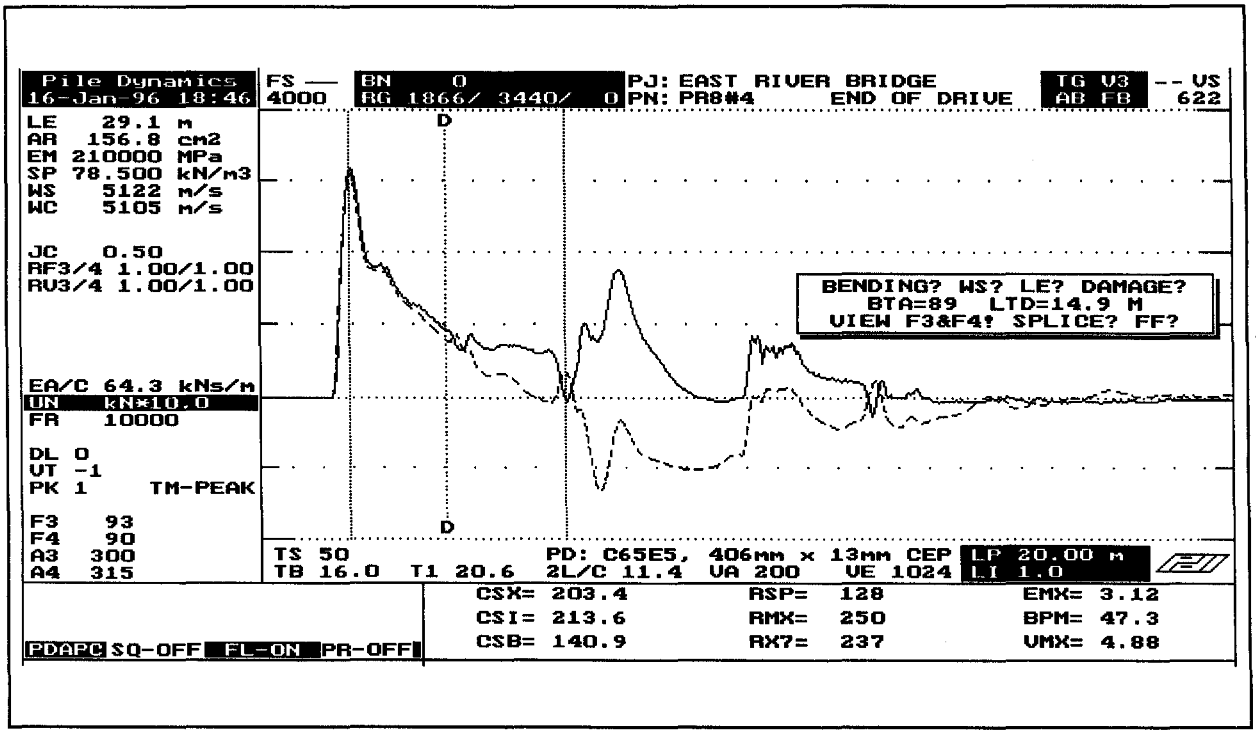


Figure 18.21 Force and Velocity Record for Damaged Pile

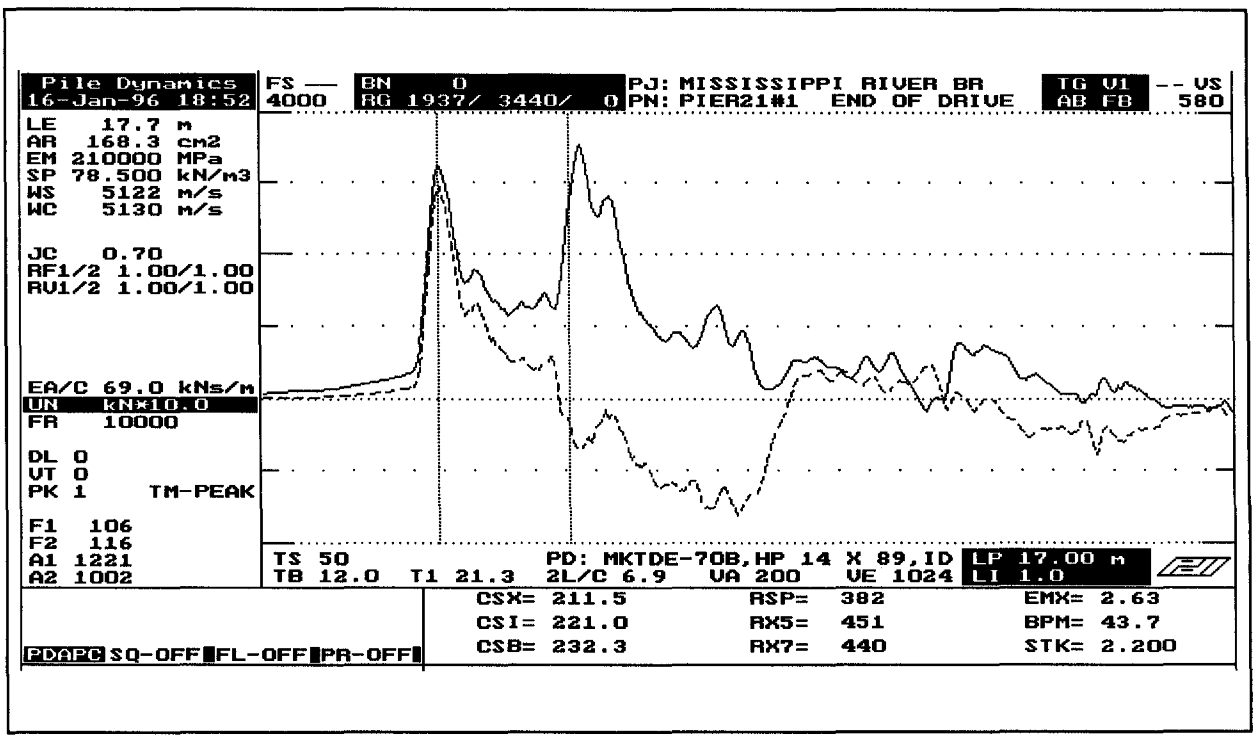


Figure 18.22 Force and Velocity Record for H-pile to Rock

blow number to each hammer blow. By comparing the pile driving records with these blow numbers, numerical and graphical summaries of the dynamic testing results versus pile penetration depth and driving resistance can be prepared. An example of a numerical summary of the dynamic testing results versus depth for a 610 mm octagonal concrete pile is presented in Table 18-3 with accompanying graphical results presented in Figure 18.23. These results can then easily be compared to project requirements by construction personnel.

TABLE 18-3 TYPICAL TABULAR PRESENTATION OF DYNAMIC TESTING RESULTS VERSUS DEPTH

```
Proj: PDAPLOT EXAMPLE                        Increment = (depth)          Pg1
Pile: PRESTRESSED CONCRETE                   FileName  = FGCTP5.MDF
Desc: DIESEL HAMMER                          BL#     2 to 1349    31-Oct-94
---------------------------------------------------------------------------
CSX: Max Measured C-Stress                   RMX: Capacity - RMX
TSX: Max Computed T-Stress                   BPM: Blows Per Minute
EMX: Max Transferred Energy                  STK: Stroke (O.E.Diesels)
---------------------------------------------------------------------------
```

BL# end	bl/m	depth m	TYPE	#Bls	CSX MPa	TSX MPa	EMX kN-m	RMX kN	BPM bl/min	STK m
11	18	8.00	AVG	6	8.48	2.05	18.85	285	56.1	1.439
34	23	9.00	AVG	9	12.04	2.92	27.03	584	49.6	1.696
64	30	10.00	AVG	23	14.67	3.63	27.40	1107	47.6	1.845
92	28	11.00	AVG	12	15.25	3.43	27.98	1210	47.5	1.846
123	31	12.00	AVG	15	15.62	4.08	28.03	1059	47.6	1.843
166	43	13.00	AVG	14	15.93	5.33	30.79	577	48.1	1.801
199	33	14.00	AVG	11	17.29	3.92	30.55	1143	47.1	1.882
225	26	15.00	AVG	8	17.62	4.96	33.44	875	47.5	1.850
245	20	16.00	AVG	7	18.29	6.23	33.91	586	47.4	1.857
266	21	17.00	AVG	6	18.03	6.47	33.83	340	48.1	1.802
289	23	18.00	AVG	8	18.09	6.65	35.90	284	48.0	1.808
322	33	19.00	AVG	16	18.25	6.14	36.47	529	47.1	1.880
370	48	20.00	AVG	24	20.80	5.75	39.59	1104	45.9	1.989
409	39	21.00	AVG	20	25.14	7.97	45.27	1137	45.3	2.045
453	44	22.00	AVG	22	26.33	7.91	45.81	1124	45.5	2.026
493	40	23.00	AVG	20	26.31	8.10	46.87	1003	45.6	2.014
568	75	24.00	AVG	30	15.65	1.52	33.30	1040	47.3	1.863
609	41	25.00	AVG	13	16.63	3.56	34.01	931	47.8	1.824
641	32	26.00	AVG	15	17.01	4.48	32.45	893	48.3	1.780
668	27	27.00	AVG	14	18.19	5.50	33.36	857	48.2	1.793
696	28	28.00	AVG	14	19.54	6.30	36.02	864	47.5	1.847
730	34	29.00	AVG	16	19.88	6.67	35.87	816	48.2	1.794
759	29	30.00	AVG	14	20.50	6.51	39.53	870	47.1	1.876
785	26	31.00	AVG	13	23.97	8.49	44.45	868	46.9	1.896
820	35	32.00	AVG	18	25.61	9.52	45.19	761	47.2	1.868
857	37	33.00	AVG	18	27.16	9.16	48.37	861	46.4	1.939
893	36	34.00	AVG	18	28.13	8.13	51.74	1056	45.8	1.994
933	40	35.00	AVG	20	27.48	9.15	49.39	868	46.5	1.933
971	38	36.00	AVG	19	26.64	9.04	45.58	767	47.3	1.867
1012	41	37.00	AVG	41	25.49	7.91	38.45	899	48.9	1.739
1050	38	38.00	AVG	38	24.55	9.00	36.31	796	49.4	1.699
1078	28	39.00	AVG	14	24.27	9.75	36.11	671	49.5	1.692
1106	28	40.00	AVG	10	22.62	8.52	35.04	596	49.6	1.689
1199	93	41.00	AVG	49	15.60	1.67	28.10	948	49.4	1.703
1235	144	41.25	AVG	18	17.01	0.60	31.84	1672	47.6	1.839
1275	160	41.50	AVG	18	16.88	1.16	30.38	2072	47.7	1.835
1329	216	41.75	AVG	26	19.70	1.50	40.17	2434	44.7	2.115
1349	200	41.85	AVG	9	33.30	6.11	67.31	2623	41.5	2.451

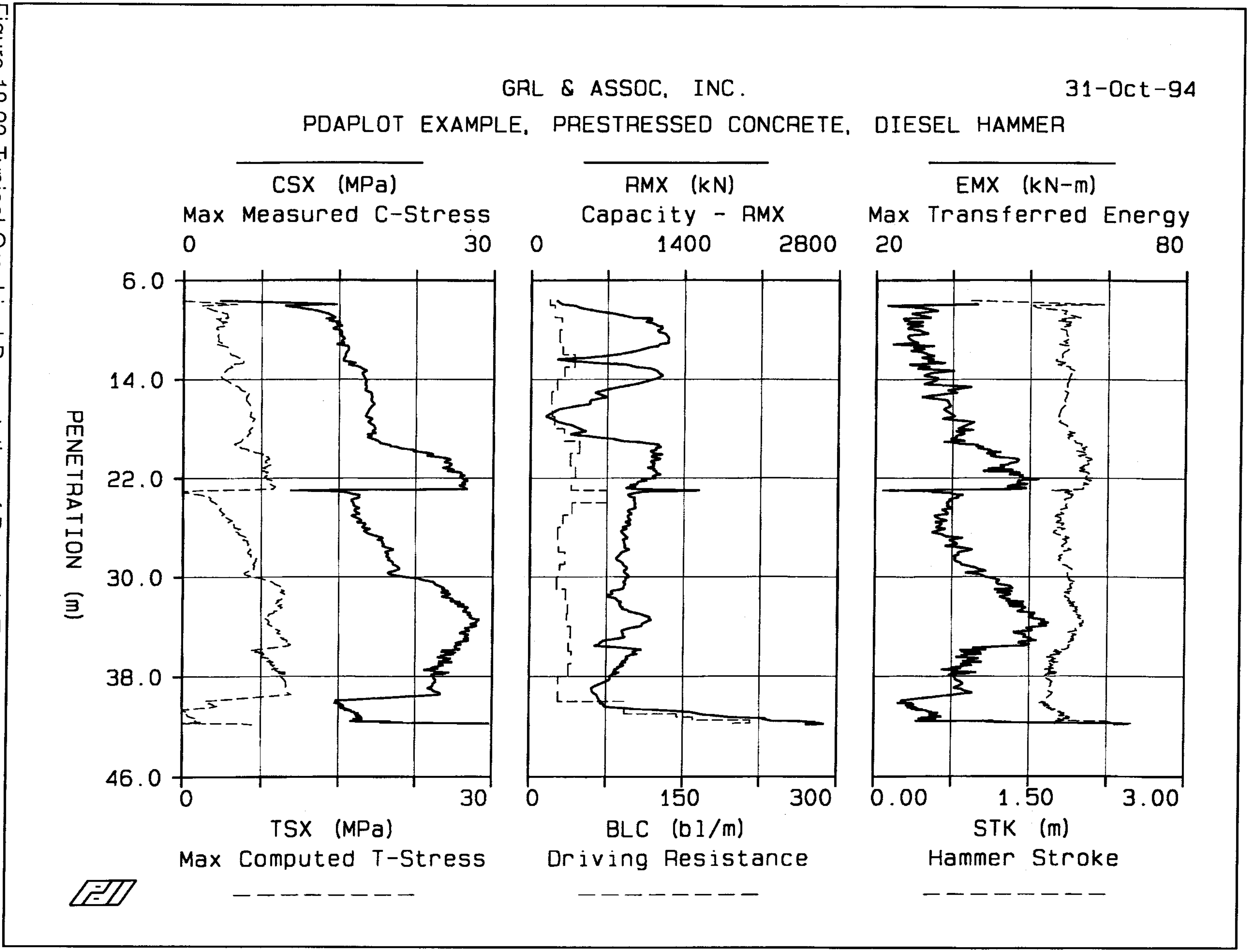


Figure 18.23 Typical Graphical Presentation of Dynamic Testing Results versus Depth

18.8 ADVANTAGES

Dynamic tests provide information on the complete pile installation process. Test results can be used to estimate pile capacity, to check hammer and drive system performance, to monitor driving stresses, and to assess pile structural integrity.

Many piles can be tested during initial driving or during restrike in one day. This makes dynamic testing an economical and quick testing method. Results are generally available immediately after each hammer blow.

On large projects, dynamic testing can be used to supplement static pile load tests or reduce the overall number of static tests to be performed. Since dynamic tests are more economical than static tests, additional coverage can also be obtained across a project at reduced costs. On small projects where static load tests may be difficult to justify economically, dynamic tests offer a viable construction control method.

Dynamic tests can provide information on pile capacity versus depth, capacity variations between locations, and capacity variations with time after installation through restrike tests. This information can be helpful in augmenting the foundation design, when available from design stage test pile programs, or in optimizing pile lengths when used early in construction test programs.

When used as a construction monitoring and quality control tool, dynamic testing can assist in early detection of pile installation problems such as poor hammer performance or high driving stresses. Test results can then facilitate the evaluation and solution of these installation problems.

On projects where dynamic testing was not specified and unexpected or erratic driving behavior or pile damage problems develop, dynamic testing offers a quick and economical method of troubleshooting.

Results from dynamic testing and analysis can be used for driving criteria development including wave equation input parameter selection and refinement of wave equation results as described in Section 17.6.6.

18.9 DISADVANTAGES

Dynamic testing to determine the ultimate static pile capacity requires that the driving system mobilize all the soil resistance acting on the pile. Shaft resistance can generally be mobilized at a fraction of the movement required to mobilize the toe resistance. However, when driving resistances approach 100 blows per quarter meter, the full soil resistance is difficult to mobilize at and near the pile toe. In these circumstances, dynamic test capacities tend to produce lower bound capacity estimates unless a larger hammer or higher stroke can be used to increase the pile net penetration per blow.

Dynamic testing estimates of static pile capacity indicate the capacity <u>at the time of testing</u>. Since increases and decreases in the pile capacity with time typically occur due to soil setup/relaxation, restrike tests after an appropriate waiting period are usually required for a better indication of long term pile capacity. This may require an additional move of the pile driving rig for restrike testing.

Larger diameter open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. This is particularly true if a soil plug does not form during driving. In these cases, limited toe bearing resistance develops during the dynamic test. However, under slower static loading conditions, these open section piles may develop a soil plug and therefore a higher pile capacity under static loading conditions. Interpretation of test results by experienced personnel is important in these situations.

18.10 CASE HISTORY

The following case history illustrates how dynamic pile testing and analysis was used on a small single span bridge constructed in a remote area. The subsurface exploration for the project found a 30 m deposit of moderately clean, medium dense to dense sands with SPT N values ranging from 17 to 50. Based upon these conditions, the foundation report recommended 324 mm O.D. closed end pipe piles be used for the bridge abutment foundations. The pipe piles had an estimated length of 12 m for an ultimate pile capacity of 1450 kN. The foundation report recommended wave equation analysis be used for construction control. Dynamic testing of one test pile at each abutment was also specified with the test pile information to be used by the engineer to provide the contractor pile order lengths.

The Case Method was used to evaluate pile capacity versus penetration depth during the test pile driving. More rigorous CAPWAP analyses were also performed on the dynamic test data to check the Case Method results at selected pile penetration depths. During initial driving at Abutment 1, the 324 mm pipe pile drove beyond the estimated pile penetration depth without developing the required ultimate capacity. The pile was driven to a depth of 23 m and had an end of drive ultimate capacity of 1044 kN. A restrike dynamic test performed one day after initial driving indicated the pile capacity increased slightly to 1089 kN.

While the test pile information from Abutment 1 was being evaluated, three additional test piles were driven at Abutment 2. First, dynamic testing of a 406 mm O.D. closed end pipe pile was performed to determine if a larger diameter pipe pile could develop the required ultimate pile capacity and, if so what pile penetration depth was necessary. The 406 mm was driven to a depth of 27 m and had an end of drive ultimate capacity of 989 kN. A one day restrike test on this pile indicated an ultimate capacity of 1245 kN. The 406 mm pile was driven deeper following the restrike test to a final penetration depth of 34 m. With the additional driving, the end of redrive ultimate capacity decreased to 1067 kN.

Approximately two weeks later, a 324 mm O.D. closed end pipe pile and a 356 mm diameter Monotube pile with a 7.6 m tapered lower section were driven at Abutment 2. The 324 mm pipe pile was driven to a penetration depth of 29 m with an end of drive ultimate capacity of 778 kN. The Monotube pile was driven to a depth of 13 m and had an end of drive ultimate capacity of 845 kN. One day restrike tests on both piles indicated a slight increase in ultimate capacity to 800 kN and 911 kN, respectively. During this same site visit, a 16 day restrike test was performed on the 406 mm pipe pile. The long term restrike ultimate capacity for the 406 mm pipe pile was 1778 kN.

The dynamic testing results from both abutments indicated that the desired ultimate pile capacity could not be obtained at or near the estimated pile penetration depth with the 324 mm pipe piles. However, two foundation solutions were indicated by the dynamic testing results. If a reduced ultimate capacity were chosen, the test results indicated a Monotube pile driven to a significantly shorter penetration depth could develop about the same ultimate pile capacity as could be developed by the 324 mm pipe piles. Alternatively, if the original ultimate pile capacity was desired, 406 mm pipe piles could be driven on the order of 28 m below grade.

Although not originally planned, two static load tests were performed to confirm the ultimate pile capacities that could be developed at the site. The 324 mm pipe and the 356 mm Monotube piles at Abutment 2 were selected for testing. The static load test results indicated the 324 mm pipe pile with a pile penetration depth of 29 m had an ultimate capacity of 1022 kN and the Monotube pile with a pile penetration depth of 13 m had an ultimate capacity of 978 kN. The dynamic test restrike capacities were in good agreement with these static load tests results particularly when the additional time between the dynamic restrike tests and static load tests is considered.

Based on the required pile lengths and capacities determined from the dynamic and static load testing, a cost evaluation of the foundation alternatives was performed. The cost analysis indicated that the Monotube piles would be the most economical pile foundation type. This case study illustrates how the routine application of dynamic testing on a small project helped facilitate the solution to an unexpected foundation problem.

18.11 LOW STRAIN INTEGRITY TESTING METHODS

The previous sections of the chapter described high strain dynamic testing methods and their applications. This section will discuss low strain integrity testing methods which can be used on driven pile foundations. These low strain methods may be used to evaluate pile length or integrity of piles with a high impedance (EA/C), such as solid concrete piles or concrete filled pipe piles. Additional details on low strain methods including equipment requirements and analysis of measurements may be found in ASTM D-5882 Standard Test Method for Low Strain Integrity Testing of Piles. Low strain integrity methods are not applicable to steel H-piles or unconcreted pipe piles.

18.11.1 Pulse Echo Method

Pulse echo pile testing consists of applying a low strain impact to the head of a pile, and monitoring the resulting pile head response. A small hand-held hammer (0.5 to 4 kg) is employed to deliver a clean impact to the pile head. An accelerometer, temporarily attached to the pile head, records pile head response as the generated low strain stress wave propagates down the pile length. Any changes in pile impedance (determined by the cross sectional area, the elastic modulus of the pile material and the stress wave speed of the pile material) along the pile shaft will generate a partial reflection of the downward travelling stress wave, thus identifying pile damage. At the pile toe a significant change in impedance would also occur, therefore allowing determination of pile length. The accelerometer records the magnitude and arrival time of the reflected waves. For undamaged piles, if a toe reflection is apparent, then it is possible to reasonably estimate an unknown pile length based upon an assumed wave speed.

The returning analog signals are captured and digitized by a portable high accuracy analog to digital data acquisition system. A display panel presents the record of one or more (averaged) blows for review and interpretation. Typically, the acceleration versus time data is integrated to a velocity versus time record to facilitate record evaluation.

This test method can also be used in cases where the pile length is known but the pile integrity is in question. In this application, a clearly indicated toe signal, together with a fairly steady velocity trace between the impact time and toe reflection, are signs of a sound pile. Strong velocity reflections before the expected toe signal are the result of changes in pile cross section and indicate pile damage.

Pulse echo integrity records of velocity versus time are presented in Figures 18.24 and 18.25 for two 305 mm square prestressed concrete piles. These records were obtained after a slope failure occurred during construction and the integrity of the driven piles was questioned. Figure 18.24 shows an amplified record for an undamaged 16.3 m long pile. Note the record drops below the origin at a depth 5 m which corresponds to soil resistance effects. A clear toe signal is apparent in the record at a depth of 16.3 m.

In Figure 18.25, an amplified pulse echo record on a nearby pile is presented. This pile has a clear indication of damage due to the slope movements based on the positive velocity reflection starting at a depth of 4 m.

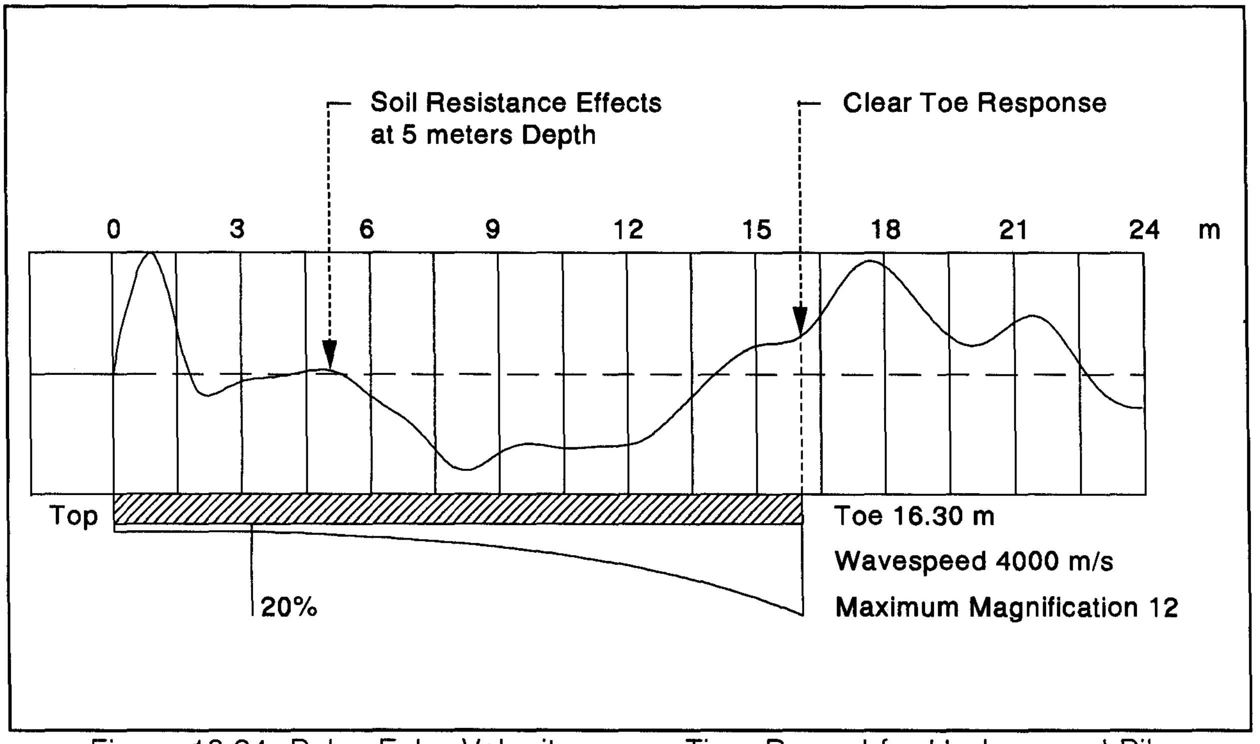


Figure 18.24 Pulse Echo Velocity versus Time Record for Undamaged Pile

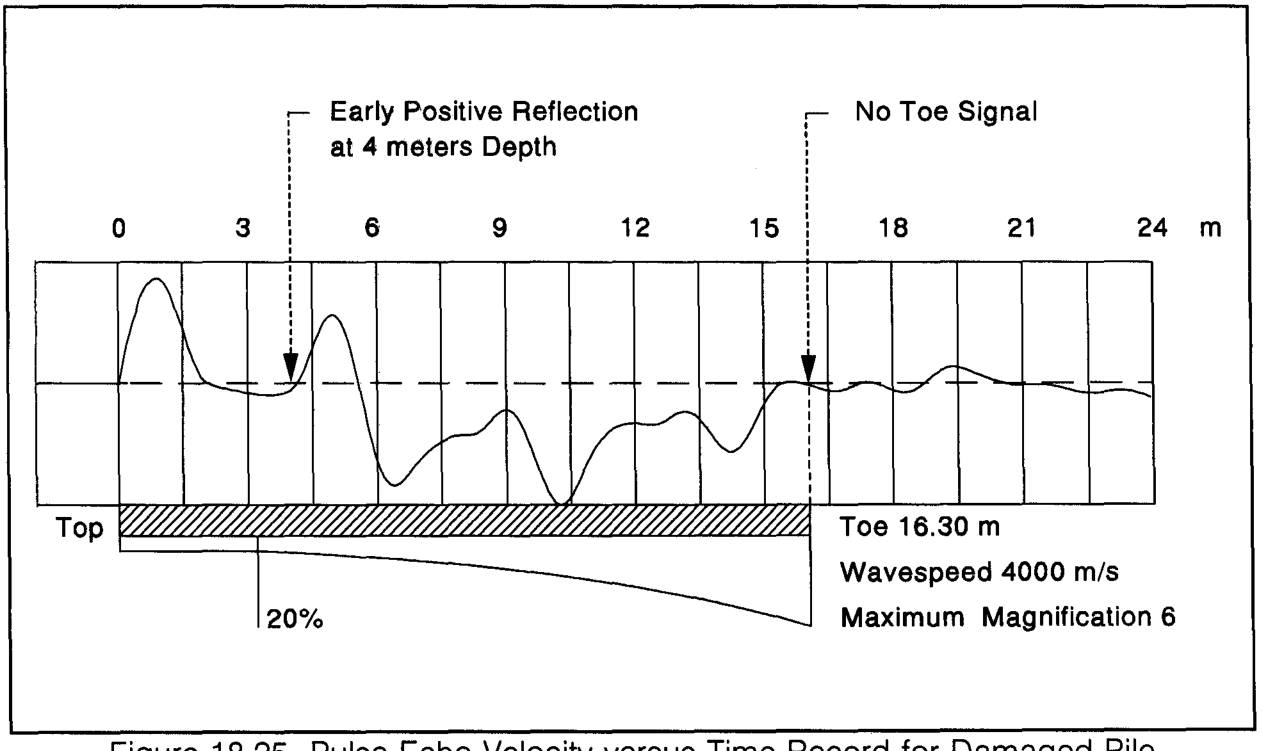


Figure 18.25 Pulse Echo Velocity versus Time Record for Damaged Pile

18.11.2 Transient Response Method (TRM)

In the TRM method, both the pile head response and the impact force are measured. A simple hand held hammer can adequately produce the frequency components necessary to test both well constructed and defective piles with TRM. The standard TRM plot of the ratio of the frequency velocity spectrum to force spectrum is called "mobility", and is an indication of the pile's velocity response to a particular excitation force at a certain frequency. Figure 18.26 depicts a typical response curve for a TRM test.

A mobility peak occurs at a frequency indicative of the time when the velocity changes due to a reflection from the pile toe or an intermediate impedance reduction or defect. Mobility peaks occurring at regular intervals are indicative of a dominant frequency Δf. The corresponding length to the pile toe or to a major defect at which the change in frequency occurs is calculated from:

$$L = C / 2\,\Delta f$$

Where: C = Wave speed.
L = Pile length.
Δf = Change in frequency.

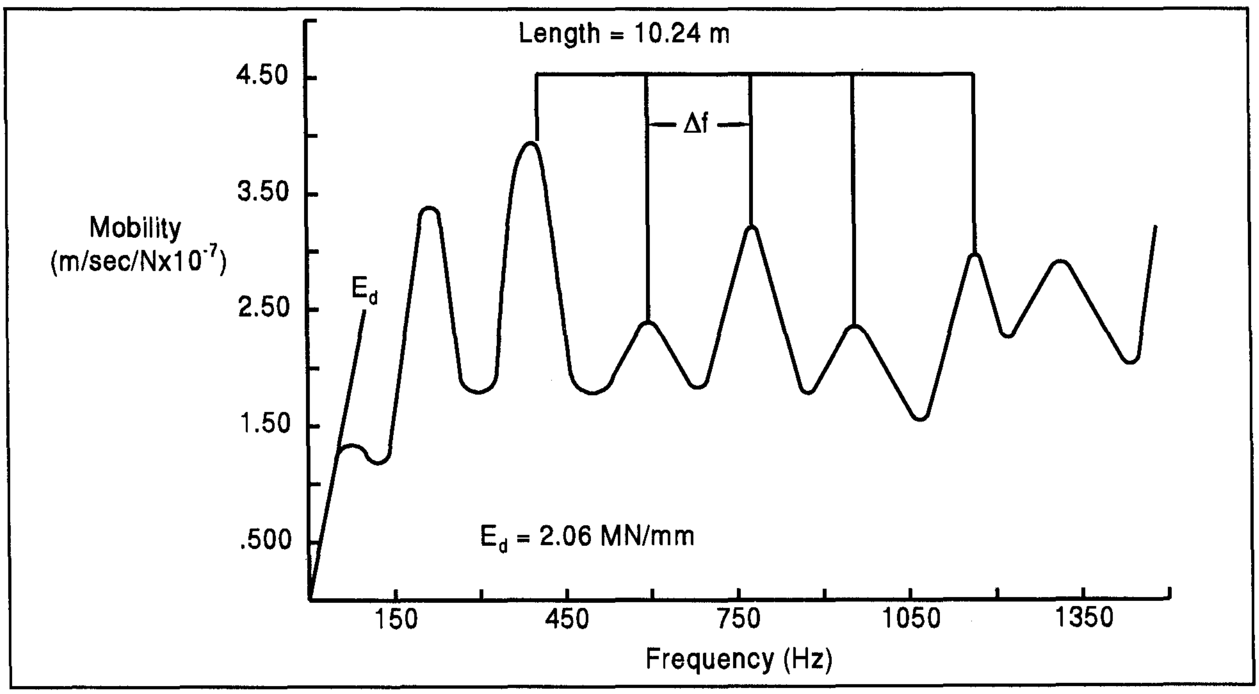


Figure 18.26 Typical Response Curve from a TRM Test

In practice, low frequency (*i.e.* near static) values are divided by the associated mobility yielding a so-called dynamic stiffness, E_d. This quantity increases with decreasing pile toe response. A low pile toe response is often the result of high soil resistance. A low pile toe resistance may also be caused by highly variable pile properties of internal pile damping, and is therefore only indirectly related to pile capacity. However, E_d is calculated, since it does provide a quantitative result for the evaluation of pile quality. Generally, higher stiffness values (for piles on the same site and of comparable length) indicate piles of higher strength (structural and soil) while lower stiffnesses indicate piles with potential defects or lower soil strength.

18.11.3 Low Strain Applications to Unknown Foundations

Design or construction records on many older bridges are not available. In some cases, the foundation supporting these structures is unknown and therefore the performance of these structures under extreme events such as scour is uncertain. A recent NCHRP research effort by Olson (1996) on the application of non-destructive testing methods to the evaluation of unknown foundations found the pulse echo and transient response methods fair to excellent in their ability to identify the depth of exposed piles and poor to good in their ability to determine the depth of footing or pile cap. These techniques are most applicable when the bridge is supported on a columnar substructure rather than a pier or abutment. Access to the bridge substructure is also generally required for implementation of these techniques. FHWA Geotechnical Guideline No. 16 (1998), provides a summary of this NCHRP study.

18.11.4 Limitations and Conclusions of Low Strain Methods

The low strain methods can typically be used for integrity or length assessments of pile foundations where the length to diameter ratio does not exceed about 30. For piles with severe cracks or manufactured mechanical joints, the stress wave will generally not be transmitted below the gap. Therefore, the pile integrity or length below this gap cannot be evaluated. Records from piles with multiple or varying (*i.e.* tapered piles) cross sectional areas can also be difficult to interpret. For piles of low impedance (H-piles and unfilled pipe piles) low strain methods are generally not suitable. When used for pile length determinations, the length information obtained from a toe signal (or a governing frequency) is only as accurate as the wave speed value assumed in the processing of the records. Wave speed variations of approximately 10% are not uncommon. Some defects can also have secondary and tertiary wave reflections. For example, if an impedance reduction occurs in the middle of the pile, then what may appear to be the pile toe response may actually be a secondary reflection of the mid-pile defect.

The additional force measurement obtained during TRM testing provides supplemental information of cross sectional changes near the pile head, *i.e.* within the distance corresponding to the impact signal. The minor additional expense of the force measurements is therefore worthwhile whenever questions arise as to the integrity of upper (1.5 m) pile portion.

Using low strain methods, many piles can be tested for integrity in a typical day. Therefore, low strain methods are a relatively economical test method and can provide valuable information when used in the proper application such as illustrated in the case study discussed in Section 18.11.1. Low strain testing has been used to assist in evaluating integrity questions on high impedance piles due to construction equipment or vessel impact, pulling on out of position piles, and storm damage.

REFERENCES

American Association of State Highway and Transportation Offices (AASHTO). Standard Method of Test for High Strain Dynamic Testing of Piles, AASHTO Designation T-298-33.

American Society for Testing and Materials, ASTM (1994). Annual Book of Standards, ASTM D-4945, Standard Test Method for High-Strain Dynamic Testing of Piles.

American Society for Testing and Materials, ASTM (1996). Annual Book of Standards, ASTM D-5882, Standard Test Method for Low-Strain Dynamic Testing of Piles.

Cheney, R.S. and Chassie, R.G. (1993). Soils and Foundations Workshop Manual. Second Edition, Publication No. FHWA HI-88-009, Federal Highway Administration, National Highway Institute, Washington, D.C., 353-362.

Davisson, M.T. (1972). High Capacity Piles. Proceedings of the Soil Mechanics Lecture Series on Innovations in Foundation Construction, American Society of Civil Engineers, ASCE, Illinois Section, 81-112.

Eiber, R.J. (1958). A Preliminary Laboratory Investigation of the Prediction of Static Pile Resistances in Sand. Master's Thesis, Department of Civil Engineering, Case Institute of Technology, Cleveland, OH.

Federal Highway Administration (1998). Geotechnical Guideline No. 16, Geotechnical Engineering Notebook, U.S. Department of Transportation, Washington D.C., 69.

Goble, G.G., Likins, G.E. and Rausche, F. (1975). Bearing Capacity of Piles from Dynamic Measurements. Final Report, Department of Civil Engineering, Case Western Reserve University, Cleveland, OH.

Goble, G.G. and Rausche, F. (1970). Pile Load Test by Impact Driving. Highway Research Record, Highway Research Board, No. 333, Washington, DC.

Goble, G.G., Rausche, F. and Likins, G.E. (1980). The Analysis of Pile Driving - A State-of-the-Art. Proceedings of the 1st International Seminar on the Application of Stress-Wave Theory on Piles, Stockholm, H. Bredenberg, Editor, A.A. Balkema Publishers, 131-161.

Hannigan, P.J. (1990). Dynamic Monitoring and Analysis of Pile Foundation Installations. Deep Foundations Institute Short Course Text, First Edition, 69.

Olson, L.D. (1996). Non-Destructive Testing of Unknown Subsurface Bridge Foundations - Results of NCHRP Project 21-5, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 42.

Pile Dynamics, Inc. (1996). Pile Driving Analyzer Manual; Model PAK, Cleveland, OH.

Rausche, F., Goble, G.G. and Likins, G.E. (1985b). Dynamic Determination of Pile Capacity. American Society of Civil Engineers, ASCE, Journal of the Geotechnical Engineering Division, Vol 111, No. 3, 367-383.

Rausche, F. and Goble, G.G. (1979). Determination of Pile Damage by Top Measurements. Behavior of Deep Foundations. American Society for Testing and Materials, ASTM STP 670, R. Lundgren, Editor, 500-506.

Rausche, F., Likins, G.E., Goble, G.G. and Miner, R. (1985a). The Performance of Pile Driving Systems. Main Report, U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development, Washington, D.C., Volumes I-IV.

Rausche, F., Moses, F., and Goble, G.G. (1972). Soil Resistance Predictions from Pile Dynamics. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM9.

Reiding, F.J., Middendorp, P., Schoenmaker, R.P., Middendorp, F.M. and Bielefeld, M.W. (1988). FPDS-2, A New Generation of Foundation Diagnostic Equipment, Proceeding of the 3rd International Conference on the Application of Stress Wave Theory to Piles, Ottawa, B.H. Fellenius, Editor, BiTech Publishers, 123-134.

19. STATIC PILE LOAD TESTING

Static load testing of piles is the most accurate method of determining load capacity. Depending upon the size of the project, static load tests may be performed either during the design stage or construction stage. Conventional load test types include the axial compression, axial tension and lateral load tests.

The purpose of this chapter is to provide an overview of static testing and its importance as well as to describe the basic test methods and interpretation techniques. For additional details on pile load testing, reference should be made to FHWA publication FHWA-SA-91-042, "Static Testing of Deep Foundation" by Kyfor *et al.* (1992) as well as the other publications listed at the end of this chapter.

19.1 REASONS FOR LOAD TESTING

1. Load tests are performed to develop information for use in the design and/or construction of a pile foundation.

2. Load tests are performed to confirm the suitability of the pile-soil system to support the pile design load with an appropriate factor of safety.

19.2 PREREQUISITES FOR LOAD TESTING

In order to adequately plan and implement a static load testing program, the following information should be obtained or developed.

1. A detailed subsurface exploration program at the test location. A load test is not a substitute for a subsurface exploration program.

2. Well defined subsurface stratigraphy including engineering properties of soil materials and identification of groundwater conditions.

3. Static pile capacity analyses to select pile type(s) and length(s) as well as to select appropriate location(s) for load test(s).

19.3 DEVELOPING A LOAD TEST PROGRAM

The goal of a load test program should be clearly established. A significantly different level of effort and instrumentation is required if the goal of the load test program is to confirm the ultimate pile capacity or if detailed load-transfer information is desired for design. The following items should be considered during the test program planning so that the program provides the desired information.

1. The capacity of the loading apparatus (reaction system and jack) should be specified so that the pile(s) may be loaded to plunging failure. A loading apparatus designed to load a pile to only twice the design load is usually insufficient to obtain plunging failure. Hence, the true factor of safety on the design load cannot be determined, and the full benefit from performing the static test is not realized.

2. Specifications should require use of a load cell and spherical bearing plate as well as dial gages with sufficient travel to allow accurate measurements of load and movement at the pile head. (Where possible, deformation measurements should also be made at the pile toe and at intermediate points to allow for an evaluation of shaft and toe bearing resistance).

3. The load test program should be supervised by a person experienced in this field of work.

4. A test pile installation record should be maintained with installation details appropriately noted. Too often, only the hammer model and driving resistance are recorded on a test pile log. Additional items such as hammer stroke (particularly at final driving), fuel setting, accurately determined final set, installation aids used and depths such as predrilling, driving times, stops for splicing, *etc.*, should be recorded.

5. Use of dynamic monitoring equipment on the load test pile is recommended for estimates of pile capacity at the time of driving, evaluation of drive system performance, calculation of driving stresses, and subsequent refinement of soil parameters for wave equation analysis.

19.4 ADVANTAGES OF STATIC LOAD TESTING

The advantages of performing static load tests are summarized below.

1. A static load test allows a more rational design. Confirmation of pile-soil capacity through static load testing is considerably more reliable than capacity estimates from static capacity analyses and dynamic formulas.

2. An improved knowledge of pile-soil behavior is obtained that may allow a reduction in pile lengths or an increase in the pile design load, either of which may result in potential savings in foundation costs.

3. With the improved knowledge of pile-soil behavior, a lower factor of safety may be used on the pile design load. A factor of safety of 2.0 is generally applied to design loads confirmed by load tests as compared to a factor of safety of 3.5 used on design loads in the Gates dynamic formula. Hence, a cost savings potential again exists.

4. The ultimate pile capacity determined from load testing allows confirmation that the design load may be adequately supported at the planned pile penetration depth.

Engineers are sometimes hesitant to recommend a static load test because of cost concerns or potential time delays in design or construction. While the cost of performing a static load test should be weighed against the anticipated benefits, cost alone should not be the determining factor. Cost benefits resulting from static load testing in both the design and construction stage were noted in the case studies presented in Chapter 2.

Delays to a project in the design or construction stage usually occur when the decision to perform static load tests is added late in the project. During a design stage program, delays can be minimized by determining early in the project whether a static load test program should be performed. In the construction stage, delays can be minimized by clearly specifying the number and locations of static load test to be performed as well as the time necessary for the engineer to review the results. In addition, the specifications should state that the static test must be performed prior to ordering pile lengths or commencing production driving. In this way, the test results are available to the design and construction engineer early in the project so that the maximum benefits can be obtained. At the same time the contractor is also aware of the test requirements and analysis duration and can schedule the project accordingly.

19.5 WHEN TO LOAD TEST

The following criteria from FHWA-SA-91-042 by Kyfor *et al.* (1992) summarizes the various conditions when pile load testing can be effectively utilized:

1. When the potential for substantial cost savings is readily apparent. This is often the case on large projects either involving friction piles (to prove that lengths can be reduced) or end bearing piles (to prove that the design load can be increased).

2. When a safe design load is in doubt due to limitations of an engineer's experience base or due to unusual site or project conditions.

3. When subsurface conditions vary considerably across the project, but can be delineated into zones of similar conditions. Static tests can then be performed in representative areas to delineate foundation variation.

4. When a significantly higher design load is contemplated relative to typical design loads and practice.

5. When time dependent changes in pile capacity are anticipated as a result of soil setup or relaxation.

6. When using precast concrete friction piles, it is important to determine pile cast lengths so that time consuming and costly splices can be avoided during construction.

7. When new, unproven pile types and/or installation procedures are utilized.

8. When existing piles will be reused to support a new structure with heavier design loads.

9. When a reliable assessment of pile uplift capacity or lateral behavior is important.

10. When, during construction, the estimated ultimate capacity using dynamic formulas or dynamic analysis methods differs from the estimated capacity at that depth determined by static analysis. For example, H-piles that "run" when driven into loose to medium dense sands and gravels.

Experience has also shown that load tests will typically confirm that pile lengths can be reduced at least 15 percent versus the lengths that would be required by the Engineering News formula on projects where piles are supported predominantly by shaft resistance. This 15 percent pile length reduction was used to establish the following rule of thumb formula to compute the total estimated pile length which the project must have to make the load test cost effective based purely on material savings alone.

$$\text{Total estimate pile length in meters on project} \geq \frac{\text{cost of load test}}{(0.15)\ (\text{cost/meter of pile})}$$

19.6 EFFECTIVE USE OF LOAD TESTS

19.6.1 Design Stage

The best information for design of a pile foundation is provided by the results of a load testing program conducted during the design phase. The number of static tests, types of piles to be tested, method of driving and test load requirements should be selected by the geotechnical and structural engineers responsible for design. A cooperative effort between the two is necessary. The following are the advantages of load testing during the design stage.

a. Allows load testing of several different pile types and lengths resulting in the design selection of the most economical pile foundation.

b. Confirm driveability to minimum penetration requirements and suitability of foundation capacity at estimated pile penetration depths.

c. Establishes preliminary driving criteria for production piles.

d. Pile driving information released to bidders should reduce their bid "contingency."

e. Reduces potential for claims related to pile driving problems.

f. Allows the results of load test program to be reflected in the final design and specifications.

19.6.2 Construction Stage

Load testing at the start of construction may be the only practical time for testing on smaller projects that can not justify the cost of a design stage program. Construction stage static tests are invaluable to confirm that the design loads are appropriate and that the pile installation procedure is satisfactory. Driving of test piles and load testing is frequently done to determine the pile order length at the beginning of construction. These results refine the estimated pile lengths shown on the plans and establish minimum pile penetration requirements.

19.7 COMPRESSION LOAD TESTS

Piles are most often tested in compression, but they can also be tested in tension or for lateral load capacity. Figure 19.1 illustrates the basic mechanism of performing a compression pile load test. This mechanism normally includes the following steps:

1. The pile is loaded incrementally from the pile head using some predetermined loading sequence, or it can be loaded at a continuous, constant rate.

2. Measurements of load, time, and movement at the pile head and at various points along the pile shaft are recorded during the test.

3. A load movement curve is plotted.

4. The failure load and the movement at the failure load are determined by one of the several methods of interpretation.

5. The movement is usually measured only at the pile head. However, the pile can be instrumented to determine movement anywhere along the pile. Telltales (solid rods protected by tubes) shown in Figure 19.1 or strain gages may be used to obtain this information.

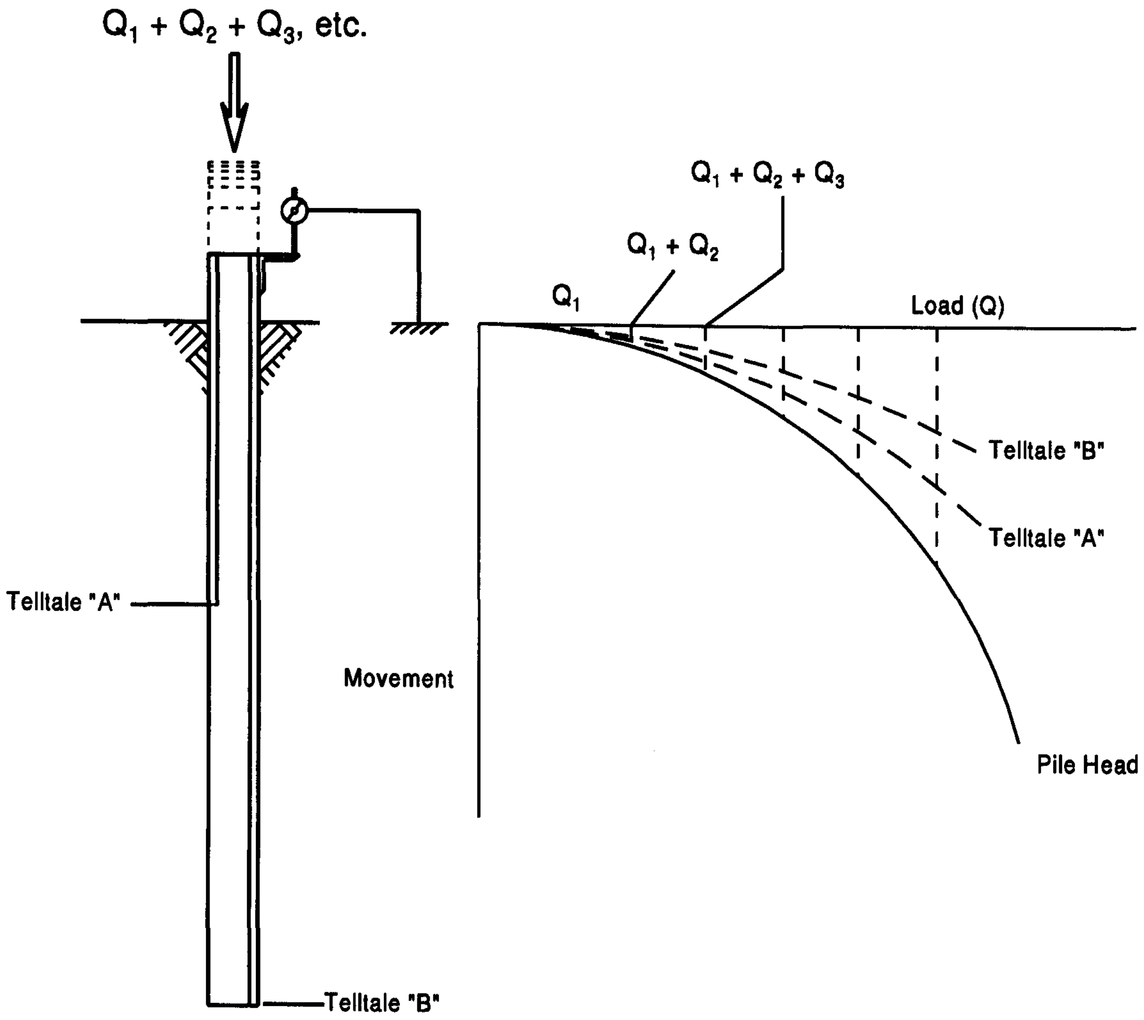


Figure 19.1 Basic Mechanism of a Pile Load Test

19.7.1 Compression Test Equipment

ASTM D-1143 recommends several alternative systems for (1) applying compressive load to the pile, and (2) measuring movements. Most often, compressive loads are applied by hydraulically jacking against a beam that is anchored by piles or ground anchors, or by jacking against a weighted platform. The primary means of measuring the load applied to the pile should be with a calibrated load cell. The jack load should also be recorded from a calibrated pressure gage. To minimize eccentricities in the applied load, a spherical bearing plate should be included in the load application arrangement.

Axial pile head movements are usually measured by dial gages or LVDT's that measure movement between the pile head and an independently supported reference beam. ASTM requires the dial gages or LVDT's have a minimum of 50 mm of travel and a precision of at least 0.25 mm. It is preferable to have gages with a minimum travel of 75 mm (particularly for long piles with large elastic deformations under load) and with a precision of 0.025 mm. A minimum of two dial gages or LVDT's mounted equidistant from the center of the pile and diametrically opposite should be used. Two backup systems consisting of a scale, mirror, and wire system should be provided with a scale precision of 0.25 mm. The backup systems should also be mounted on diametrically opposite pile faces. Both the reference beams and backup wire systems are to be independently supported with a clear distance of not less than 2.5 m between supports and the test pile. A remote backup system consisting of a survey level should also be used in case reference beams or wire systems are disturbed during the test.

ASTM specifies that the clear distance between a test pile and reaction piles be at least 5 times the maximum diameter of the reaction pile or test pile (whichever has the greater diameter if not the same pile type) but not less than 2 meters. If a weighted platform is used, ASTM requires the clear distance between cribbing supporting the weighted platform and the test pile exceed 1.5 meters.

A schematic of a typical compression load test setup is presented in Figure 19.2. A photograph of a typical compression load test arrangement using reaction piles is presented in Figure 19.3 and a weighted platform arrangement is shown in Figure 19.4. Additional details on load application as well as pile head load and movement measurements may be found in ASTM D-1143 as well as in FHWA-SA-91-042 by Kyfor *et al.* (1992).

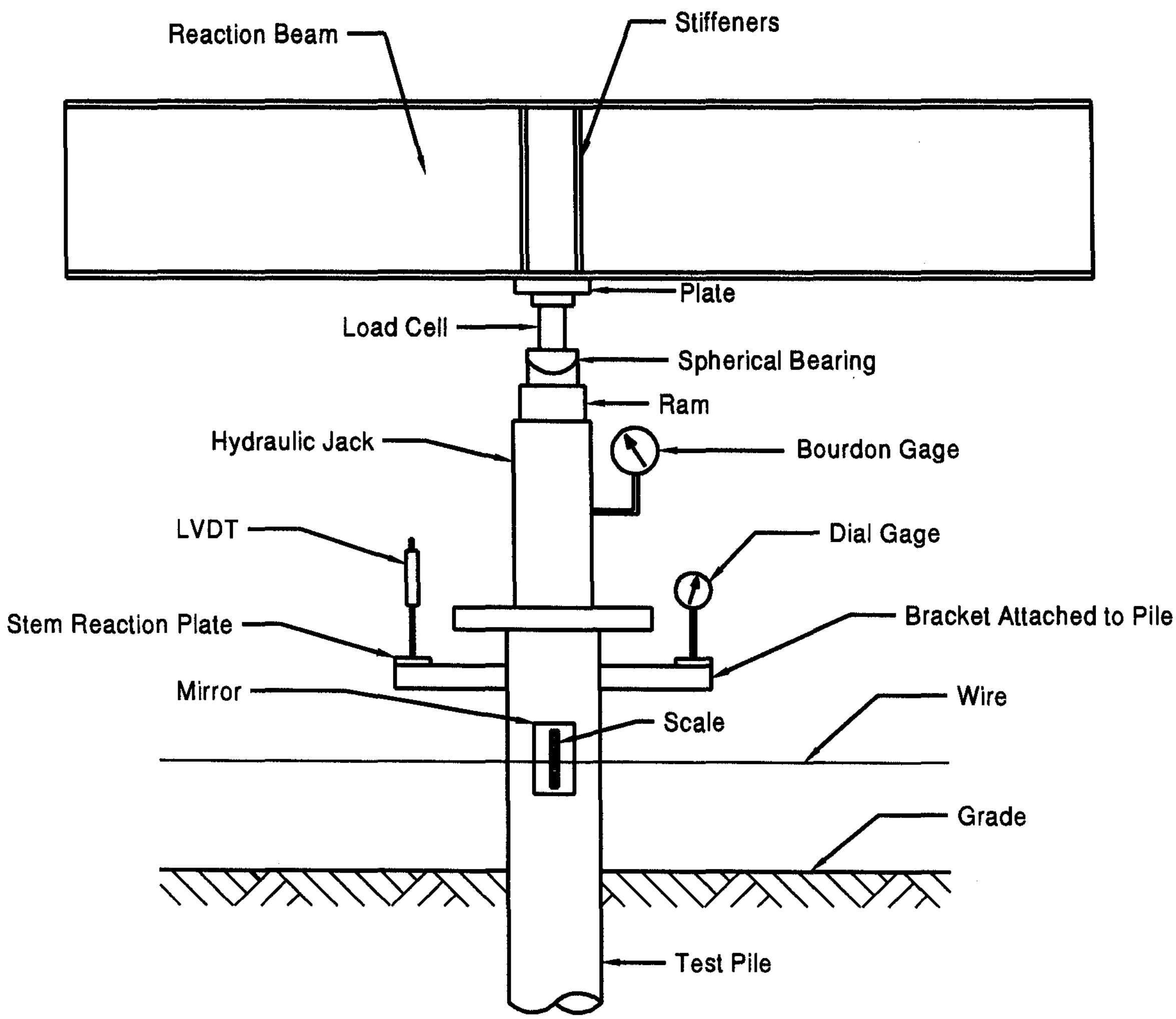


Figure 19.2 Typical Arrangement for Applying Load in an Axial Compressive Test (Kyfor *et al.* 1992)

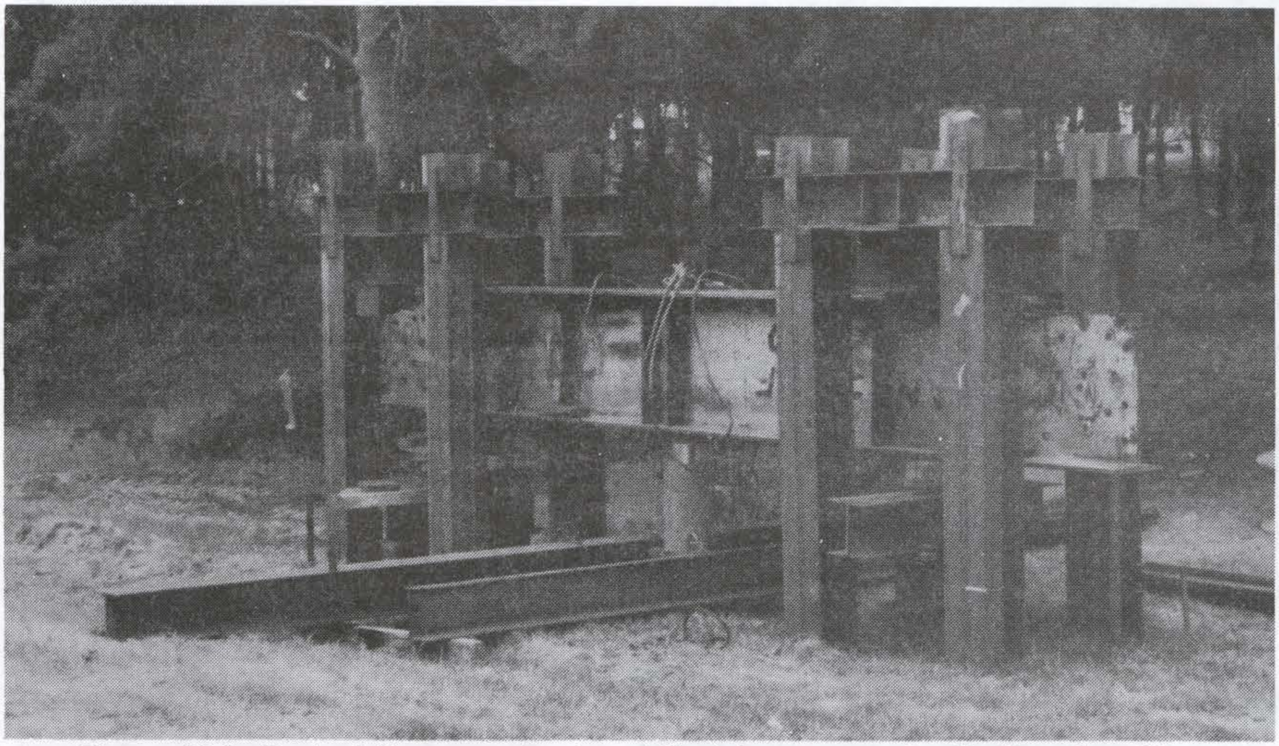


Figure 19.3 Typical Compression Load Test Arrangement with Reaction Piles

19.7.2 Recommended Compression Test Loading Method

It is extremely important that standardized load testing procedures are followed. Several loading procedures are detailed in ASTM D-1143, Standard Test Method for Piles Under Static Axial Compressive Load. The quick load test method is recommended. This method replaces traditional methods where each load increment was held for extended periods of time. The quick test method requires that load be applied in increments of 10 to 15% of the pile design load with a constant time interval of 2½ minutes or as otherwise specified between load increments. Readings of time, load, and gross movement are to be recorded immediately before and after the addition of each load increment. This procedure is to continue until continuous jacking is required to maintain the test load or the capacity of the loading apparatus is reached, whichever occurs first. Upon reaching and holding the maximum load for 5 minutes, the pile is unloaded in four equal load decrements which are each held for 5 minutes. Readings of time, load, and gross movement are once again recorded immediately after, 2½ minutes after, and 5 minutes after each load reduction, including the zero load.

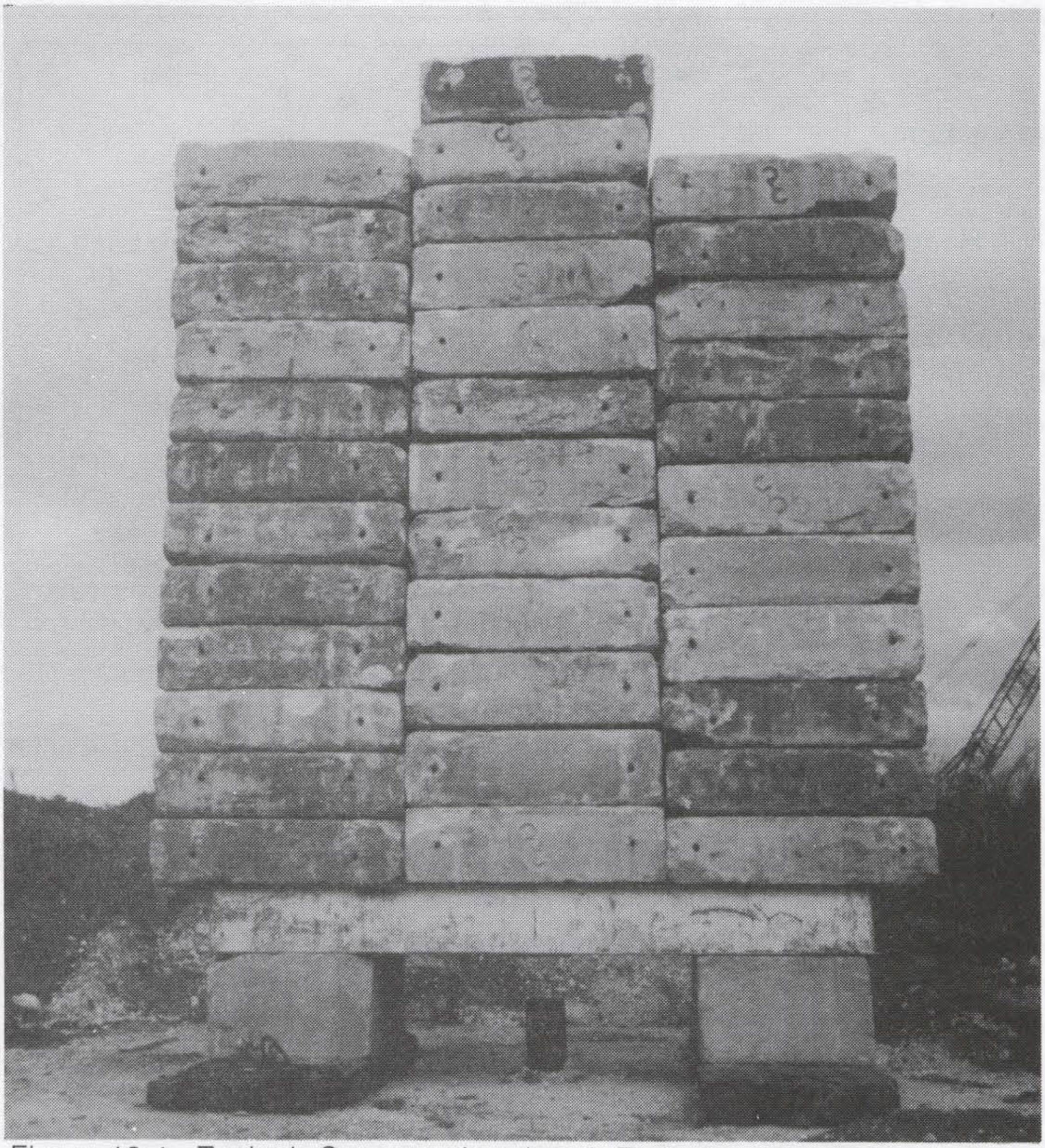


Figure 19.4 Typical Compression Load Test Arrangement using a Weighted Platform

19.7.3 Presentation and Interpretation of Compression Test Results

The results of load tests should be presented in a report conforming to the requirements of ASTM D-1143. A load-movement curve similar to the one shown in Figure 19.5 should be plotted for interpretation of test results.

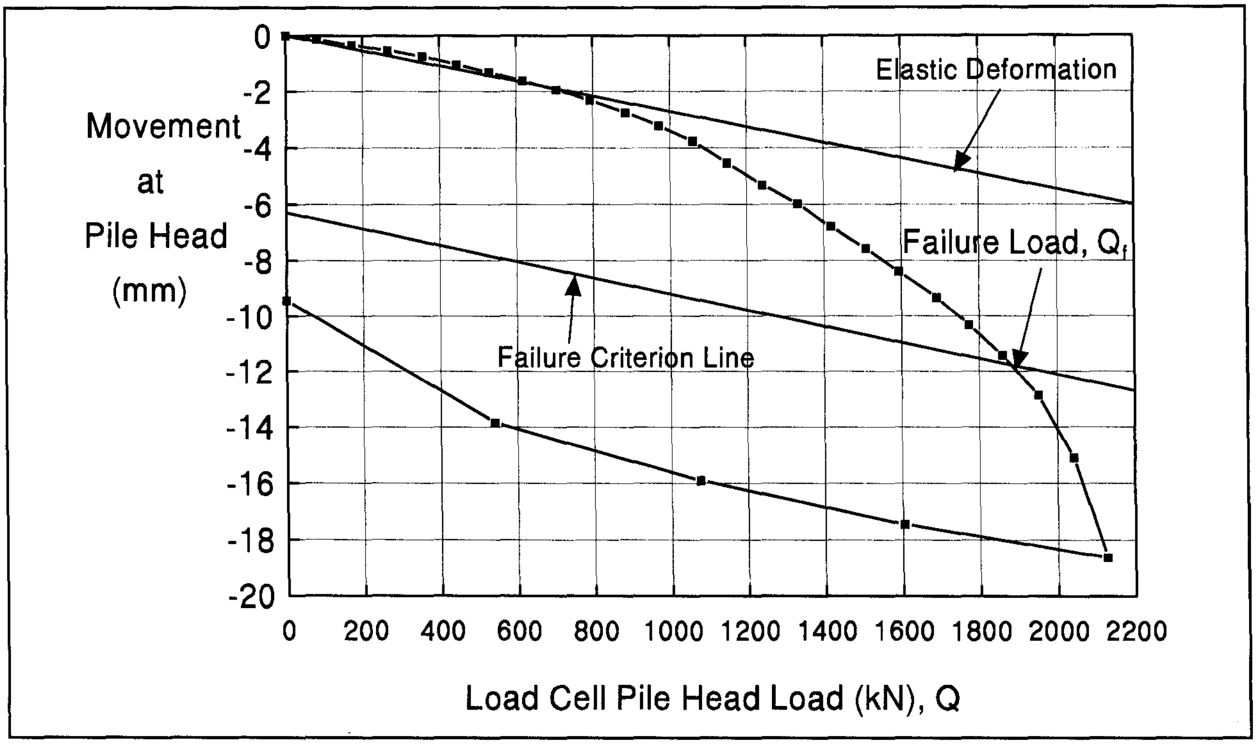


Figure 19.5 Presentation of Typical Static Pile Load-Movement Results

The literature abounds with different methods of defining the failure load from static load tests. Methods of interpretation based on maximum allowable gross movements, which do not take into account the elastic deformation of the pile shaft, are not recommended. These methods overestimate the allowable capacities of short piles and underestimate the allowable capacities of long piles. The methods which account for elastic deformation and are based on failure criterion provide a better understanding of pile performance and provide more accurate results.

AASHTO (1992) and FHWA SA-91-042, Kyfor *et al.* (1992) recommend compression test results be evaluated using an offset limit method as proposed by Davisson (1972). This method is described in the following section and is applicable for load tests in which the increment of load is held for not more than 1 hour.

19.7.4 Plotting the Load-Movement Curve

Figure 19.5 shows the load-movement curve from a pile load test. To facilitate the interpretation of the test results, the scales for the loads and movements are selected so that the line representing the elastic deformation Δ of the pile is inclined at an angle of about 20° from the load axis. The elastic deformation Δ is computed from:

$$\Delta = \frac{QL}{AE}$$

Where: Δ = Elastic deformation in mm.
Q = Test load in kN.
L = Pile length in mm.
A = Cross sectional area of the pile in m^2.
E = Modulus of elasticity of the pile material in kPa.

19.7.5 Determination of the Ultimate Load

The ultimate or failure load Q_f of a pile is that load which produces a movement of the pile head equal to:

$$s_f = \Delta + (4.0 + 0.008b)$$

Where: b = Pile diameter in mm.

A failure criterion line parallel to the elastic deformation line is plotted as shown in Figure 19.5. The point at which the observed load-movement curve intersects the failure criterion is by definition the failure load. If the load-movement curve does not intersect the failure criterion line, the pile has an ultimate capacity in excess of the maximum applied test load.

For large diameter piles (diameter greater than 610 mm), additional pile toe movement is necessary to develop the toe resistance. Therefore for large diameter piles, FHWA SA-91-042, Kyfor *et al.* (1992) recommends the failure load be determined from:

$$s_f = \Delta + \frac{b}{30}$$

19.7.6 Determination of the Allowable Load

The allowable design load is usually determined by dividing the ultimate load, Q_f, by a suitable factor of safety. A factor of safety of 2.0 is recommended in AASHTO code (1992) and is often used. However, larger factors of safety may be appropriate under the following conditions:

a. Where soil conditions are highly variable.
b. Where a limited number of load tests are specified.
c. For friction piles in clay, where group settlement may control the allowable load.
d. Where the total movement that can be tolerated by the structure is exceeded.
e. For piles installed by means other than impact driving, such as vibratory driving or jetting.

19.7.7 Load Transfer Evaluations

Kyfor *et al.* (1992) provides a method for evaluation of the soil resistance distribution from telltales embedded in a load test pile. The average load in the pile, Q_{avg}, between two measuring points can be determined as follows:

$$Q_{avg} = A \ E \ \frac{R_1 - R_2}{\Delta L}$$

Where:
- ΔL = Length of pile between two measuring points under no load condition.
- A = Cross sectional area of the pile.
- E = Modulus of elasticity of the pile.
- R_1 = Deflection readings at upper of two measuring points.
- R_2 = Deflection readings at lower of two measuring points.

If the R_1 and R_2 readings correspond to the pile head and the pile toe respectively, then an estimate of the shaft and toe resistances may be computed. For a pile with an assumed constant soil resistance distribution (uniform), Fellenius (1990) states that an estimate of the toe resistance, R_t, can be computed from the applied pile head load, Q_h. The applied pile head load, Q_h, is chosen as close to the failure load as possible.

$$R_t = 2Q_{avg} - Q_h$$

For a pile with an assumed linearly increasing soil resistance distribution (triangular), the estimated toe resistance may be calculated using:

$$R_t = 3Q_{avg} - 2Q_h$$

The estimated shaft resistance can then be calculated from the applied pile head load minus the toe resistance.

During driving, residual loads can be locked into a pile that does not completely rebound after a hammer blow (*i.e.* return to a condition of zero stress along its entire length). This is particularly true for flexible piles, piles with large frictional resistances, and piles with large toe quakes. Load transfer evaluations using telltale measurements described above assume that no residual loads are locked in the pile during driving. Therefore, the load distribution calculated from the above equations would not include residual loads. If measuring points R_1 and R_2 correspond to the pile head and pile toe of a pile that has locked-in residual loads, the calculated average pile load would also include the residual loads. This would result in a lower toe resistance being calculated than actually exists as depicted in Figure 19.6. Additional details on telltale load transfer evaluation, including residual load considerations, may be found in Fellenius (1990).

When detailed load transfer data is desired, telltale measurements alone are insufficient, since residual loads can not be directly accounted for. Dunnicliff (1988) suggests that weldable vibrating wire strain gages be used on steel piles and sister bars with vibrating wire strain gages be embedded in concrete piles for detailed load transfer evaluations. A geotechnical instrumentation specialist should be used to select the appropriate instrumentation to withstand pile handling and installation, to determine the redundancy required in the instrumentation system, to determine the appropriate data acquisition system, and to reduce and report the data acquired from the instrumentation program.

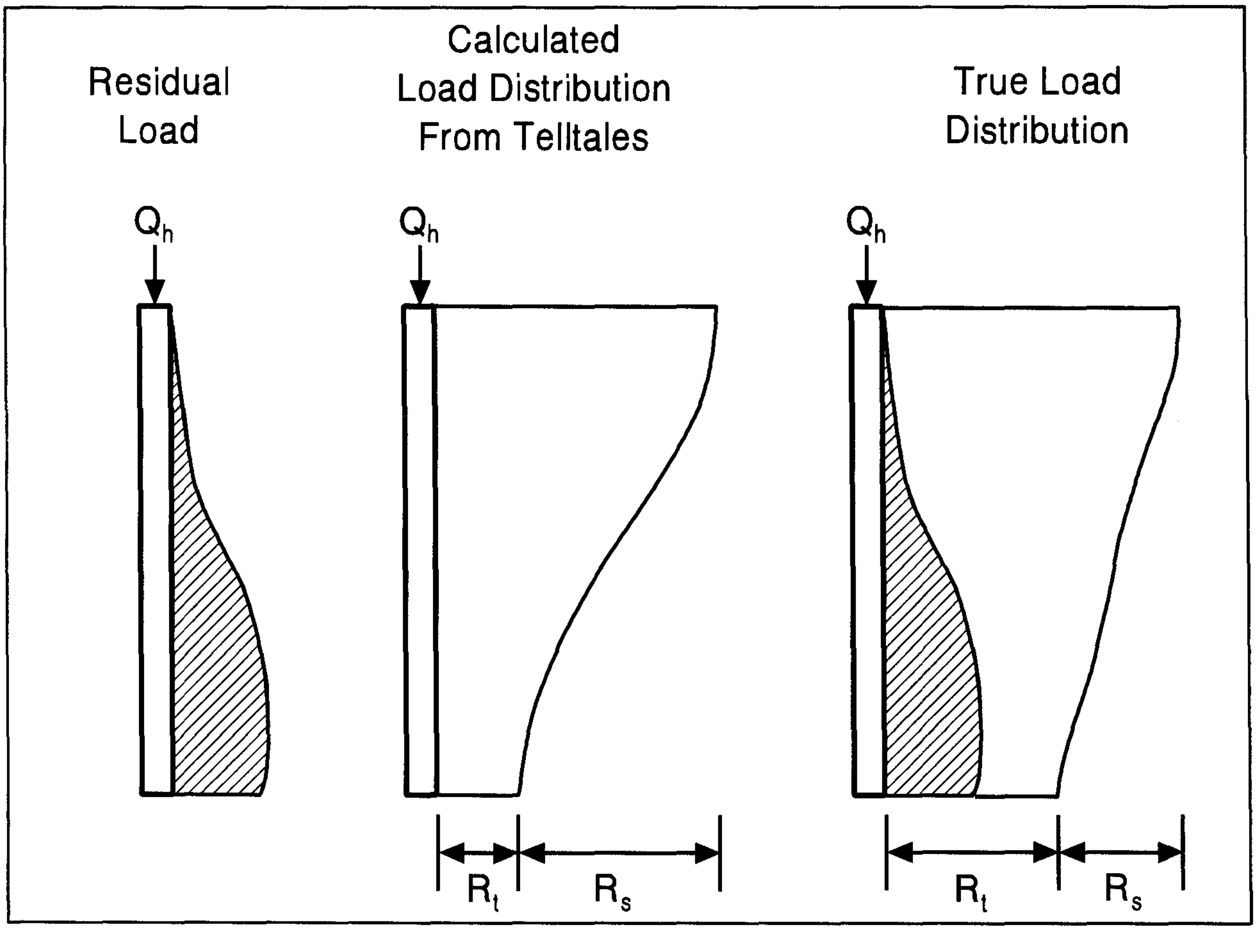


Figure 19.6 Example of Residual Load Effects on Load Transfer Evaluation

19.7.8 Limitations of Compression Load Tests

Compression load tests can provide a wealth of information for design and construction of pile foundations and are the most accurate method of determining pile capacity. However, static load test results cannot be used to account for long-term settlement, downdrag from consolidating and settling soils, or to adequately represent pile group action. Other shortcomings of static load tests include test cost, the time required to setup and complete a test, and the minimal information obtained on driving stresses or extent of pile damage (if any). Static load test results can also be misleading on projects with highly variable soil conditions.

19.8 TENSILE LOAD TESTS

Tensile load tests are performed to determine axial tensile (uplift) load capacities of piles. The uplift capacity of piles is important for pile groups subjected to large overturning moments. Hence, the importance of determining pile uplift capacity has greatly increased in recent years, particularly with regard to seismic design issues. The basic mechanics of the test are similar to compression load testing, except the pile is loaded in tension.

19.8.1 Tension Test Equipment

ASTM D-3689 describes The Standard Method of Testing Individual Piles Under Static Axial Tensile Load by the American Society of Testing Materials. Several alternative systems for (1) applying tensile load to the pile, and (2) measuring movements are provided in this standard. Most often, tensile loads are applied by centering a hydraulic jack on top of a test beam(s) and jacking against a reaction frame connected to the pile to be tested. The test beam in turn is supported by piles or cribbing. When a high degree of accuracy is required, the primary means of measuring the load applied to the pile should be from a calibrated load cell with the jack load recorded from a calibrated pressure gage as backup. A spherical bearing plate should be included in the load application arrangement.

Axial pile head movements are usually measured by dial gages or LVDT's that measure movement between the pile head and an independently supported reference beam. For tensile load testing, ASTM requires a longer travel length and higher precision for movement measuring devices than in a compression load test. For tensile testing, ASTM requires that the dial gages or LVDT's have a minimum of 75 mm of travel and a precision of at least 0.025 mm. A minimum of two dial gages or LVDT's mounted equidistant from the center of the pile and diametrically opposite should be used. Two backup systems consisting of a scale, mirror, and wire system should also be provided with a scale precision of 0.25 mm. The backup systems should be mounted on diametrically opposite pile faces and be independently supported systems. Additional details on load application, and pile head load and movement measurements may be found in ASTM D-3689. A photograph of a typical tension load test arrangement is presented in Figure 19.7.

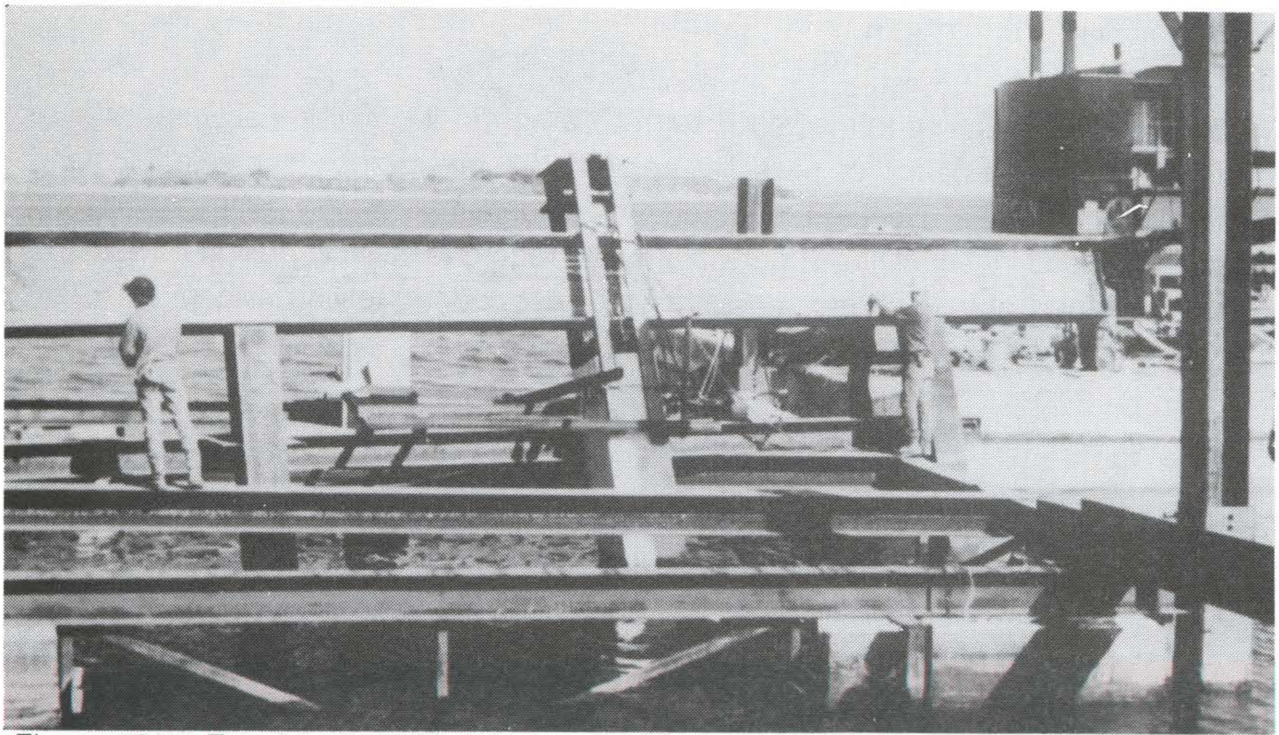


Figure 19.7 Tension Load Test Arrangement on Batter Pile (courtesy of Florida DOT)

19.8.2 Tension Test Loading Methods

Several loading procedures are detailed in ASTM D-3689. The quick loading procedure is recommended. This procedure requires that load be applied in increments of 10 to 15% of the pile design load with a constant time interval of 2½ minutes, or as otherwise specified between load increments. Readings of time, load, and gross movement are to be recorded immediately before and after the addition of each load increment. This procedure is to continue until continuous jacking is required to maintain the test load, or the capacity of the loading apparatus is reached, whichever occurs first. Upon reaching and holding the maximum load for 5 minutes, the pile is unloaded in four equal load decrements which are each held for 5 minutes. Readings of time, load, and gross movement are once again recorded immediately after, 2½ minutes after, and 5 minutes after each load reduction including the zero load. Additional optional loading procedures are detailed in ASTM D-3689.

It is generally desirable to test a pile in tensile loading to failure, particularly during a design stage test program. If construction stage tensile tests are performed on production piles, the piles should be redriven to the original pile toe elevation and the previous driving resistance upon completion of the testing.

19.8.3 Presentation and Interpretation of Tension Test Results

The results of tensile load tests should be presented in a report conforming to the requirements of ASTM D-3689. A load-movement curve similar to the one shown in Figure 19.8 should be plotted for interpretation of tensile load test results.

A widely accepted method for determining the ultimate pile capacity in uplift loading has not been published. Fuller (1983) reported that acceptance criteria for uplift tests have included a limit on the gross or net upward movement of the pile head, the slope of the load movement curve, or an offset limit method that accounts for the elastic lengthening of the pile plus an offset.

Due to the increased importance of tensile load testing, it is recommended that the elastic lengthening of the pile plus an offset limit be used for interpretation of test results. For tensile loading, the suggested offset is 4.0 mm. The load at which the load movement curve intersects the elastic lengthening plus 4.0 mm is then defined as the tensile failure load. The uplift design load may be chosen between ½ to ⅔ of this failure load.

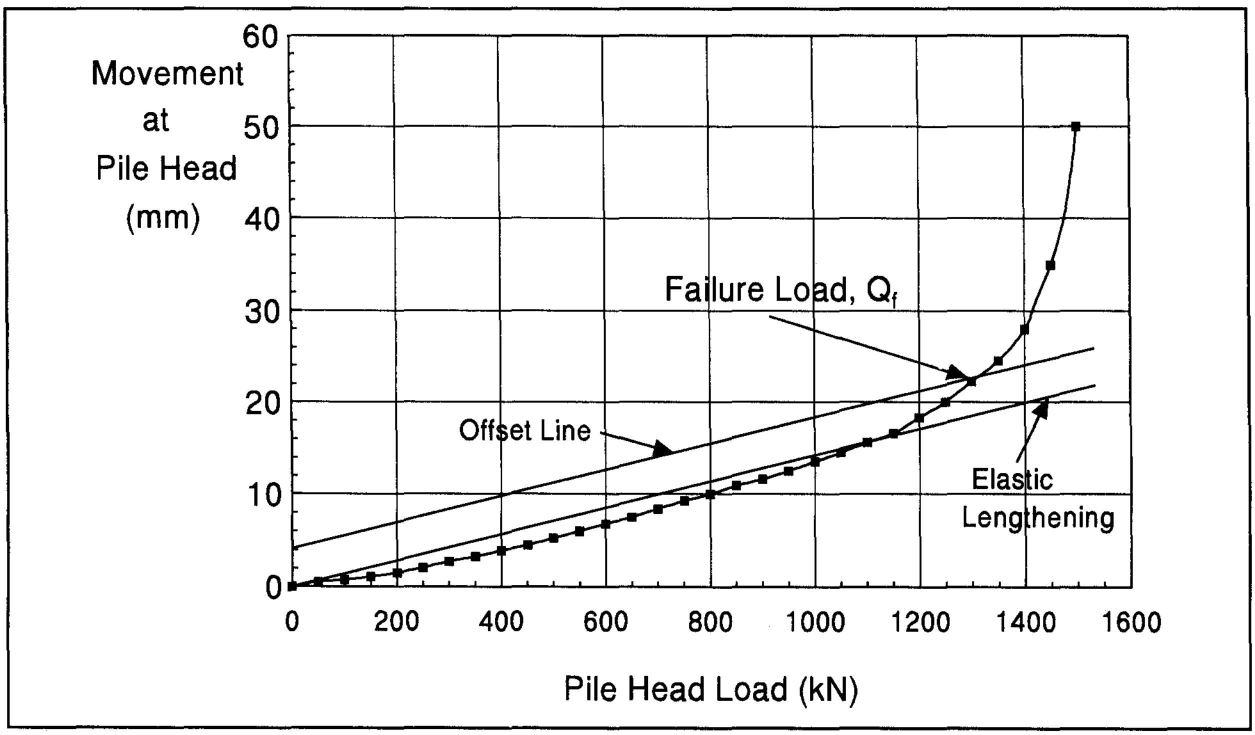


Figure 19.8 Typical Tension Load Test Load-Movement Curve

19.9 LATERAL LOAD TESTS

Lateral load tests are performed on projects where piles are subjected to significant lateral loads. The importance of determining pile response to lateral loading has greatly increased in recent years, particularly with regard to special design events such as seismic and vessel impact. This need has also increased due to the greater use of noise walls and large overhead signs. The primary purpose of lateral load testing is to determine the p-y curves to be used in the design or to verify the appropriateness of the p-y curves on which the design is based.

19.9.1 Lateral Load Test Equipment

ASTM D-3966 describes The Standard Method of Testing Piles Under Lateral Load by the American Society of Testing Materials. Several alternative systems for (1) applying the lateral load to the pile, and (2) measuring movements are provided in this standard. Most often, lateral loads are applied by a hydraulic jack acting against a reaction system (piles, deadman, or weighted platform), or by a hydraulic jack acting between two piles. The primary means of measuring the load applied to the pile(s) should be from a calibrated load cell with the jack load recorded from a calibrated pressure gage as backup. ASTM requires a spherical bearing plate(s) be included in the load application arrangement unless the load is applied by pulling.

Lateral pile head movements are usually measured by dial gages or LVDT's that measure movement between the pile head and an independently supported reference beam mounted perpendicular to the direction of movement. For lateral load testing, ASTM requires the dial gages or LVDT's have a minimum of 75 mm of travel and a precision of at least 0.25 mm. For tests on a single pile, one dial gage or LVDT is mounted on the side of the test pile opposite the point of load application. A backup system consisting of a scale, mirror, and wire system should be provided with a scale precision of 0.25 mm. The backup system is mounted on the top center of the test pile or on a bracket mounted along the line of load application.

It is strongly recommended that lateral deflection measurements versus depth also be obtained during a lateral load test. This can be accomplished by installing an inclinometer casing on or in the test pile to a depth of 10 to 20 pile diameters and recording inclinometer readings immediately after application or removal of a load increment held for a duration of 30 minutes or longer. Kyfor *et al.* (1992) noted that lateral load tests in which only the lateral deflection of the pile head is measured are seldom justifiable. Additional details on load application, and pile head load and movement measurements may be found in ASTM D-3966 and FHWA-SA-91-042. A photograph of a typical lateral load test arrangement is presented in Figure 19.9.

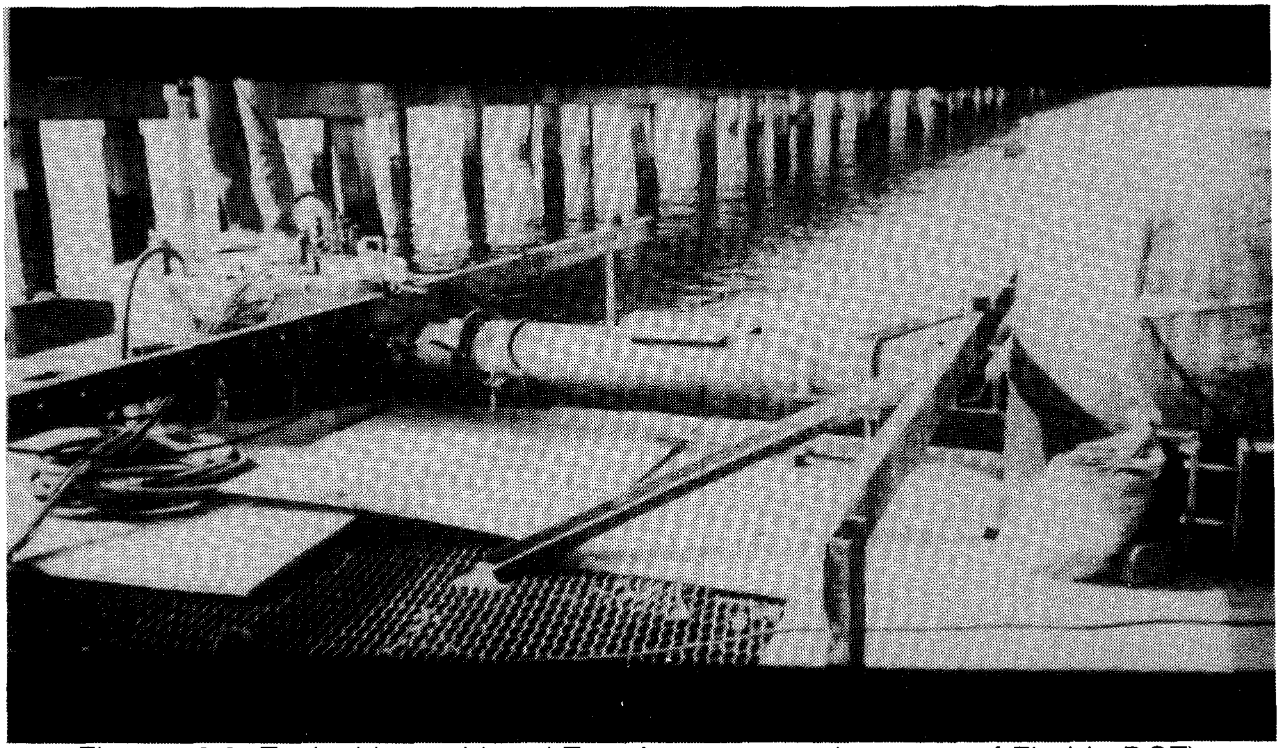


Figure 19.9 Typical Lateral Load Test Arrangement (courtesy of Florida DOT)

19.9.2 Lateral Test Loading Methods

Several loading procedures are detailed in ASTM D-3966. The standard loading procedure requires that the total test load be 200% of the proposed lateral design load. Variable load increments are applied with the magnitude of load increment decreasing with applied load. The load duration is also variable, increasing from 10 minutes early in the test to 60 minutes at the maximum load. Upon completing the maximum test load, the pile is unloaded in four load decrements equal to 25% of the maximum load with 1 hour between load decrements.

A modified lateral loading schedule was proposed by Kyfor *et al.* in FHWA-SA-91-042. The recommended loading increment is 12.5% of the total test load with each load increment held for 30 minutes. Upon reaching and holding the maximum load for 60 minutes, the pile is unloaded and held for 30 minutes at 75, 50, 25 and 5% of the test load.

Readings of time, load, and gross movement are recorded immediately after each change in load. Additional readings are taken at 1, 2, 4, 8, 15 and 30 minutes. This procedure is followed during both the loading and unloading cycle.

19.9.3 Presentation and Interpretation of Lateral Test Results

The results of lateral load tests should be presented in a report conforming to the requirements of ASTM D-3966. The interpretation and analysis of lateral load test results is much more complicated than those for compression and tensile load testing. Figure 19.10 presents a typical lateral load test pile head load-movement curve. A lateral deflection versus depth curve similar to the one shown in Figure 19.11 should also be plotted for interpretation of lateral load test results that include lateral deflection measurements versus depth. The measured lateral load test results should then be plotted and compared with the calculated result as indicated in Figure 19.11.

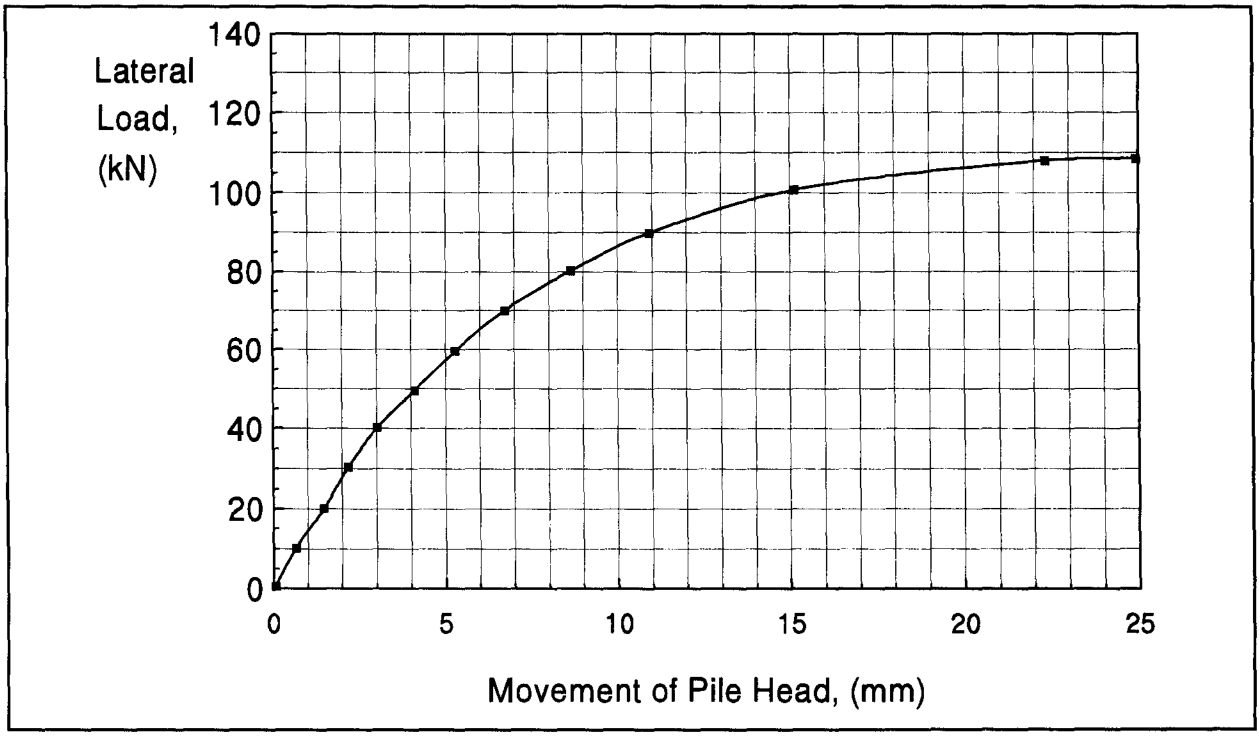


Figure 19.10 Typical Lateral Load Test Pile Head Load-Deflection Curve

Based upon the comparison of measured and predicted results, the p-y curves to be used for design (design stage tests), or the validity of the p-y curves on which the design was based (construction stage tests) can be determined.

Refer to FHWA-IP-84-11, Handbook on Design of Piles and Drilled Shafts Under Lateral Load by Reese (1984) as well as FHWA-SA-91-042, Static Testing of Deep Foundation by Kyfor *et al.* (1992) for additional information on methods of analysis and interpretation of lateral load test results.

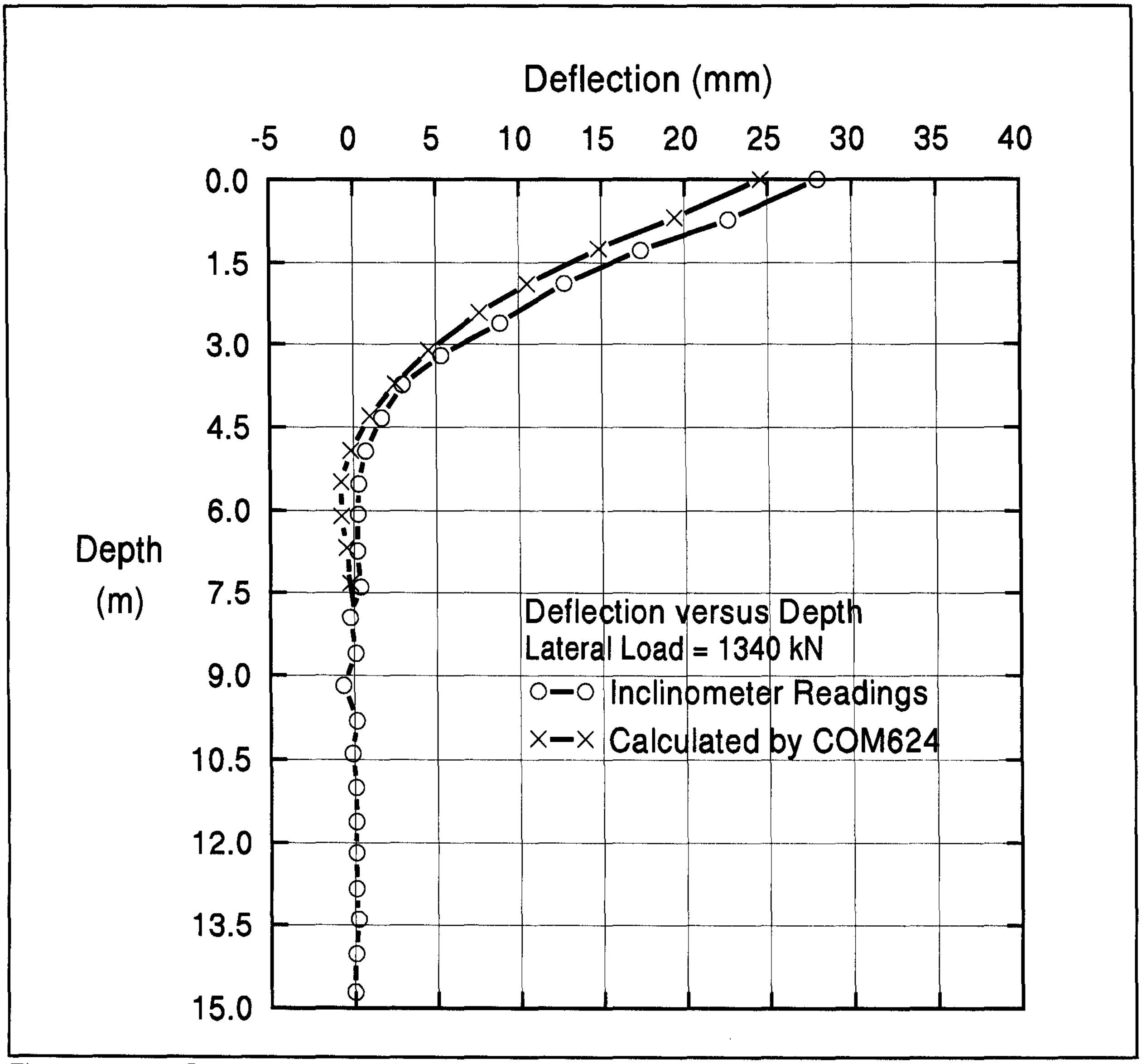


Figure 19.11 Comparison of Measured and COM624P Predicted Load-Deflection Behavior versus Depth (after Kyfor *et al.* 1992).

REFERENCES

American Association of State Highway and Transportation Officials [AASHTO], (1992). Standard Specifications for Highway Bridges. Division 2, Washington, D.C.

American Society for Testing and Materials, ASTM (1994). Annual Book of Standards, ASTM D-1143, Standard Test Method for Piles Under Static Axial Compressive Load.

American Society for Testing and Materials, ASTM (1994). Annual Book of Standards, ASTM D-3936, The Standard Method of Testing Individual Piles Under Static Axial Tensile Load.

American Society for Testing and Materials, ASTM (1994). Annual Book of Standards, ASTM D-3936, The Standard Method of Testing Piles Under Lateral Load.

Cheney, R.S. and Chassie, R.G. (1993). Soils and Foundations Workshop Manual. Second Edition, Publication No. FHWA HI-88-009, Federal Highway Administration, National Highway Institute, Washington, D.C., 353-362.

Crowther, C.L. (1988). Load Testing of Deep Foundations: the Planning, Design, and Conduct of Pile Load Tests. John Wiley & Sons, New York, 233.

Davisson, M.T. (1972). High Capacity Piles, Proceedings, Soil Mechanics Lecture Series on Innovations in Foundation Construction. American Society of Civil Engineers, ASCE, Illinois Section, Chicago, 81-112.

Dunnicliff, J. (1988). Geotechnical Instrumentation for Monitoring Field Performance. John Wiley & Sons, New York, 467-479.

Fellenius, B.H. (1990). Guidelines for the Interpretation of the Static Loading Test. Deep Foundations Institute Short Course Text, First Edition, 44.

Fuller, F.M. (1983). Engineering of Pile Installations. McGraw-Hill, New York, 286.

Kyfor, Z.G., Schnore, A.S., Carlo, T.A. and Bailey, P.F. (1992). Static Testing of Deep Foundations. Report No. FHWA-SA-91-042, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 174.

Reese, L.C. (1984). Handbook on Design of Piles and Drilled Shafts Under Lateral Load. Report No. FHWA-IP-84-11, U.S. Department of Transportation, Federal Highway Administration, Office of Implementation, McLean, 386.

STUDENT EXERCISE #13 - DETERMINATION OF LOAD TEST FAILURE LOAD

An axial compression static load test has been performed and the results must be interpreted to determine if the pile has an ultimate capacity in excess of the required ultimate capacity. The load - movement curve from the static load on a 356 mm square prestressed concrete pile is presented on the following page. The pile has a cross sectional area, A, of 0.127 m^2 and a length, L, of 24 m. The concrete compression strength, f'_c is 34.5 MPa. The pile has a required ultimate pile capacity of 2200 kN.

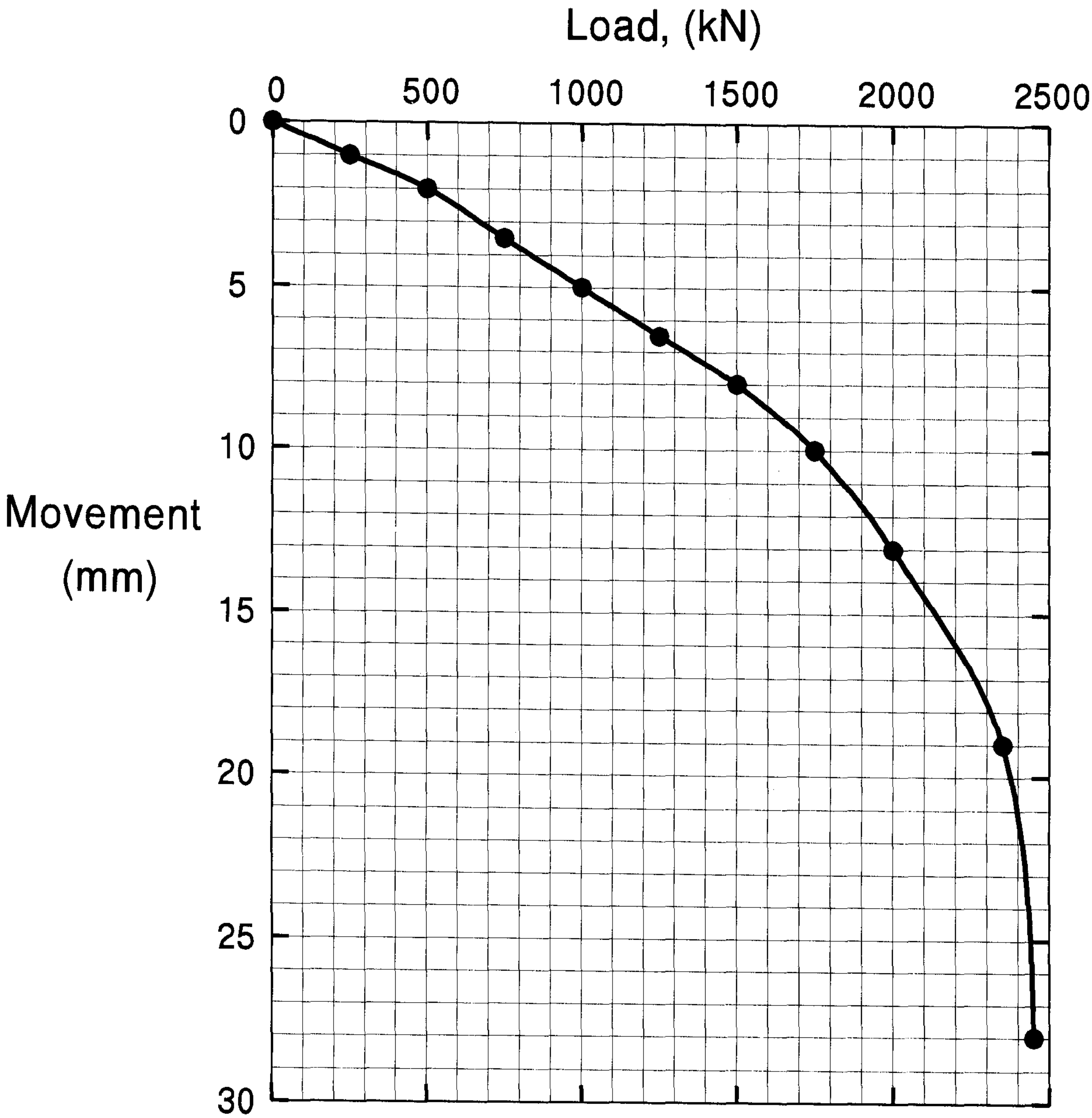
Recommended Procedure:

First determine, the elastic modulus, E, of the pile from the concrete compressive strength using $E = 4700 \sqrt{f'_c}$ where f'_c must be in MPa.

Next, calculate and plot the elastic deformation line using zero and any other load. However, for consistency between solutions and ease in plotting, calculate the elastic deformation using a load of 2500 kN from $\Delta = QL / AE$. Make sure the units for the terms in this equation are as required in the equation description provided in Section 19.7.4.

Then calculate the failure criterion line for the 356 mm pile from $s_f = \Delta + (4.0 + 0.008b)$ as described in Section 19.7.5. Remember at zero load, the failure criterion line will start at a movement equal to $(4.0 + 0.008b)$ and at 2500 kN, the failure criterion line will be equal to a movement of $s_f = \Delta + (4.0 + 0.008b)$.

Last, plot the failure criterion line on the load-movement curve and determine whether the failure load is greater than the required ultimate pile capacity of 2200 kN.



Load, (kN)
0
500
1000
1500
2000
2500
Movement
(mm)
0
5
10
15
20
25
30

20. THE OSTERBERG CELL METHOD

Another recent development for evaluation of driven pile capacity is the Osterberg Cell test or O-cell test. This device provides a simple, efficient and economical method of performing a static test on a deep foundation. The O-cell is a sacrificial jack which is generally attached to the toe of a driven pile before driving.

The Osterberg Cell test can be easily applied to driven, displacement piles such as closed end pipe piles and prestressed concrete piles. The O-cell cannot be employed with H-piles, sheet piles or timber piles. Closed end pipe piles and concrete piles require cell installation prior to driving, and thus additional prior planning is needed. For open end pipe piles and mandrel driven piles, the cell may be installed after driving is complete.

Testing a driven pile with an O-cell eliminates the need for a reaction system and can provide significant cost and time savings. The Osterberg Cell has many applications and provides the engineer with a new, cost effective tool and added versatility for the static testing of driven piles. The Osterberg Cell Method is not standardized by AASHTO or ASTM and is nationally licensed to a single source. Additional information on the Osterberg Cell may be found in FHWA publication FHWA-SA-94-035 by Osterberg (1995).

20.1 OSTERBERG CELL BACKGROUND

Dr. Jorj Osterberg, Professor Emeritus at Northwestern University, developed and patented the test which now carries his name. The device was first used in an experimental drilled shaft in 1984. Following this successful prototype test, the O-cell evolved from a bellows type expansion cell to the current design, which is very similar to the piston type jack commonly used for conventional tests. However, the piston of the O-cell extends downward instead of upward.

The first O-cell test on a driven pile occurred in 1987. In this initial driven pile application, a 457 mm diameter O-cell was welded to the toe of an 457 mm diameter, closed end, steel pipe pile. In 1994, the first O-cell tests were performed on 457 mm square, prestressed concrete piles. For these piles, the O-cell was cast into the pile toe.

Figure 20.1 presents a schematic of the difference between a conventional static load test and an O-cell test. A conventional static test loads the pile in compression from the pile head using an overhead reaction system or dead load. The combination of shaft and toe resistances resist the applied pile head load. The shaft and toe resistances can be separated by analysis of strain gage or telltale measurements.

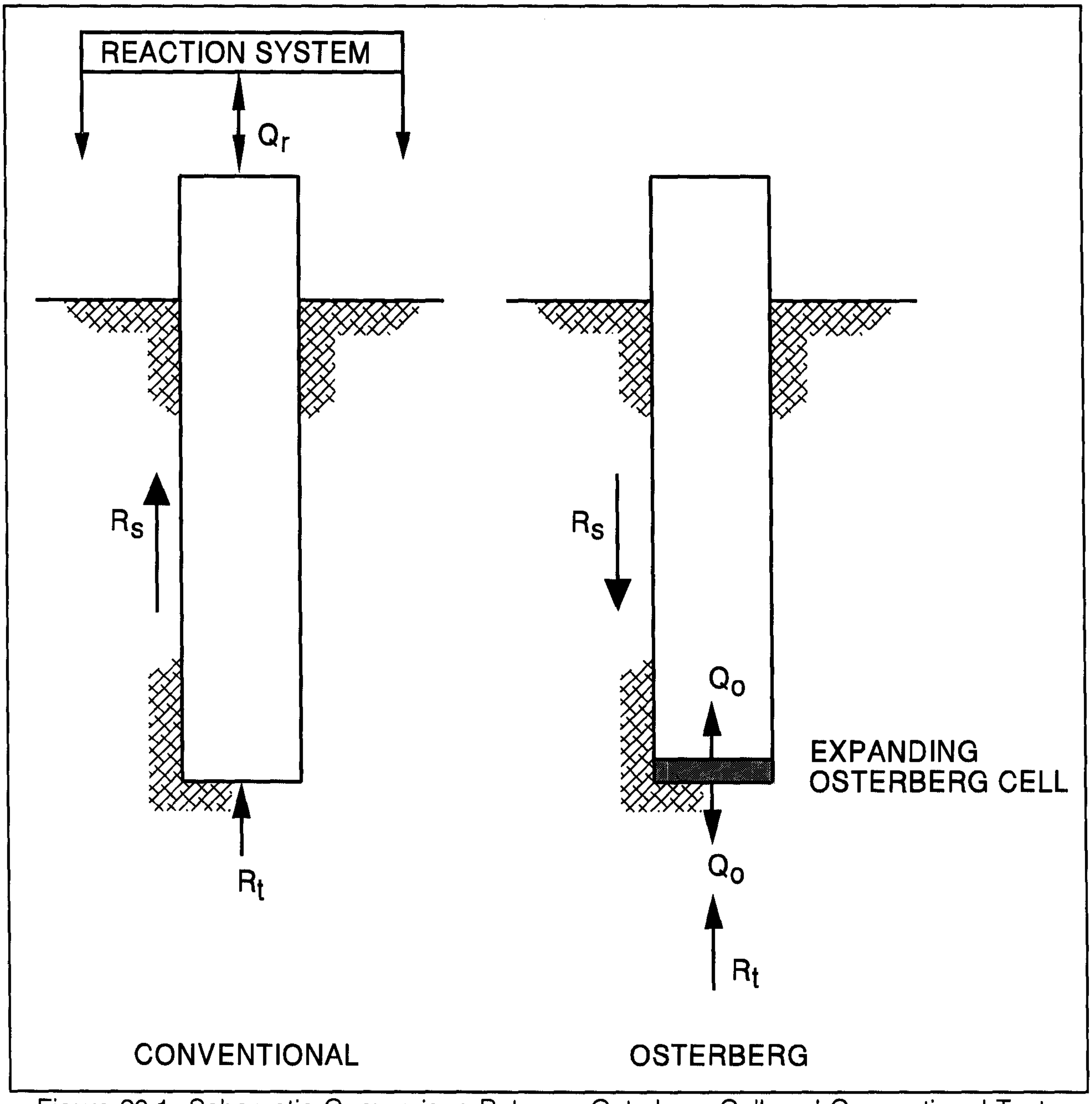


Figure 20.1 Schematic Comparison Between Osterberg Cell and Conventional Tests

In an O-cell test, the pile is also loaded in compression, but the load is applied at the pile toe. As the cell expands, the toe resistance provides reaction for the shaft resistance, and vice versa. The test is complete when either the ultimate shaft or toe resistance is reached, or the cell reaches its capacity.

An O-cell test automatically separates the toe and shaft resistance components. When one of the components fails at an O-cell load, Q_o, the conventional pile head load, Q_r, required to fail both the shaft resistance and toe resistance would have to exceed $2Q_o$. Thus, an O-cell test load placed at the pile toe is always twice as effective as the same load placed at the pile head.

20.2 TEST EQUIPMENT

The O-cell in its current design is capable of developing an internal pressure of 69 MPa. Typical cell capacities for driven piles of up to 8000 kN have been used. The cell consists of a piston and cylinder coupled to high strength pipe that extends inside the pile to the ground surface. The total allowable expansion of a standard O-cell is about 150 mm with greater expansion possible by special order. Figure 20.2 shows a typical cross section of a concrete pile and the setup for an O-cell test.

Tests performed using the O-cell usually follow the quick loading method described in ASTM D-1143, Standard Test Method for Piles Under Static Axial Compressive Load. However, other methods are not precluded. Instrumentation used to measure load and movement is similar to that used for conventional load tests. The O-cell is designed so that driving forces are transmitted through the cell without damage to the cell or the pile. An O-cell ready for placement in a 457 mm prestressed concrete pile is shown in Figure 20.3. After this pile was cast, the only visible parts of the O-cell were the bottom plates.

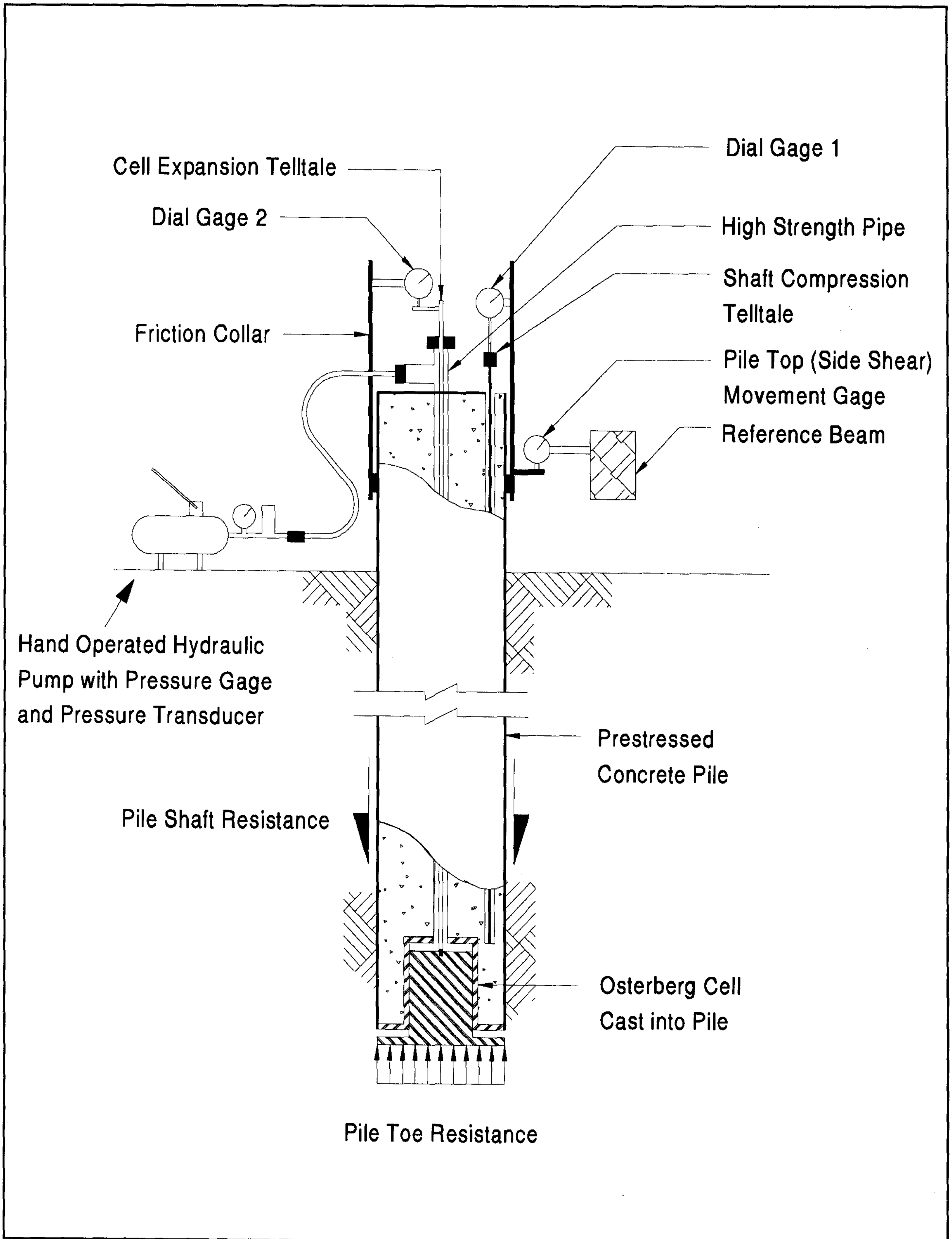


Figure 20.2 Osterberg Cell and Related Equipment Used for Static Pile Tests

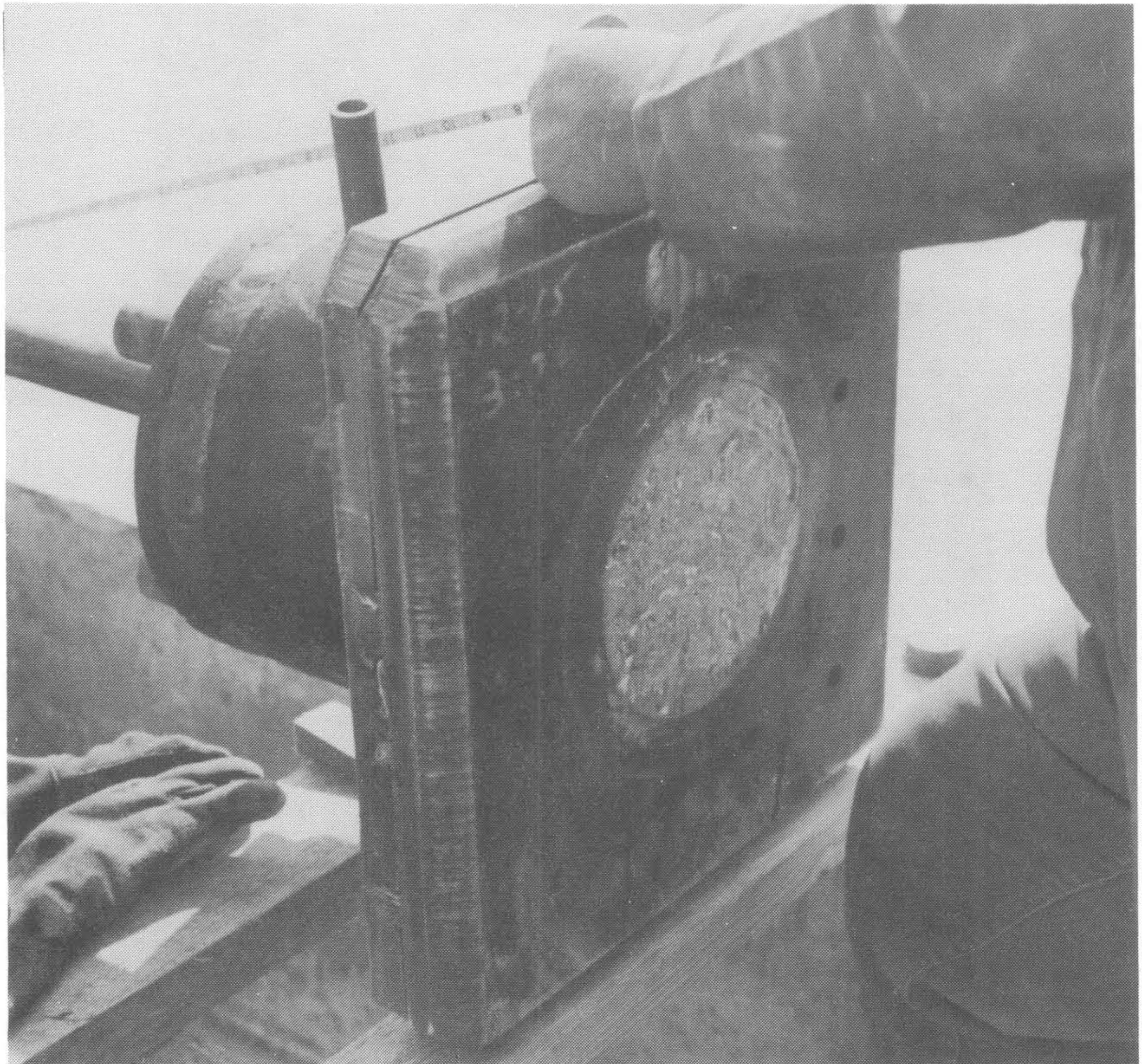


Figure 20.3 Osterberg Cell Ready for Placement in Concrete Pile Form (courtesy of Loadtest, Inc.)

After the pile is driven, a hand pump or small automatic pump (electric or air driven) is connected to a central pipe which provides a pressure conduit to the O-cell. The load applied by an O-cell is calibrated versus hydraulic pressure before installation and the pressure applied to the cell is measured using a Bourdon gage or pressure transducer. The O-cell seals typically limit internal friction to less than 2% of the applied load. In Figure 20.4, both a vibrating wire piezometer and a test gage are being used to measure the cell pressure, which is applied with a hand pump.

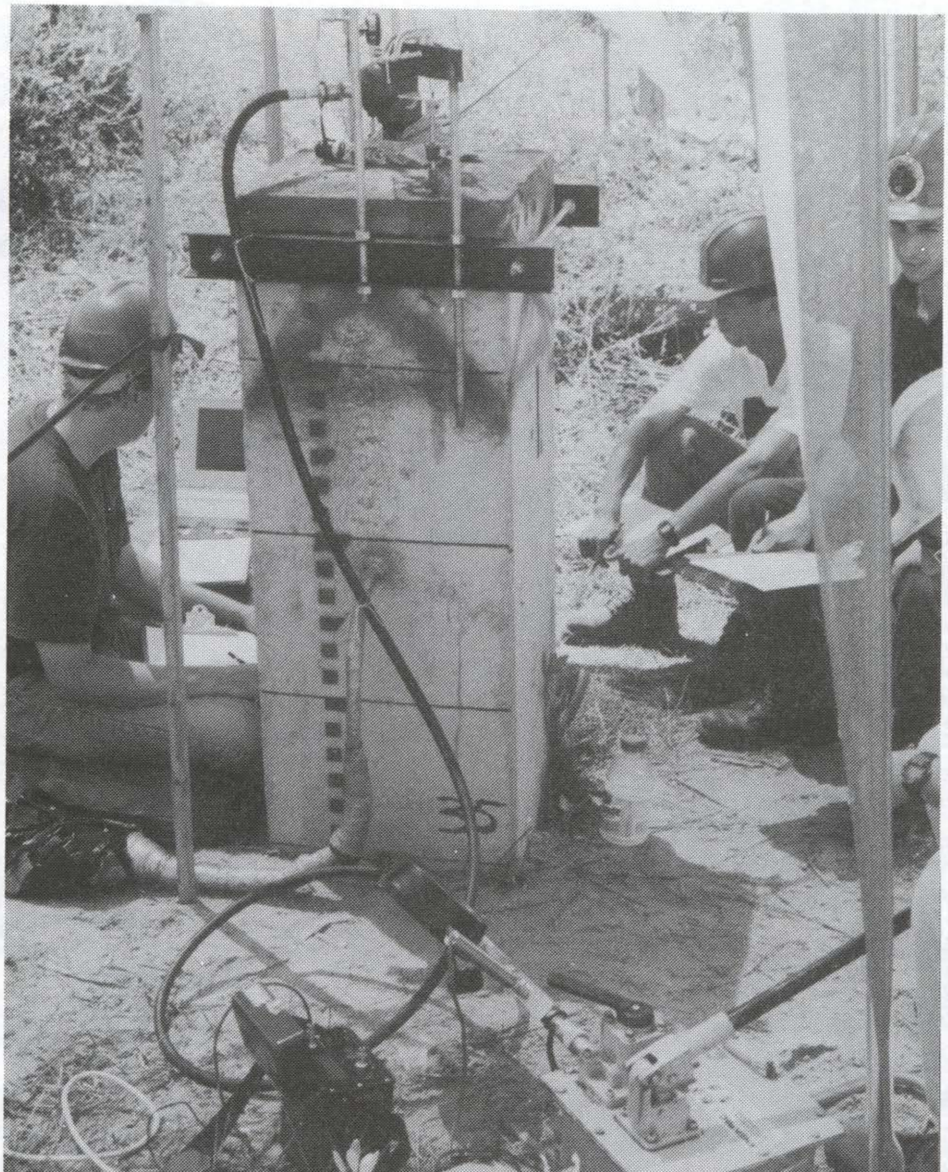


Figure 20.4 Osterberg Test in Progress on a 457 mm Concrete Pile (courtesy of Loadtest, Inc.)

For closed end pipe piles the cell must be installed before driving the pile but the pressure pipe and connection tee may be installed afterwards. The cell and pipe are normally installed in concrete piles during construction of the pile and the pipe tee is welded on after driving. For open ended pipe piles and mandrel driven piles, the cell and pipe assembly may be placed as a combined unit after the pile is driven and then concreted in place.

Movements during an O-cell test are typically measured using mechanical or electronic gages. The cell expansion (less any pile compression) is directly indicated by a steel telltale which extends to the bottom of the cell. This telltale is placed in the central pressure pipe and exits through an O-ring seal on the connection tee. Other telltales, indicating compression of the pile, are usually installed in pairs. They help with estimating the shaft resistance distribution and calculations for movement. The upward movement of the pile, which is resisted by the downward shaft resistance, is measured by gages mounted to a reference beam and checked by an independent measurement such as a survey level.

When an O-cell test is performed on a production pile it will usually be necessary to grout the O-cell after completing the test. This is accomplished by unscrewing the cell expansion telltale illustrated in Figure 20.2 from the O-cell and inserting a grout pipe in its place. The grout pipe is gradually removed as the grout flows out of the pipe under gravity flow. A fitting can also be attached to the top of the high strength pipe and the grout pipe connected directly to the grout pump in cases where it is desirable to place the grout under pressure to partially mobilize the pile toe resistance. However, to avoid soil creep, the grout pressure should be maintained below the maximum pressure applied in the O-cell test.

20.3 INTERPRETATION OF TEST RESULTS

The Osterberg Cell loads the test pile in compression similar to a conventional static load test. Data from an Osterberg test is therefore analyzed much the same as conventional static test data. The only significant difference is that the O-cell provides two load-movement curves, one for shaft resistance and one for toe resistance. The failure load for each component may be determined from these curves using a failure criteria similar to that recommended for conventional load tests. To determine the shaft resistance capacity, the buoyant weight of the pile should be subtracted from the upward O-cell load, and the elastic deformation of the pile shaft should be included. Analysis for the toe resistance should not include the elastic pile deformation since the load is applied directly at the pile toe.

The engineer may further utilize the component curves to construct an equivalent pile head load-movement curve and investigate the overall pile capacity. Construction of the equivalent pile head load-movement curve begins by determining the shaft resistance at an arbitrary movement point on the shaft resistance-movement curve. If the pile is assumed rigid, the pile head and toe move together and have the same movement at this load. By adding the shaft resistance to the mobilized toe resistance at the chosen movement, a single point on the equivalent pile head load-movement curve is determined. Additional points may then be calculated to develop the curve up to the maximum movement (or maximum extrapolated movement) of the component that did not fail. Points beyond the maximum movement of the non-failing component may also be obtained by conservatively assuming that at greater movements it remains constant at the maximum applied load. Example results using this method are included with the case history data below.

As noted by Osterberg (1994), the above construction makes three basic assumptions:

1. The shaft resistance load-movement curve resulting from the upward movement of the top of the O-cell is the same as developed by the downward pile head movement of a conventional compression load test.

2. The toe resistance load-movement curve resulting from the downward movement of the bottom of the O-cell is the same as developed by the downward pile toe movement of a conventional load test.

3. The compression of the pile is considered negligible, *i.e.* a rigid pile.

The first of these assumptions highlights a significant difference between the O-cell test and a conventional compression load test, namely the change in direction of the mobilized shaft resistance from downward to upward. Researchers at the University of Florida have investigated the effect of this direction reversal using the finite element method. Their results indicate that the O-cell produces slightly lower shaft resistance than a conventional load test, but that in general the effect is small and may be ignored. A few full scale field tests tend to confirm these findings. Note that the shaft resistance direction in an O-cell test matches that in a conventional tension or uplift test.

Lower confining stresses due to the gap induced around the expanding cell may also cause the O-cell to measure a slightly lower toe resistance, but this effect is conservative and also seems negligible. The compression of the pile is normally a second order effect and the assumption of a rigid pile causes a negligible error. In general, the above assumptions seem to produce conservative and reasonable results.

20.4 APPLICATIONS

Although its use is not feasible for all pile types, the O-cell test has many potential applications with common driven piles. Its versatility also provides additional options. A partial list of applications follows:

1. Displacement Piles: The O-cell may be installed prior to driving solid concrete piles and closed end pipe piles.

2. Mandrel Driven Piles: Mandrel driven piles can be tested by grouting the O-cell into the pile toe after removing the mandrel.

3. Open Ended Pipe Piles: The O-cell may be installed in open ended pipe piles and voided concrete piles by removing the soil plug after driving.

4. Batter Piles: Conventional static load tests to evaluate the axial capacity of batter piles can be very difficult to perform. For applicable pile types in these situations, the O-cell test offers an alternate test method that is easier to perform.

5. Testing Over Water or at Constricted Sites: Because the O-cell test requires no overhead reaction, the surface test setup is minimized. Tests over water require only a work platform. Sites with poor access, limited headroom or confined work area are ideal applications for an O-cell test.

6. Proof Tests: Because of the simplicity and usually lower cost of O-cell tests compared to conventional static load tests, several piles can be economically proof tested as a check of pile capacity.

7. Repetitive Tests: Multiple static tests on the same pile may be performed with the O-cell to investigate the effect of time on pile capacity. Use of the O-cell minimizes the mobilization required for each static test.

8. Exploratory Testing: With the proper design, it is possible to use the O-cell to test the same pile at different pile penetration depths. After each test, the pile is driven deeper and retested. This method also develops the shaft resistance distribution incrementally.

20.5 ADVANTAGES

Osterberg (1994) and Schmertmann (1993) summarized a number of potential advantages vs. conventional testing that may be realized by using the Osterberg Cell. These include:

1. Economy: The O-cell test is usually less expensive to perform than a conventional static test despite sacrificing the cell. Savings are realized through reduced setup time and capital outlay, less heavy equipment, fewer structural connections and less test design effort. O-cell tests are typically ⅓ - ⅔ the cost of conventional tests. The relative economy improves as the required maximum test load increases.

2. Static Creep and Setup Effects: Because the O-cell test is static, and the test load can be held for any desired length of time (typically 5 minute increments), data about the creep behavior of the shaft and toe resistances can be obtained. Creep limits may be obtained which are similar to those from pressuremeter tests described in ASTM D4719 (ASTM,1993). Soil setup effects can also be conveniently measured at any time after driving.

3. Improved Safety: Because there is no overhead load, failure of the load system creates a minimal safety hazard.

4. Reduced Work Area: The work area required to perform an O-cell test is much smaller, both overhead and laterally, than the area required for a conventional load system.

5. High Load Capacity: Very high capacity loading is possible for large piles or whole groups of piles. Drilled shafts have been tested to over 53,400 kN equivalent conventional test load.

6. Shaft/Toe Resistance Determination: The O-cell test clearly separates the shaft and toe resistance components.

7. Multiple Tests: The O-cell provides a convenient method to obtain additional tests on the same pile, at multiple toe elevations and/or after elapsed time at the same toe elevation.

20.6 DISADVANTAGES

The O-cell has some disadvantages or limitations compared to conventional tests as discussed below:

1. Not Suitable for Certain Types of Piles: The O-cell cannot be used to test H-piles. Installation of an O-cell on a timber pile would be difficult. Installation in open end pipe piles is feasible, but requires internal pile cleanout after driving for cell placement and subsequent concrete or grout placement above the installed cell. In tapered piles, the equivalent shaft resistance of a tapered pile loaded in compression will not be developed since the effects of the taper will be lost when loaded upward from the pile toe in an O-cell test.

2. Need for Planning: With closed end and solid displacement piles, the O-cell must be installed prior to driving. For these pile types, an O-cell test cannot be chosen after installation.

3. Limited Capacity: An O-cell test reaches the ultimate load in only one of the two resistance components. The pile capacity demonstrated by the O-cell test is limited to two times the failed component. Also, once installed, the cell capacity cannot be increased if inadequate. To use the cell efficiently, the engineer should first analyze the expected shaft and toe resistances, and then attempt to balance the two or ensure a failure in the preferred component.

4. Equivalent Pile Head Load-Movement Curve: Although the equivalent static load-movement curve can be constructed from O-cell test data, it is not a direct measurement and may be too conservative.

20.7 CASE HISTORIES

To date, only closed end pipe piles and prestressed concrete driven piles have been tested using the O-cell. Case studies for both pile types are presented below.

In 1987, a 457 mm diameter steel pipe pile with an O-cell of the same diameter welded to the pile toe was driven at the Pines River Bridge in Saugus, MA. As shown in Figure 20.5, this pile was driven through soft clay and a layer of glacial till, then founded in weathered Argillite rock at a depth of 36 m below the ground surface. It was driven to practical refusal with a Delmag D 36-13 diesel hammer with a rated energy 112.7 kJ. The final driving resistance was 10 blows for the last 13 mm.

As indicated by the shaft and toe resistance load-movement curves shown in Figure 20.6, the Pines River pile failed in shaft resistance at a cell load of 1910 kN. The small upward movement evident during the initial portion of the test is due to pressure effects on the central pipe and has little effect on the capacity results. After subtracting the pile weight to get shaft resistance, the minimum ultimate capacity of this pile was estimated as 3740 kN. An equivalent pile head load-movement curve constructed from the test data is included in Figure 20.7. The maximum toe resistance of 1910 kN was used at movements greater than 1.0 mm. For reference, the numbered pile head load-movement points were calculated at movements corresponding to the numbered points on the shaft resistance curve. Thompson *et al.* (1989) provides additional details on this case history.

O-cell tests can also be useful for special investigations. For example, O-cells were recently cast into four 457 mm square, prestressed concrete piles which were then driven and tested as part of a research project by the University of Florida (UF). This research project is investigating long term shaft resistance changes. The O-cell is being used in this application to perform repeated tests over a period of at least two years after the piles are driven. To allow the prestressed pile manufacturer to cast ordinary production piles along with the research piles, the O-cell used for the UF research is designed to fit within the standard prestressed cable pattern. The strands were then pulled through holes drilled in the load plates of the cell. These cells have a 229 mm diameter piston and a maximum stroke of 152 mm. They provide a capacity of 2700 kN at a pressure of 69 MPa. To prevent damage during driving extra lateral reinforcement was added at the pile toe. Longitudinal reinforcement was also added above the O-cell to insure good load transfer during testing and driving. Otherwise, the research piles followed a standard Florida DOT design and were cast as part of a full production bed of piles.

The pile driven at Aucilla River is 22 m long and has been tested four times over a 2 month period. Its shaft resistance has increased 64% over this time period to 1490 kN, and indications are that it will continue to increase in capacity with additional time. As shown in Figure 20.8, this pile was driven to bearing on limerock at 16 blows for the final 25 mm using a Fairchild 32 air hammer with a rated energy of 43.4 kJ. Figures 20.9 and 20.10 show the component and equivalent pile head load-movement curves for the most recent test. The maximum toe resistance of 1560 kN was used at deflections greater than 2.3 mm. Repeated tests have influenced the toe resistance, which now shows some disturbance effects in the early loads. Otherwise this test is representative of the research results.

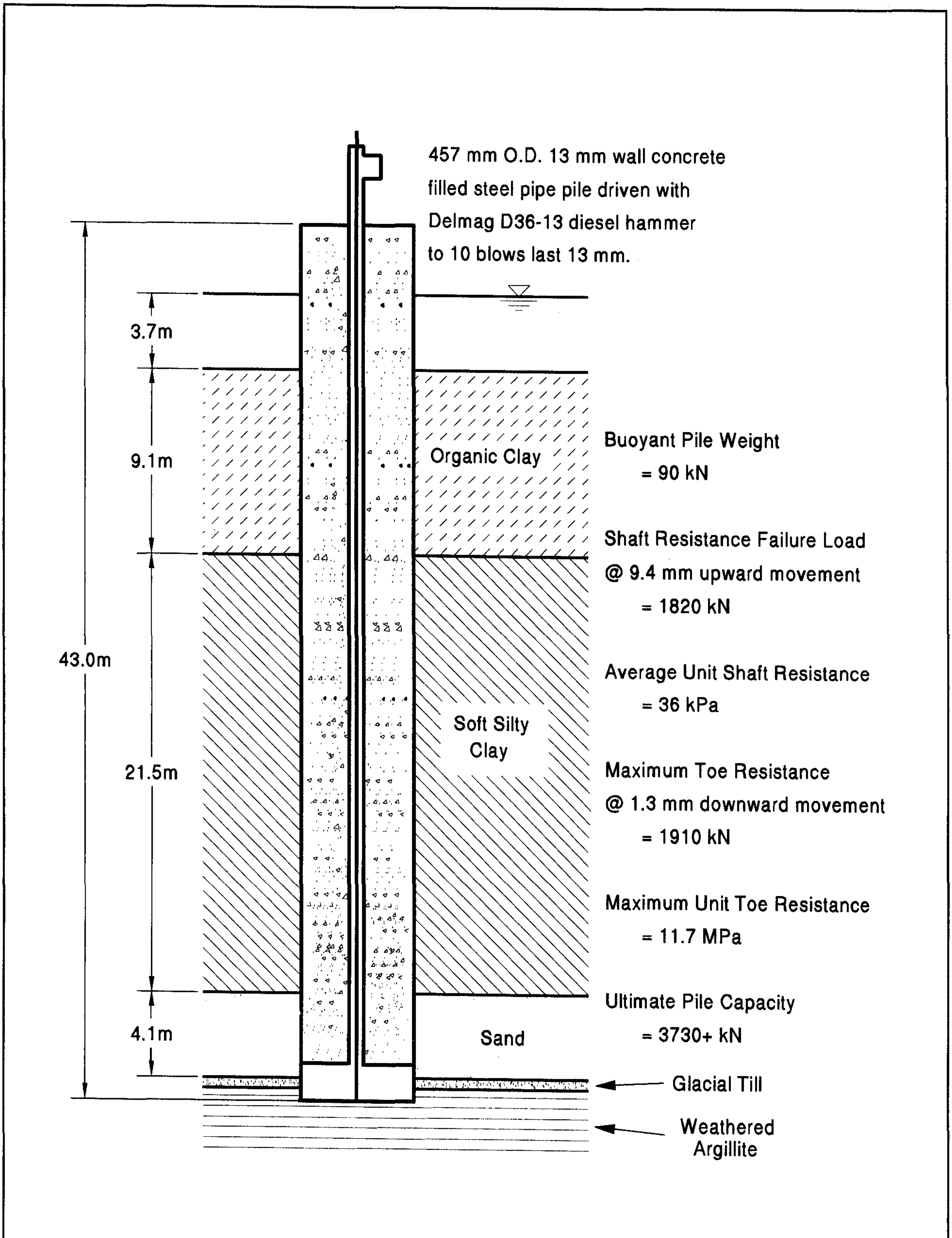


Figure 20.5 Summary of Subsurface Profile and Test Results at Pines River Bridge, MA

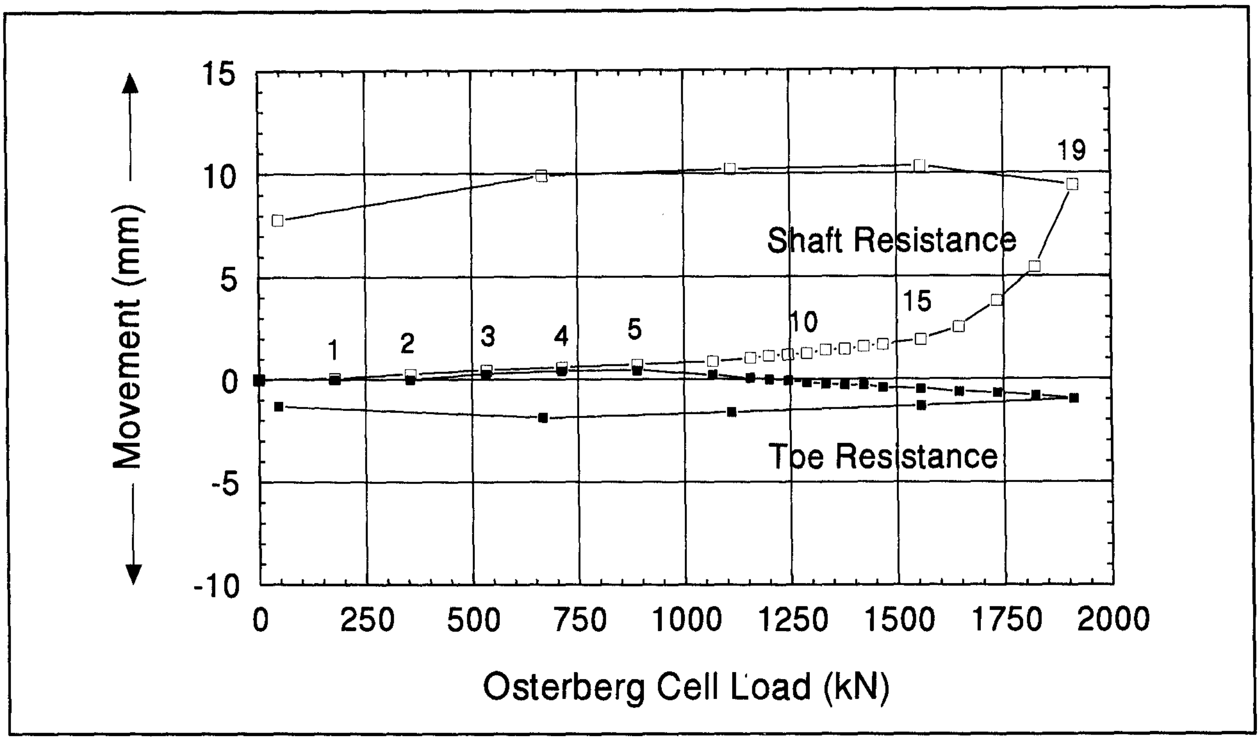


Figure 20.6 Test Results from Pines River Bridge, MA

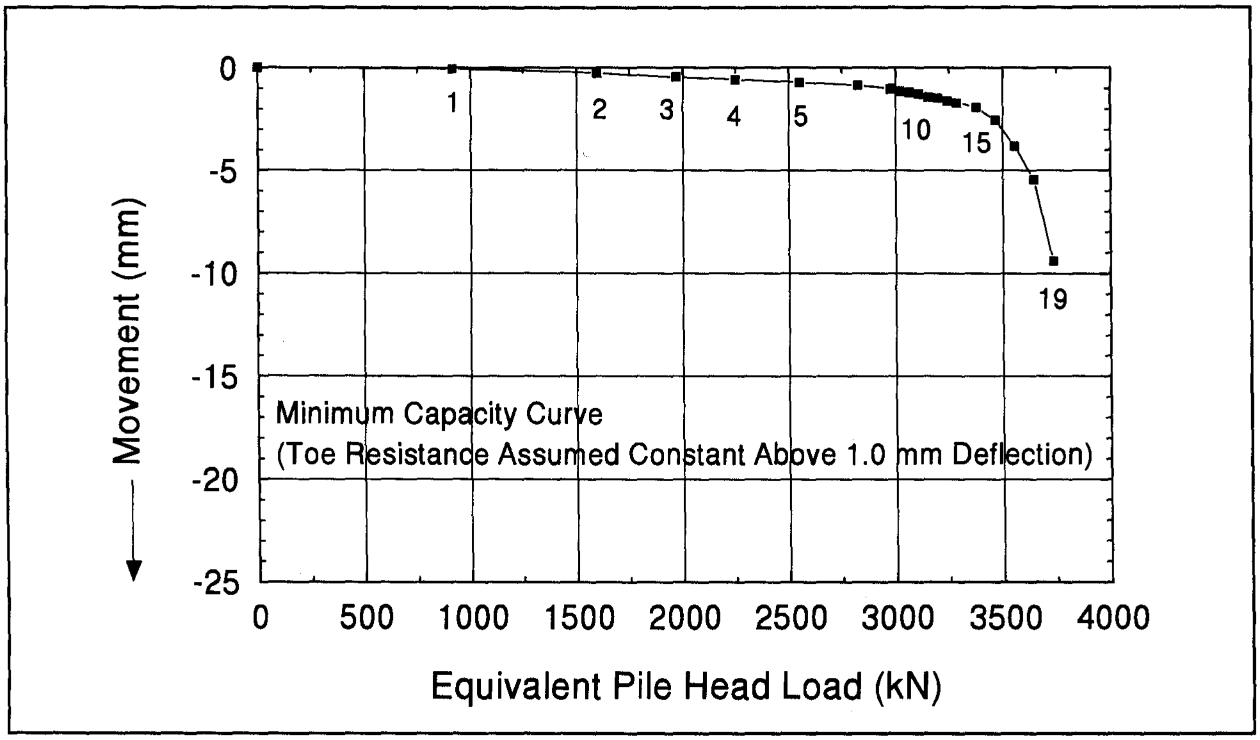


Figure 20.7 Equivalent Pile Head Load-Movement Curve from Pines River Bridge, MA

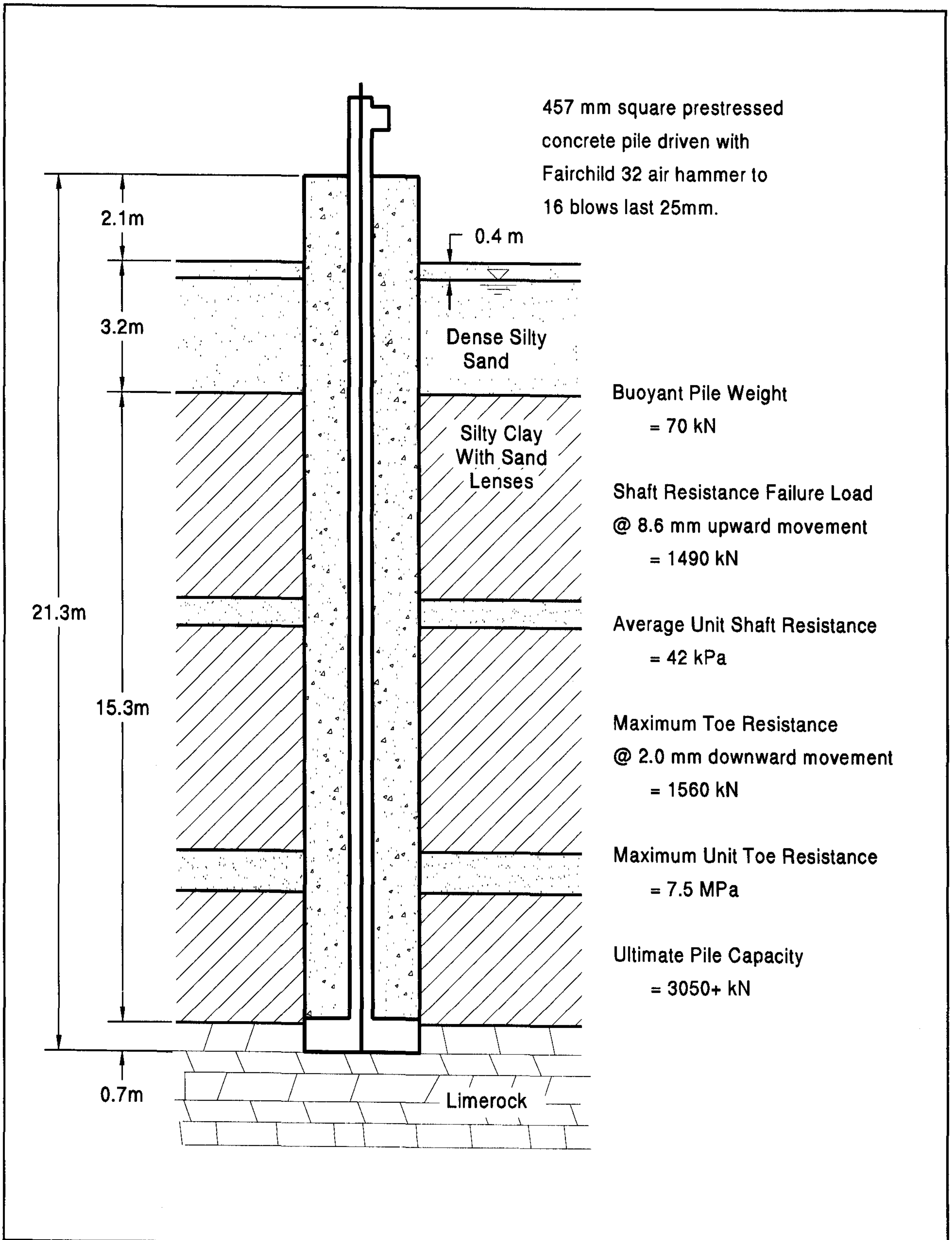


Figure 20.8 Summary of Subsurface Profile and Test Results at Aucilla River Bridge, FL

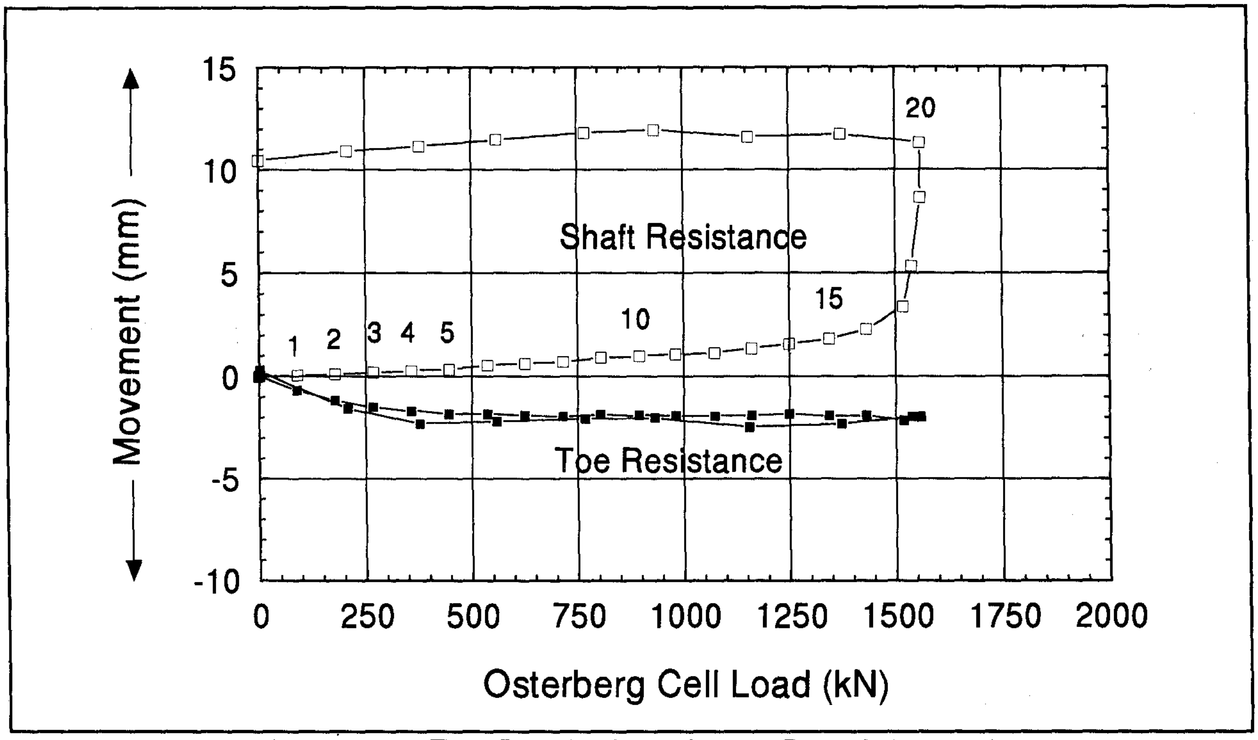


Figure 20.9 Test Results from Aucilla River Bridge, FL

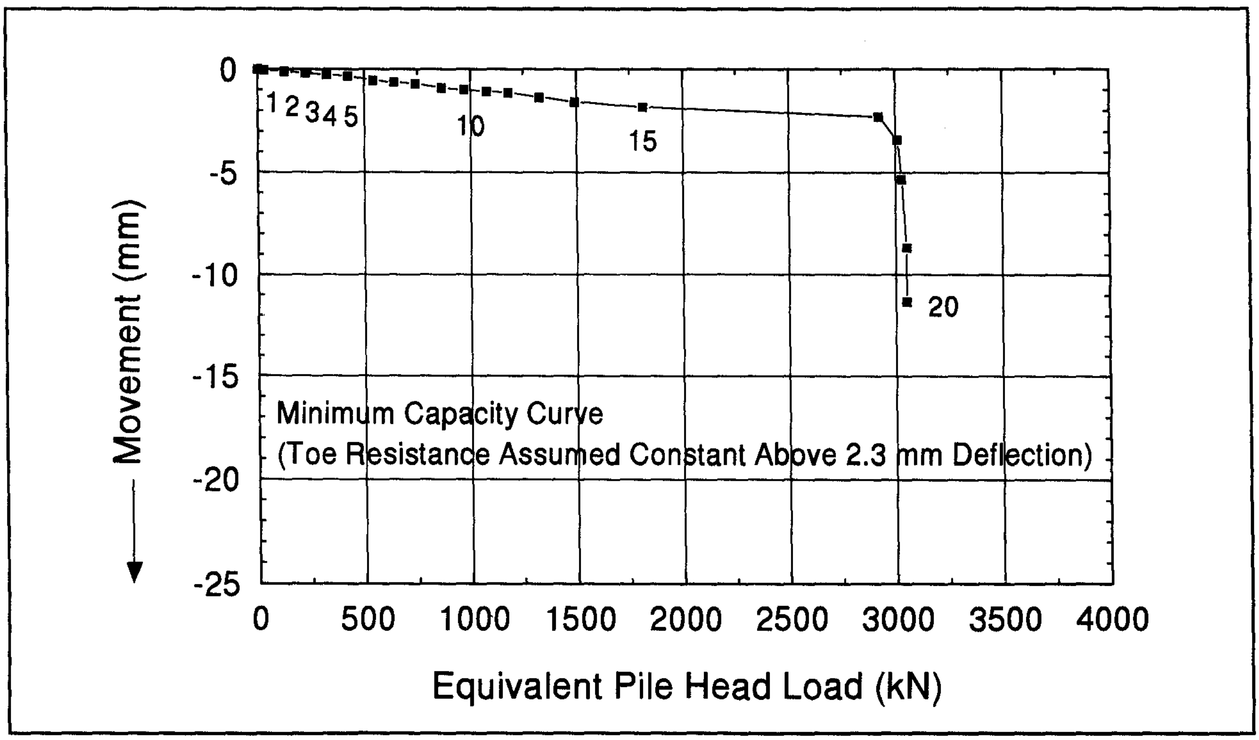


Figure 20.10 Equivalent Pile Head Load-Movement Curve from Aucilla River Bridge, FL

REFERENCES

American Society for Testing and Materials [ASTM], (1994). Annual Book of Standards. ASTM D-1143, Standard Test Method for Piles Under Axial Compressive Load, Vol. 4.08, Philadelphia, PA.

Goodwin, J.W. (1993). Bi-Directional Load Testing of Shafts to 6000 tons. Design and Performance of Deep Foundations: Piles and Piers in Soil and Soft Rock, ASCE, NY, NY, 204-217.

Osterberg, J.O. (1984). A New Simplified Method for Load Testing Drilled Shafts. Foundation Drilling, ADSC, Dallas, TX, Vol. 23, No. 6, 9-11.

Osterberg, J.O. (1994). Recent Advances in Load Testing Driven Piles and Drilled Shafts using the Osterberg Load Cell Method. Geotechnical Lecture Series, Geotechnical Division of the Illinois Section, ASCE, Chicago, IL.

Osterberg, J.O. (1995). The Osterberg Cell for Load Testing Drilled Shafts and Driven Piles. Report No. FHWA-SA-94-035, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 92.

Schmertmann, J.H. (1993). The Bottom-up, Osterberg Cell Method for Static Testing of Shafts and Piles. Progress in Geotechnical Engineering Practice Proceedings, Hershey PA, Central Pennsylvania ASCE, Harrisburg, PA, 7.

Thompson, D.E., Erikson, C.M. and Smith, J.E. (1989). Load Testing of Deep Foundations Using the Osterberg Cell. Proceedings of the 14th Annual Members Conference, Deep Foundations Institute, Sparta, NJ, 31-44.

21. THE STATNAMIC METHOD

A recent testing development for evaluation of driven pile capacity is the Statnamic testing method, Bermingham and Janes, (1989). The Statnamic test method uses solid fuel burned within a pressure chamber to rapidly accelerate upward the reaction mass positioned on the pile head. As the gas pressure increases, an upward force is exerted on the reaction mass, while an equal and opposite force pushes downward on the pile. Loading increases to a maximum and then unloads by a venting of the gas pressure. Built-in instrumentation (load cell, accelerometer, and laser sensor) measures load, acceleration and displacement. The Statnamic test method is not standardized by AASHTO or ASTM and is a proprietary method.

21.1 STATNAMIC BACKGROUND

The principles of Statnamic can be described by Newton's Laws of Motion:

1. A body will continue in a state of rest or uniform motion unless compelled to change by an external force.

2. A body subjected to an external force accelerates in the direction of the external force and the acceleration is proportional to the force magnitude ($F = ma$).

3. For every action there is an opposite and equal reaction ($F_{12} = -F_{21}$).

In the Statnamic test, a reaction mass is placed on top of the pile to be tested. The ignition and burning of the solid fuel creates a gas pressure force, F, that causes the reaction mass, m, to be propelled upward so that the acceleration amounts to about 20 g's ($F=ma$). An equivalent downward force is applied to the foundation element, ($F_{12} = -F_{21}$). The Statnamic concept is illustrated in Figure 21.1.

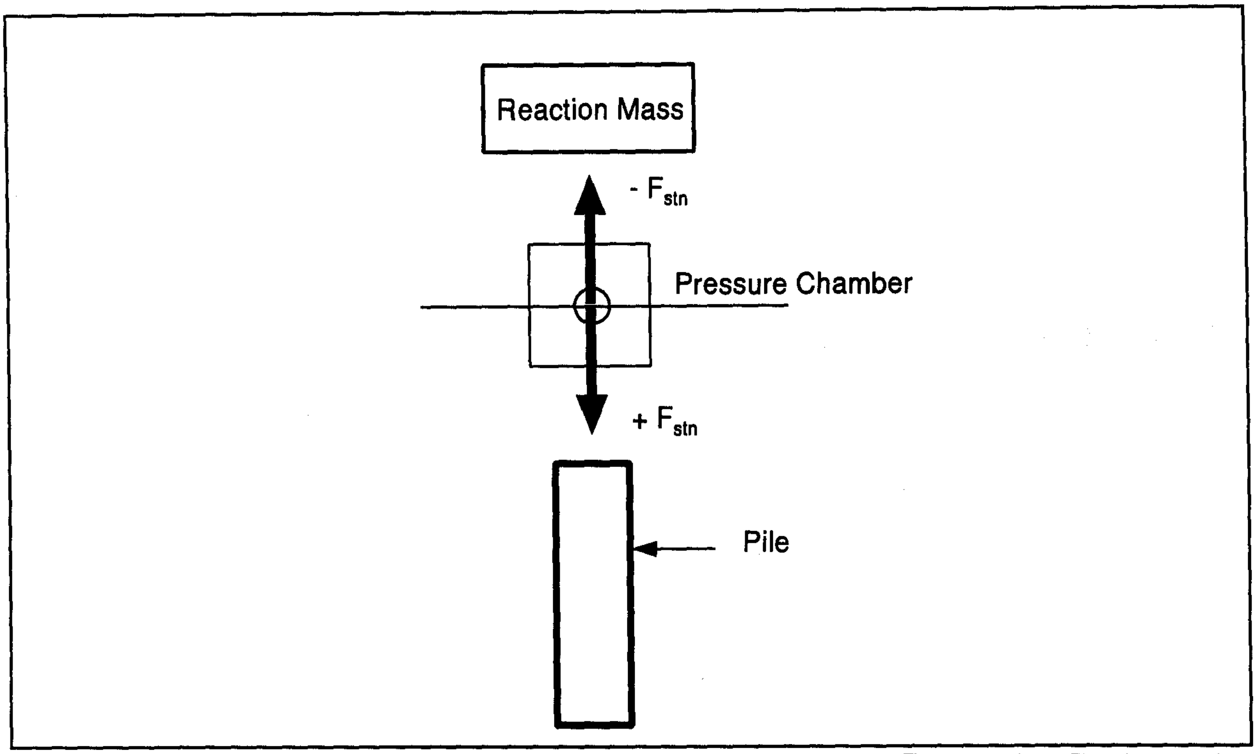


Figure 21.1 Statnamic Concept (courtesy of Berminghammer Foundation Equipment)

21.2 TEST EQUIPMENT

Development began in 1988 with a Statnamic device capable of a 100 kN test load. From 1988 through 1992, the test load capability was incrementally increased to 16,000 kN. In 1994, a 30,000 kN testing device was introduced.

The components of the Statnamic test equipment are shown in Figure 21.2 and a test in progress is shown in Figure 21.3. The base plate is attached to the pile head. The load cell, accelerometer, photo voltaic laser sensor, and piston base are positioned on top of the base plate. Next, the launching cylinder is placed on top of the piston base, thus enclosing the pressure chamber and propellant material. The reaction mass is then stacked on the launching cylinder and a retention structure is placed around the reaction mass. Finally, a sand or gravel backfill is placed in the annulus between the reaction mass and corrugated retention structure. After propellent ignition and reaction mass launch, the granular backfill slumps into the remaining void to cushion the reaction mass fall. Last, a remote laser reference source is positioned about 20 meters from the test apparatus.

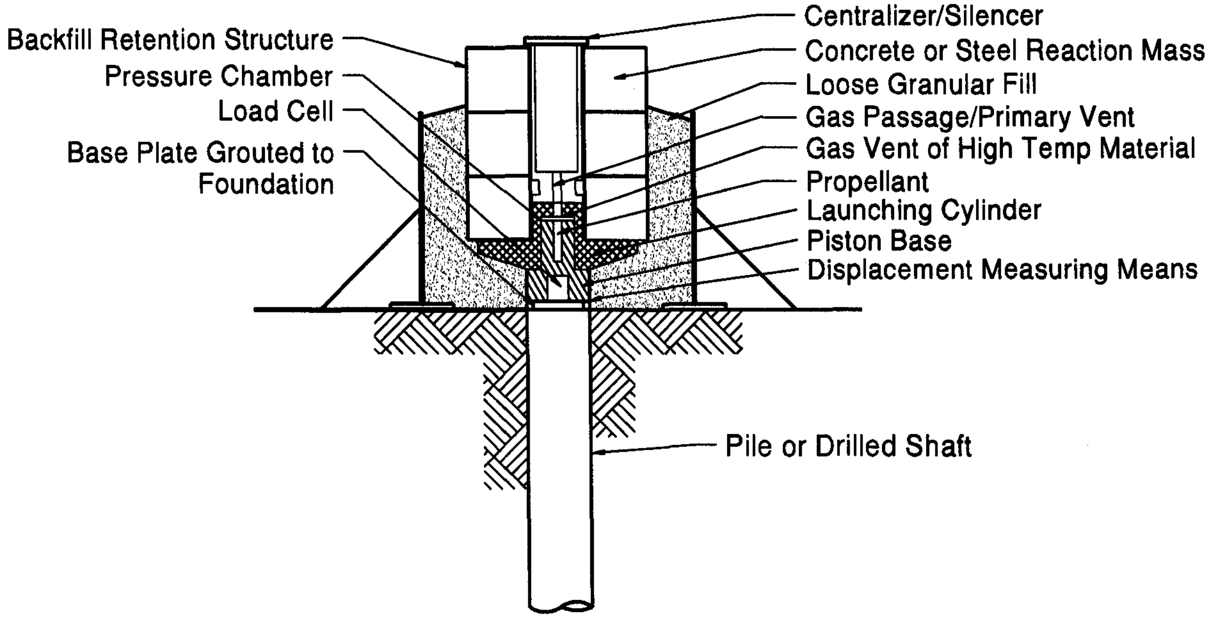


Figure 21.2 Schematic of Statnamic Loading System (after Bermingham and Janes, 1989)

The magnitude and duration of the applied load and the loading rate are controlled by the selection of piston and cylinder size, the fuel mass, the fuel type, the reaction mass, and the gas venting technique. The force applied to the pile is measured by the load cell. The acceleration of the pile head is monitored by the accelerometer and is integrated once to obtain pile head velocity and again to obtain displacement. Pile displacement relative to the reference laser source is measured with the photo voltaic laser sensor. Load and displacement data from the load cell and photo voltaic cell are

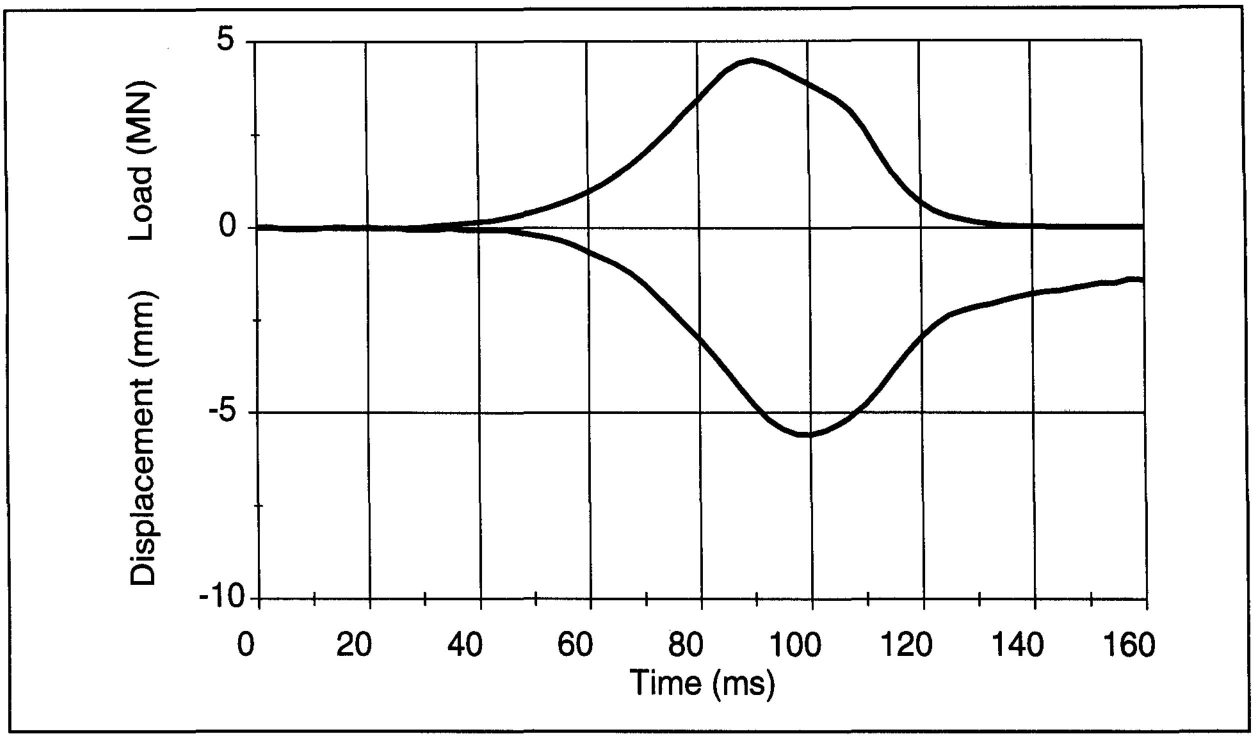


Figure 21.4 Measured Statnamic Signals (courtesy of Berminghammer Foundation Equipment)

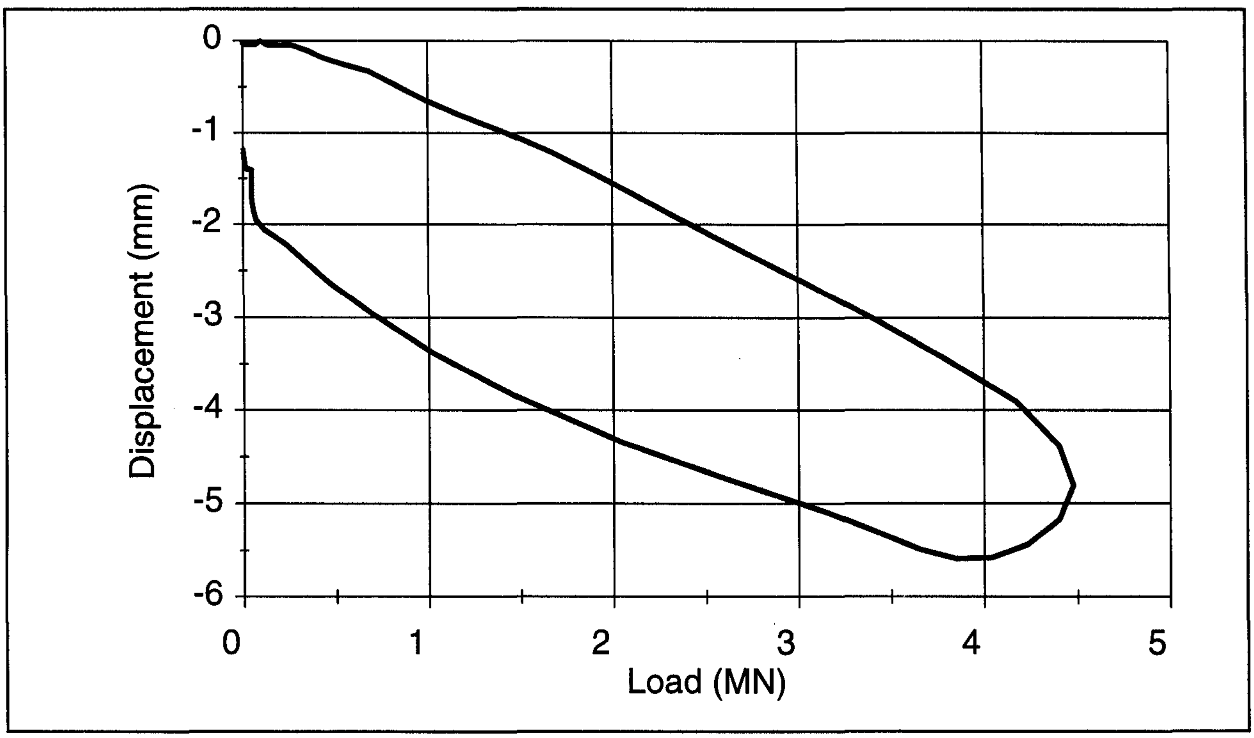


Figure 21.5 Load versus Displacement (courtesy of Berminghammer Foundation Equipment)

21.3 TEST INTERPRETATION

Initial correlations of Statnamic tests with static load tests for toe bearing piles founded in till and rock showed good agreement without adjustment of the Statnamic load - displacement results, Janes *et al.* (1991). However, later tests found that Statnamic can overpredict the ultimate pile capacity in some soils due to the dynamic loading effects. Middendorp *et al.* (1992) proposed an analysis procedure to adjust the raw Statnamic load - displacement results for dynamic loading rate effects, which is described below.

Because the duration of loading in a Statnamic test is about 100 ms, all elements of the pile move in the same direction and with almost the same velocity. According to the developers, this allows the pile to be treated as a rigid body undergoing translation. However, analytical studies by Brown (1995) have shown that this rigid body assumption can result in overpredictions of capacity and is not appropriate for long slender shafts or piles. The forces acting on the pile during a Statnamic test include the Statnamic induced load, F_{stn}, the pile inertia force, F_a, and the soil resistance forces which include the static soil resistance, F_u, the dynamic soil resistance, F_v, and the resistance from pore water pressure, F_p. A free body diagram of the forces acting on a pile during a Statnamic test is presented in Figure 21.6. The soil resistance forces shown in the free body diagram are distributed along the pile shaft as well as at the pile toe.

In mathematical terms, the force equilibrium on the pile may be described as follows:

$$F_{stn}(t) = F_a(t) + F_u(t) + F_v(t) + F_p(t)$$

This equation may be rewritten in terms of static soil resistance as follows:

$$F_u(t) = F_{stn}(t) - F_a(t) - F_v(t) - F_p(t)$$

A simplifying assumption is made that the pore water pressure resistance, F_p, can be treated as part of the damping resistance, F_v. This simplifies the above equation to:

$$F_u(t) = F_{stn}(t) - F_a(t) - F_v(t)$$

Consider the Statnamic load - displacement data presented in Figure 21.7, representative of a Statnamic load causing high dynamic loading effects. The Statnamic load - displacement data can be separated into five stages. Stage 1 includes the

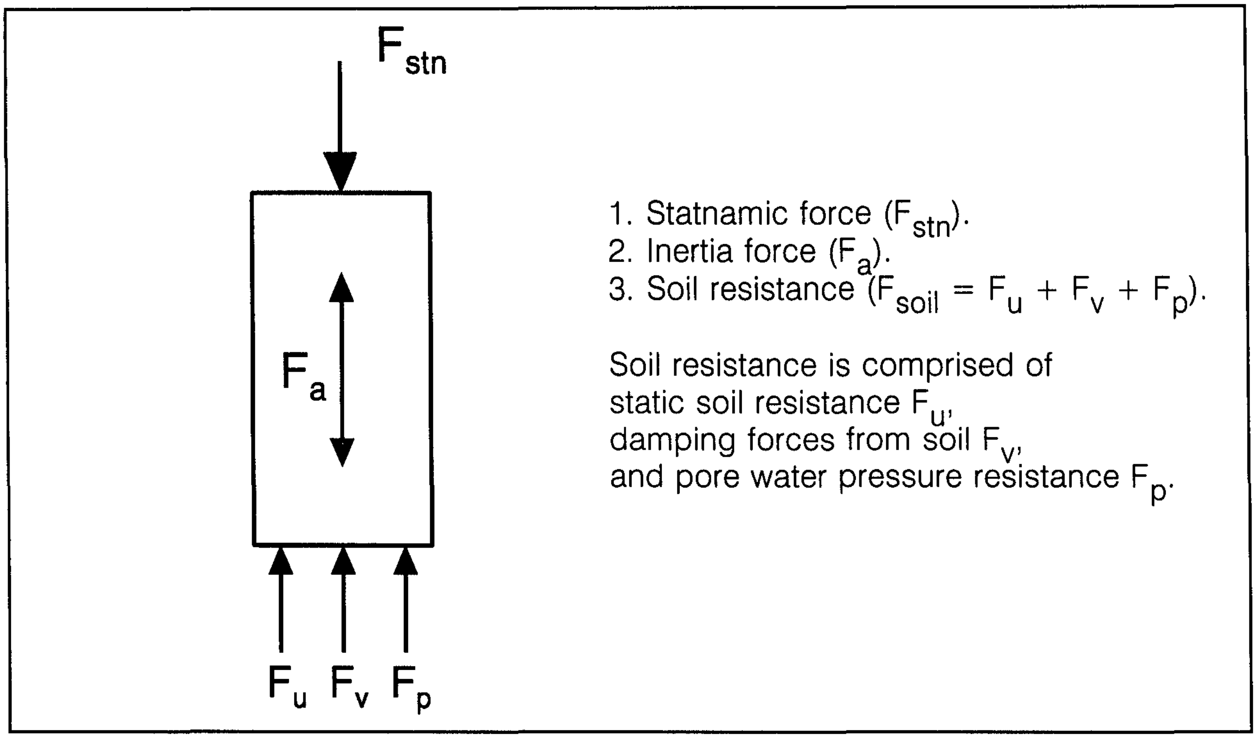


Figure 21.6 Free Body Diagram of Pile Forces in a Statnamic Test (after Middendorp *et al.* 1992)

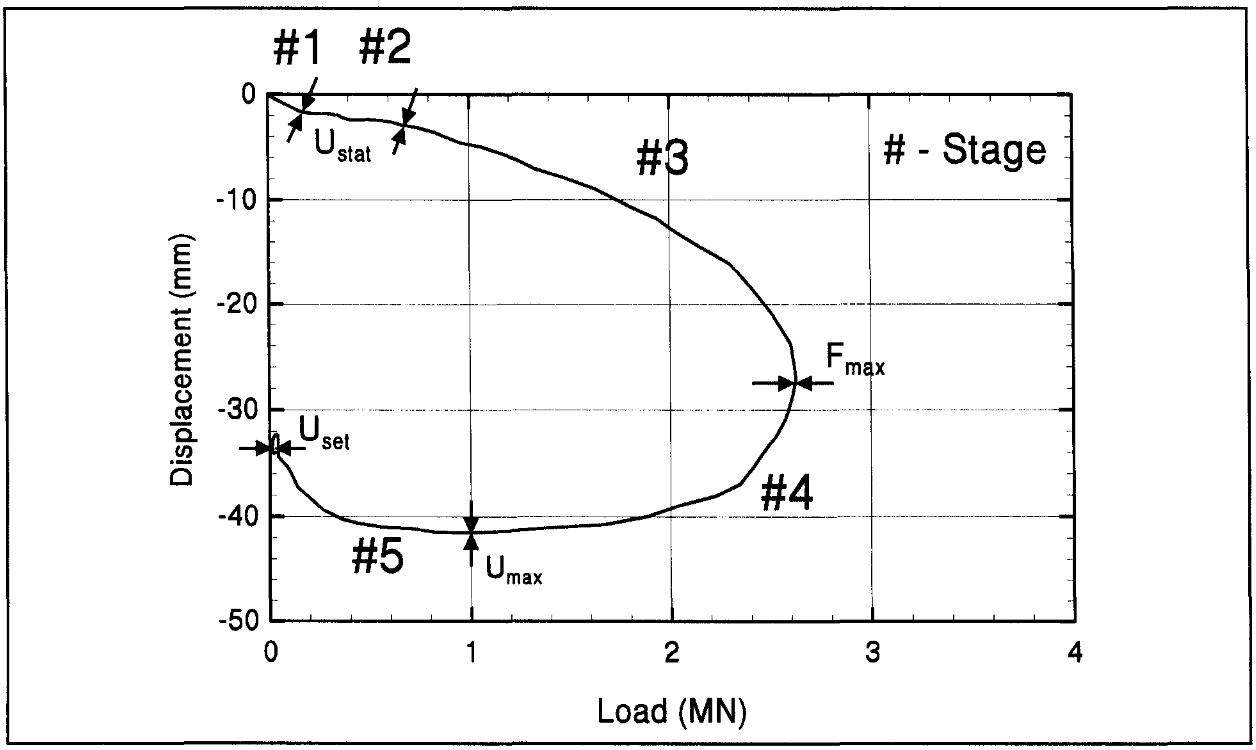


Figure 21.7 Five Stages of a Statnamic Test (after Middendorp *et al.* 1992)

assembling of the Statnamic piston and reaction mass and thus is a static loading phase. The reaction mass is launched and Stage 2 therefore provides the initial loading of the dynamic event. The soil resistance is treated as linearly elastic. Pile acceleration and velocity are small, resulting in low inertia and damping forces on the pile.

Stage 3 is the basic load application portion of the cycle with fuel burning and pressure in the combustion chamber. In Stage 3, significant nonlinear soil behavior occurs as the pile and soil experience high acceleration and velocity. Thus the highest inertia and damping forces are generated in this stage. The maximum Statnamic applied load is reached at the end of Stage 3.

In Stage 4 pressure in the combustion chamber is allowed to vent. Pile downward velocity and displacement continue but decrease throughout Stage 4. While the maximum Statnamic load is reached at the end of Stage 3, the maximum displacement occurs at the end of Stage 4. This is often due to the pile inertia force or significant dynamic resistance forces, $F_v(t)$ but may also occur in soils with strain softening (the residual soil resistance is significantly lower than the peak resistance). Since the pile velocity is zero at the point of maximum displacement, t_{umax}, the viscous damping, $F_v(t)$, on the pile is also zero at the end of Stage 4 and the static pile capacity may be expressed only at that time as:

$$F_u(t_{umax}) = F_{stn}(t_{umax}) - F_a(t_{umax})$$

In Stage 5, the soil rebounds from the loading event and to achieve final equilibrium the pile unloads and rebounds as load and movement cease. The displacement at the end of Stage 5 is the permanent displacement or set experienced under the test event.

The data processing system records the applied Statnamic load and pile head acceleration and displacement throughout the test. The ultimate static soil resistance, F_u, can then be calculated from the Statnamic load at the point of maximum displacement, $F_{stn}(t_{umax})$, minus the pile inertia force. This ultimate static soil resistance yields one point on the derived static load - displacement curve and may occur at a large displacement. If a limiting movement criterion such as described in Section 19.7.5 is used for load test interpretation, the ultimate pile capacity may be less than this ultimate static soil resistance.

To obtain the remaining points on the derived static load - displacement curve, the damping resistance, $F_{v,}$ at other load - displacement points must be determined. Assuming all damping is viscous (e.g., linear), then the damping resistance force can be expressed in terms of a damping constant, C_4, times the pile velocity at the corresponding time v(t). The pile velocity is obtained by differentiating the measured pile head displacement.

If the maximum applied Statnamic load is greater than the ultimate pile capacity, then the soil resistance at the beginning of Stage 4 through the point of maximum displacement at the end of Stage 4 will be a constant and will be equal to $F_u(t_{max})$, assuming the soil is perfectly plastic and does not exhibit strain hardening. The damping constant, C_4, may be calculated from the maximum Statnamic load at the beginning of Stage 4, t_4. This may be expressed as:

$$C_4 = [\ F_{stn}(t_4) - F_u(t_{umax}) - ma(t_4)\ ]\ /\ v(t_4)$$

Assuming the damping constant, C_4, is constant throughout the Statnamic loading event, the derived static load may be calculated at any point in time from:

$$F_u(t) = F_{stn}(t) - ma(t) - Cv(t)$$

The derived Statnamic load - displacement curve is then constructed using the above equation and corresponding pile head displacement. An example of the derived load-displacement curve illustrating how the dynamic rate effects are subtracted from the new Statnamic results is presented in Figure 21.8.

21.4 APPLICATIONS

Statnamic tests for evaluation of static pile capacity have been performed on steel, concrete and timber piles. Individual piles or pile groups with a combined static and dynamic resistance less than 30,000 kN can be tested. Axial compressive capacity tests have been conducted on both vertical and battered piles. The test method has been used on land and over water.

Recently, the feasibility of using the Statnamic method to conduct lateral load test has begun to be explored, Berminghammer (1994). However, significant research work

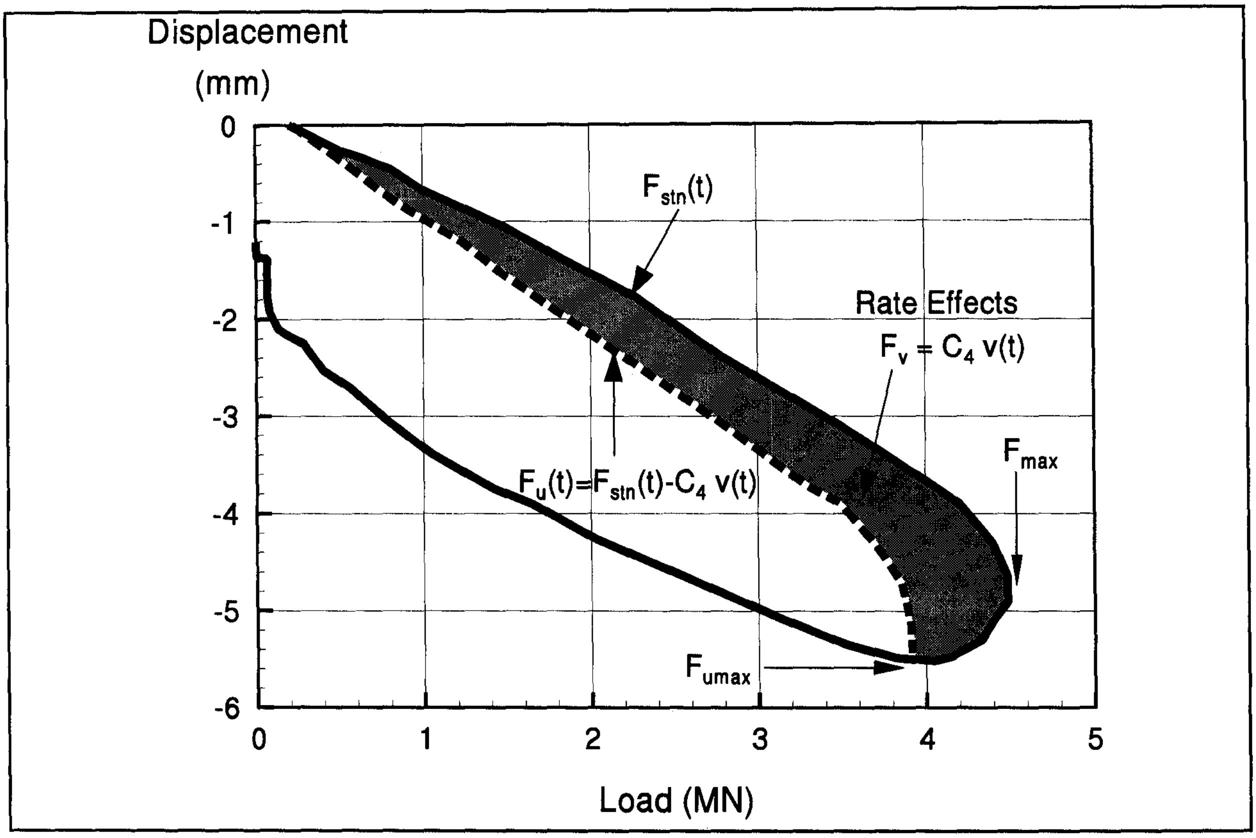


Figure 21.8 Derived Statnamic Load-Displacement Curve With Rate Effects (courtesy of Berminghammer Foundation Equipment)

remains to be done for this potential application. In September 1995, the FHWA granted partial funding to the Alabama DOT to conduct a series of lateral Statnamic tests to simulate vessel impact loading to further study this application. The Statnamic test for lateral load application is also being studied in the current NCHRP research project 24-09, Static and Dynamic Lateral Loading of Pile Groups. A lateral Statnamic test on a nine pile group is shown in Figure 21.9. The maximum lateral load applied to date in a Statnamic test is 7320 kN. However, this is not a limit of the Statnamic test device but rather of the pile group response.

21.5 CASE HISTORIES

Statnamic test results for two cases are presented with comparisons to static load test results. The first case involves an 18.6 meter long HP 310 x 110 H-pile driven through gravelly clay and into sandstone in Pittsburgh, Pennsylvania. As indicated in the results

Figure 21.9 Lateral Statnamic Test on Nine Pile Group (courtesy of Utah State University)

shown in Figure 21.10 the dynamic resistance appears low and the agreement between the maximum Statnamic load, the Statnamic unloading point, and the static load test maximum capacity appears to be good. It should be noted that the Statnamic and static test results are from piles nearby, but not the same piles. Unfortunately, neither the static test nor the Statnamic test loaded the pile to a traditional failure load based on the measured displacements and loads, and therefore the ultimate static load is not determined from either test method. Hence, this case is more of a comparison in load deflection behavior than a correlation case of ultimate pile capacity from the two test methods.

The second study presents a correlation case in which the soils have a significant dynamic resistance. This case was for the I-280 test program in San Francisco, California and involved a 406 mm diameter, 32.2 meter long, closed-ended pipe pile in very soft bay mud. Test results presented in Figure 21.11 illustrate that the maximum Statnamic load is 3.3 MN or 2.5 times greater than the maximum static load test

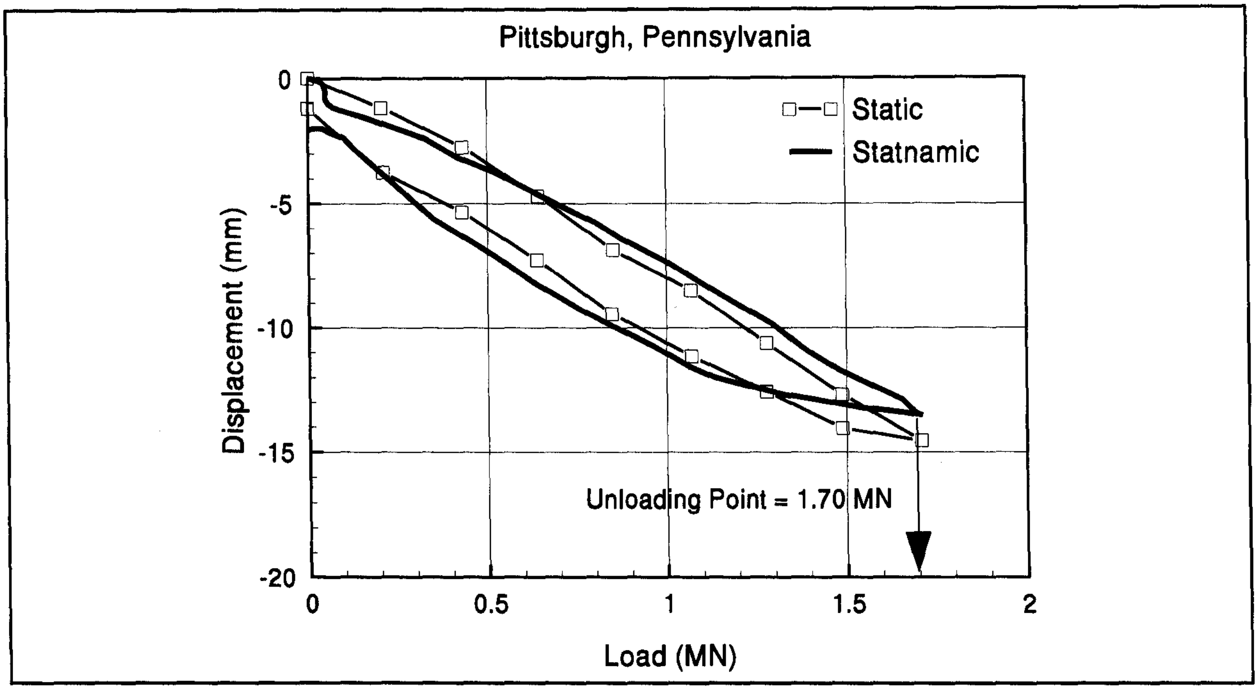


Figure 21.10 Static Load Test and Statnamic Comparison from Pittsburgh Site (courtesy of Berminghammer Foundation Equipment)

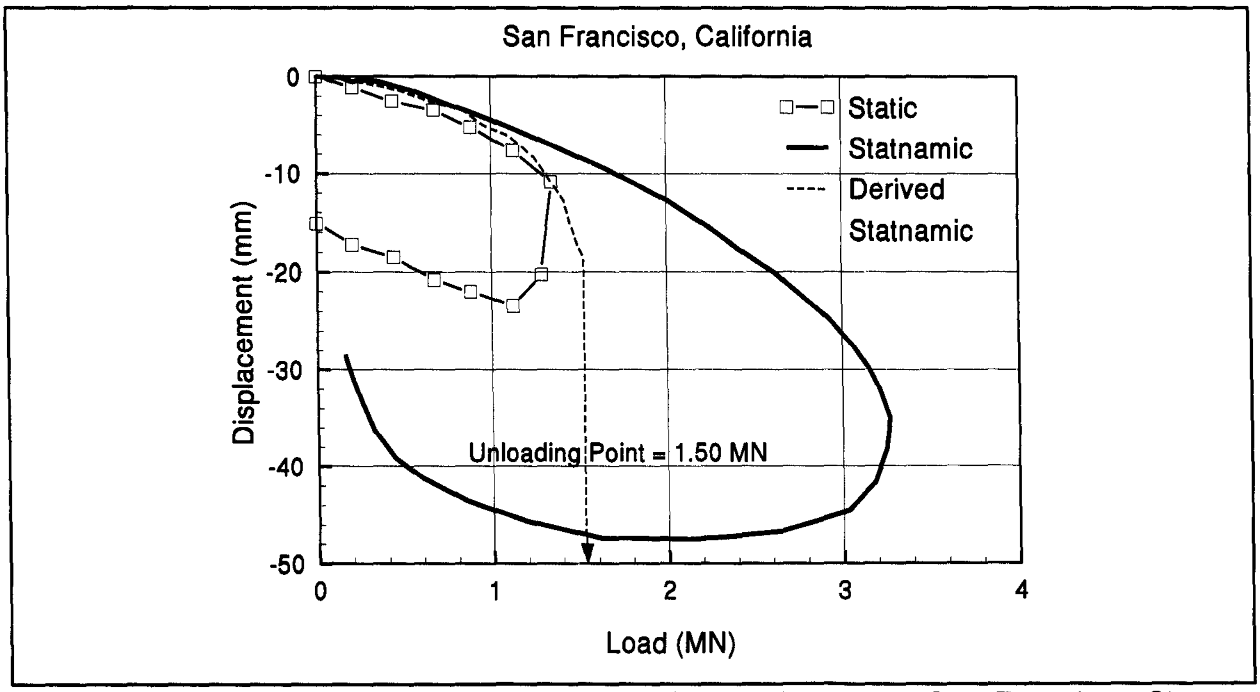


Figure 21.11 Static Load Test and Statnamic Comparison from San Francisco Site (courtesy of Berminghammer Foundation Equipment)

capacity of 1.3 MN. The maximum displacement occurs at a load of 2.1 MN or 1.6 times greater than the maximum static load capacity. The Statnamic capacity determined using the unloading point method is 1.5 MN or 1.15 times the reported ultimate static pile capacity. Additional information on this test may be found in Berkovitz and Hahn (1995).

21.6 ADVANTAGES

Advantages of Statnamic testing include lower cost, shorter test time, and mobility. Depending upon the magnitude of load, the site location, and labor costs, the cost of a Statnamic test is on the order of one quarter to one half the cost of an equivalent capacity static load test. Savings may increase for higher pile capacities or for multiple tests performed.

Once Statnamic is mobilized to a site, one or two tests can typically be performed in one day. The design of a segmental reaction mass allows assembly with relatively small hoisting equipment. In addition, since the reaction mass is typically 5 to 10 percent of the applied load, movement around a site for multiple tests is easier than for a static test using dead weight. Recent equipment advances include a hydraulic catch mechanism to replace the gravel retention structure. This mechanism, shown in Figure 21.12, permits a higher number of tests to be conducted per day.

Applied pile head load and displacement are measured by load cell and photo voltaic laser. The laser eliminates problems with measuring displacement from required reference beams during a static test, although the laser source can be sensitive to ground vibrations. The load, acceleration, and displacement readings are digitized 4000 samples per second.

The Statnamic method is a simple concept governed by Newtonian principles.

21.7 DISADVANTAGES

Some of the disadvantages of Statnamic testing may be attributed to its recent and continuing development. Correlations with conventional static tests are still being obtained and refinement of the analysis procedure is expected to continue. Both Janes

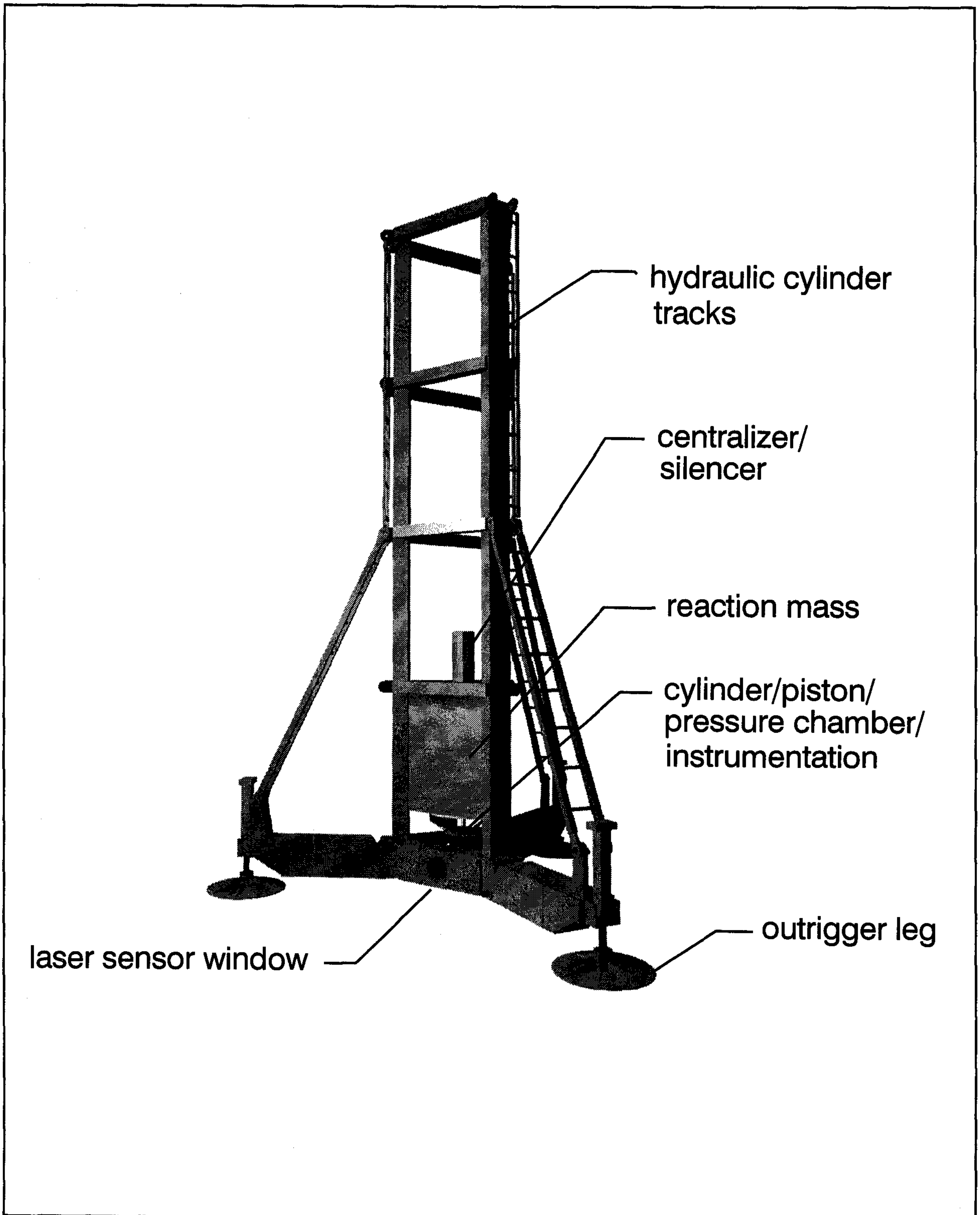


Figure 21.12 Statnamic Hydraulic Catch Mechanism (courtesy of Berminghammer Foundation Equipment)

et al (1994) and Brown (1995) have recommended additional Statnamic - static load testing correlations be obtained to enhance the data base and to improve interpretive procedures. In addition, the interpretation method is sufficiently complicated that it is difficult to independently check the proprietary result. The instrumentation is also complex and lacks the redundant checks available in conventional static or dynamic testing to verify the calibration accuracy.

Middendorp *et al.* (1992) noted that the 100 ms duration of loading is short enough that Statnamic is still a dynamic test. Hence, adjustment of the raw field results for dynamic phenomena is required. When proposing the current analysis method, Middendorp *et al.* (1992), suggested that the unloading point method allows direct calculation of the maximum static soil resistance when pore pressures play a minor role. He further concluded that a method to determine the pore pressure effect and make a correction for it in a Statnamic test has to be developed.

A subsequent Statnamic test reported by Matsumoto *et al.* (1994), instrumented with pore pressure transducers, measured Statnamic induced pore pressures near the pile toe on the order of 0.4 MPa for a pile founded in mudstone. This resulted in a reduction of the Statnamic unloading point capacity by about 4%.

To assure that the ultimate pile capacity has been achieved, a significant permanent pile set at the conclusion of a Statnamic test must be achieved. This often requires the applied Statnamic force to be larger than the combined ultimate static and dynamic soil resistances. If the Statnamic test does not cause soil failure and a significant permanent set, then an overprediction of static capacity may occur (Janes 1994). For example, if the Statnamic test in Figure 21.11 had only been loaded to 2.5 MN, a Statnamic load-displacement curve similar to the Statnamic result shown in Figure 21.8 would likely be obtained. This type of load-displacement result in this soil condition would make determination and subtraction of the dynamic rate effects difficult and increase the probability of static capacity overprediction. Additional discussion of Statnamic - load test correlations may be found in Brown (1994), Brown (1995) and Goble *et al.* (1995).

A summary of FHWA recent experience with Statnamic testing may be found in Berkovitz and Hahn (1995). Until the Statnamic interpretation procedures have been modified to fully account for inertia, damping and pore pressure effects, the FHWA recommends the Statnamic test be accompanied by a correlating static test.

REFERENCES

Berkovitz, B.C., and Hahn, M. (1995). A Review of Statnamic Experiences for Transportation Applications. FHWA Regions 4 and 6 Design and Field Engineers Conference, 17.

Bermingham, P. and Janes, M. (1989). An Innovative Approach to Load Testing of High Capacity Piles. Proceedings of the International Conference on Piling and Deep Foundations, Volume 1, J.B Burland and J.M. Mitchell Editors, A.A. Balkema Publishers, Rotterdam, 409-413.

Berminghammer Corporation, Ltd. (1994). Statnamic Newsletter, Volume 2, Number 1.

Brown, D.A. (1994). Evaluation of Static Capacity of Deep Foundations from Statnamic Testing. ASTM, Geotechnical Testing Journal, GTJODJ, Vol. 17, No. 4, 403-414.

Brown, D.A. (1995). Closure - Evaluation of Static Capacity of Deep Foundations from Statnamic Testing. ASTM, Geotechnical Testing Journal, GTJODJ, Vol. 18, No. 4, 495-498.

Goble, G.G., Rausche, F. and Likins, G. (1995). Discussion of Evaluation of Static Capacity of Deep Foundations from Statnamic Testing. ASTM, Geotechnical Testing Journal, GTJODJ, Vol. 18, No. 4, 493-495.

Janes, M., Sy, A. and Campanella, R.G. (1994). A Comparison of Statnamic and Static Load Tests on Steel Pipe Piles in the Fraser Delta. Deep Foundations. Proceedings of the 8th Annual Vancouver Geotechnical Society Symposium, Vancouver, 1-17.

Janes, M., Bermingham, P. and Horvath, B. (1991). Pile Load Test Results Using the Statnamic Method. Proceedings of the 4th International Conference on Piling and Deep Foundations, Vol. 1, Deep Foundations Institute, Editor, A.A Balkema Publishers, Rotterdam, 481-489.

Janes, M. (1995). Statnamic Load Testing of Bridge Pier Foundations in North America. Proceedings of the 31st First Symposium on Engineering Geology and Geotechnical Engineers, Utah State University, Logan, Utah.

Matsumoto, T., Tsuzuki, M. and Michi, Y. (1994). Comparative Study of Static Loading Test and Statnamic On a Steel Pipe Pile Driven in Soft Rock. Proceedings of the 5th International Conference and Exhibition on Piling and Deep Foundations, Deep Foundations Institute, Bruges, 5.3.1-5.3.7.

Middendorp, P., Bermingham, P. and Kuiper, B. (1992). Statnamic Load Testing of Foundation Piles. Proceedings of the 4th International Conference on the Application of Stress-Wave Theory to Piles, A.A. Balkema Publishers, Rotterdam, 581-588.

22. PILE DRIVING EQUIPMENT

The task of successfully installing piles involves selecting the most cost-effective equipment to drive each pile to its specified depth without damage in the least amount of time. The pile driving system is also used as a measuring instrument to evaluate driving resistance. Therefore, the challenge to both the engineer and the pile contractor becomes one of knowing, or learning about, the most suitable equipment for a given set of site conditions, and then confirming that the driving system is operating properly.

Figures 22.1 and 22.2 show the components of a typical driving system. The crane, leads, hammer and helmet are the primary components of any driving system. Followers and equipment for jetting, predrilling, and spudding, may be permitted under certain circumstances for successful pile driving. This chapter presents a basic description of each component of a driving system. For additional guidance, readers are referred to pile driving equipment manufacturer's and suppliers.

22.1 LEADS

The function of a set of leads is to maintain alignment of the hammer-pile system so that a truly concentric blow is delivered to the pile for each impact. Figures 22.1 through 22.4 show several lead systems used for pile driving. Figure 22.5 shows various lead types. The box lead is the most versatile lead and its use allows all the configurations shown in Figures 22.1 through 22.4.

Swinging leads, illustrated in Figure 22.1, are widely used because of their simplicity, lightness and low cost. The most common arrangement is shown in Figure 22.1(b) where the lead and hammer are held by separate crane lines. The leads can also be hung from the boom with hanger straps as illustrated in Figure 22.1(a) with the hammer held by a crane line. Swinging leads are free to rotate sufficiently to align the hammer and the head of the pile without precise alignment of the crane with the pile head. Because the weight of the leads is low, this type of lead generally permits the largest crane operating radius, providing more site coverage from one crane position.

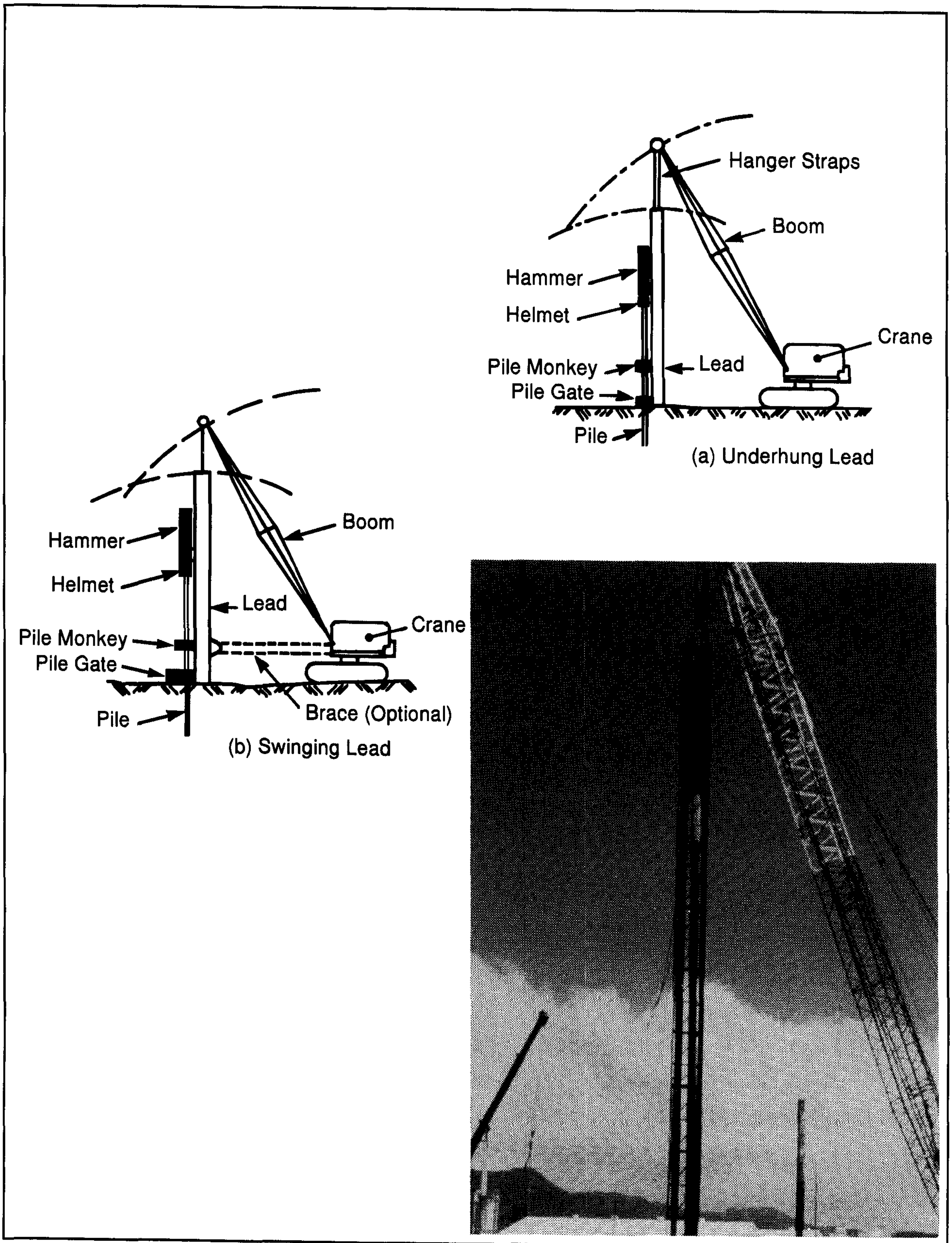
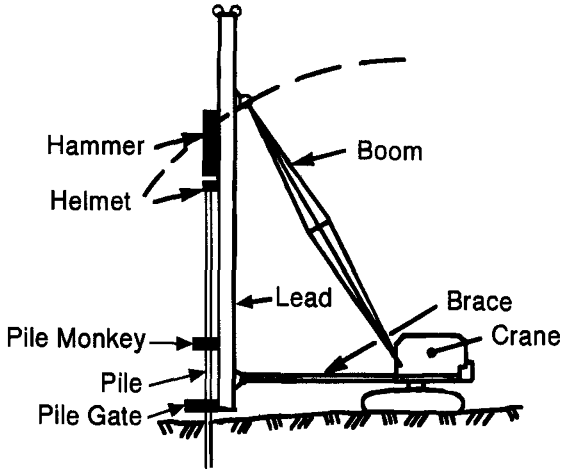
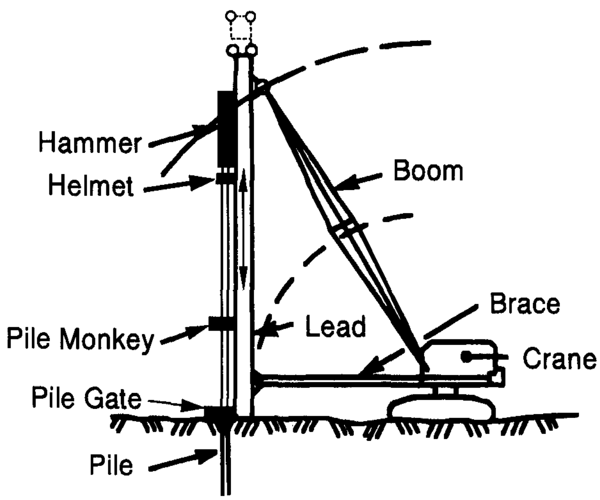


Figure 22.1 Swinging Lead Systems (after D.F.I. Publication, 1981)



(a) Fixed or Extended with Brace



(b) Semifixed or Vertical Travel with Brace

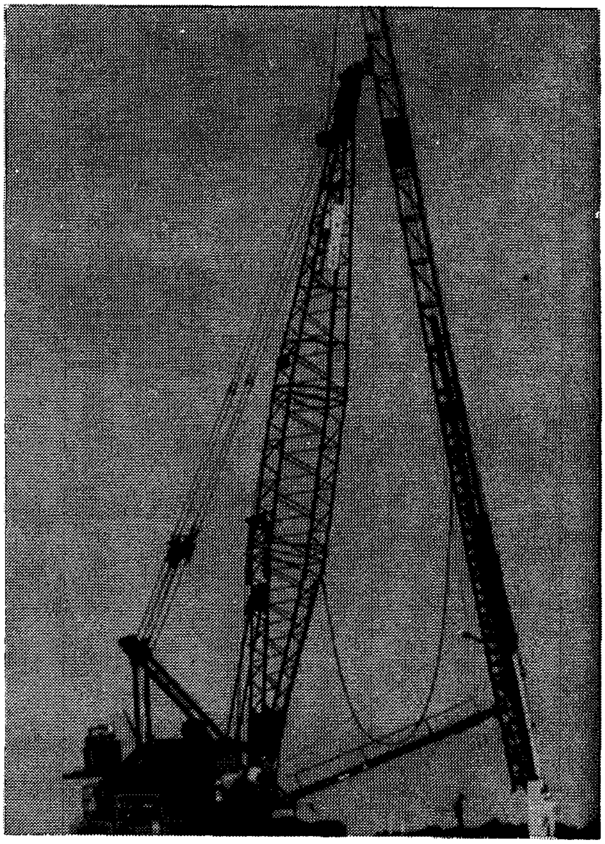


Figure 22.2 Fixed Lead Systems (after D.F.I. Publication, 1981)

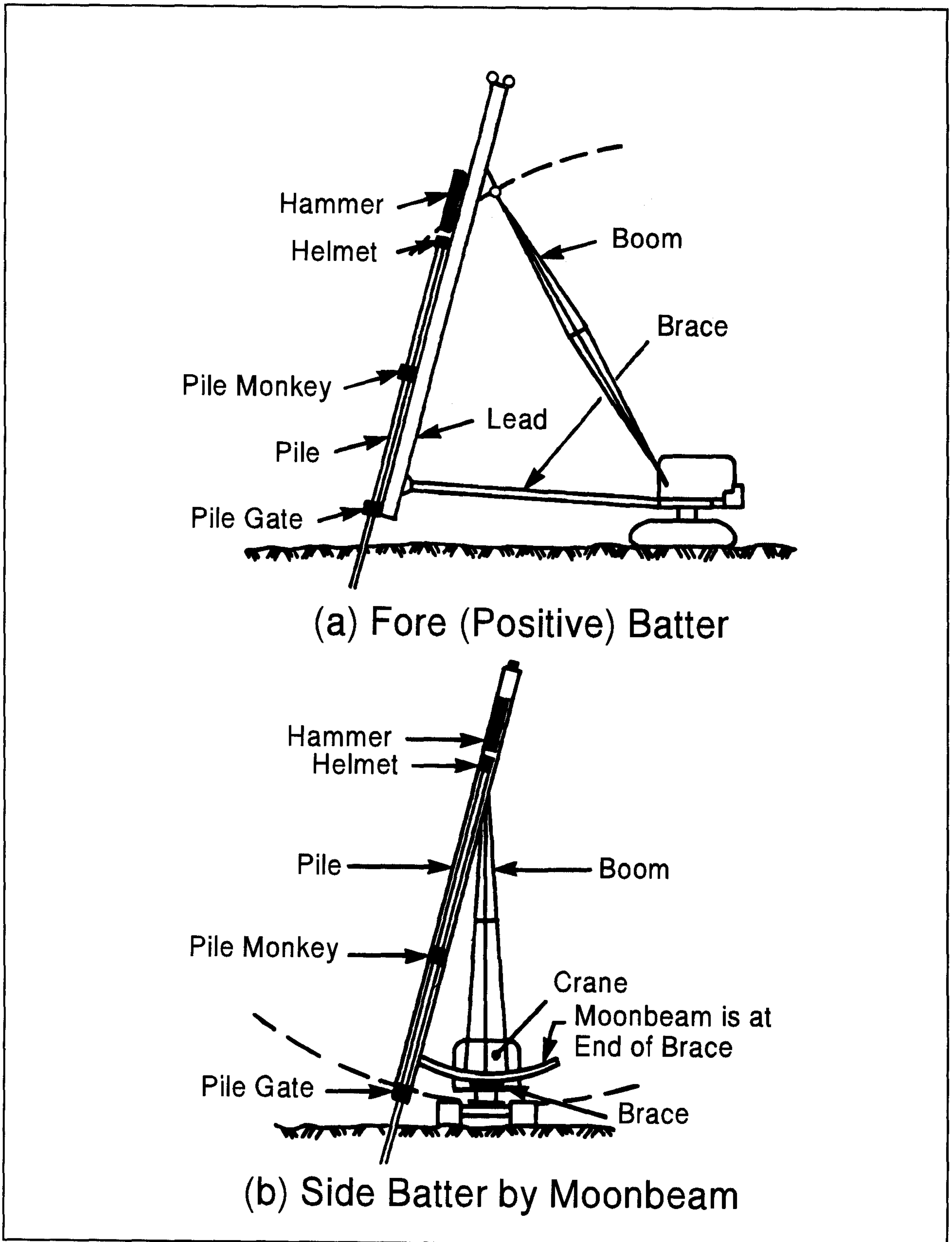


Figure 22.3 Lead Configurations for Batter Piles (after D.F.I. Publication, 1981)

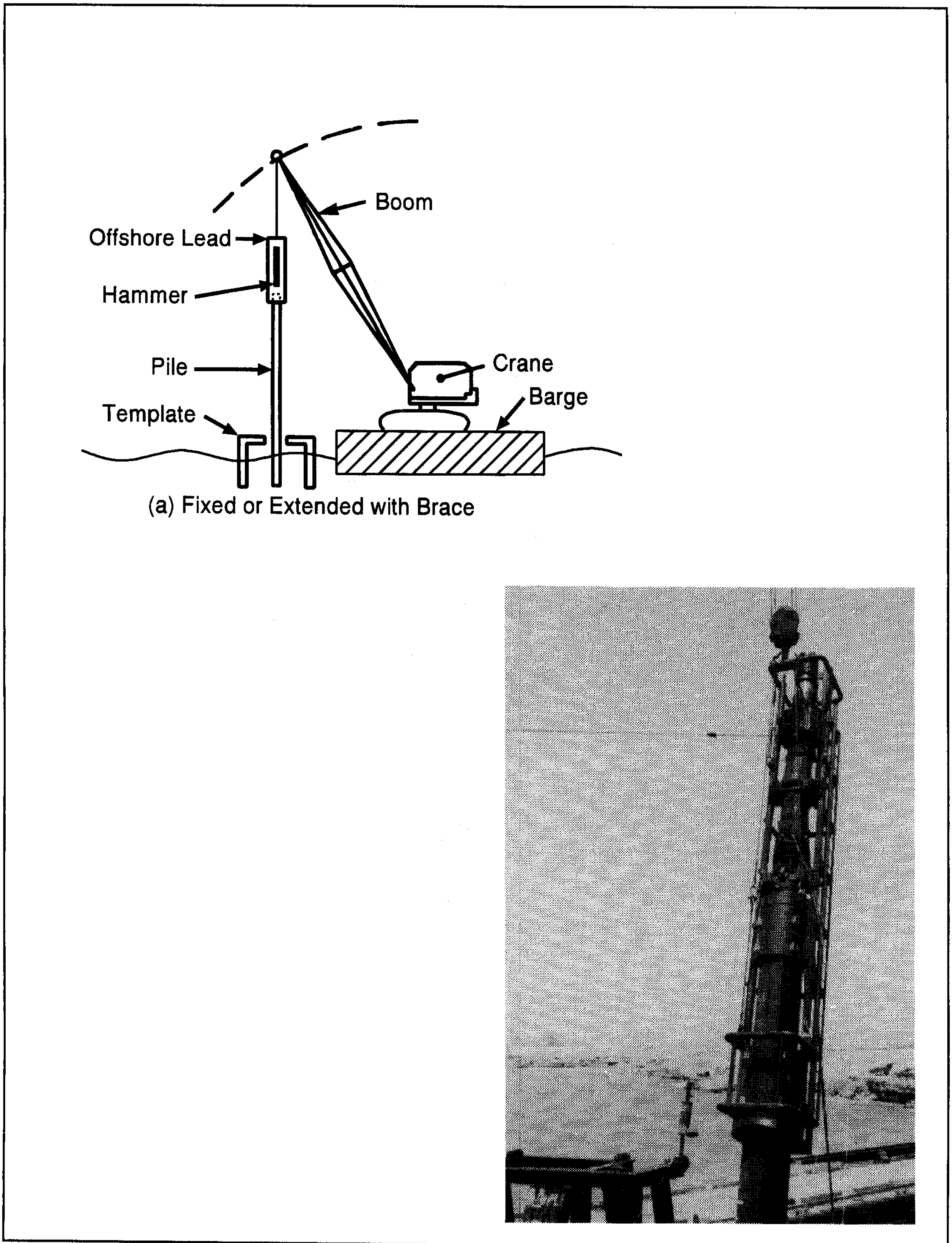


Figure 22.4 Typical Offshore Lead Configuration

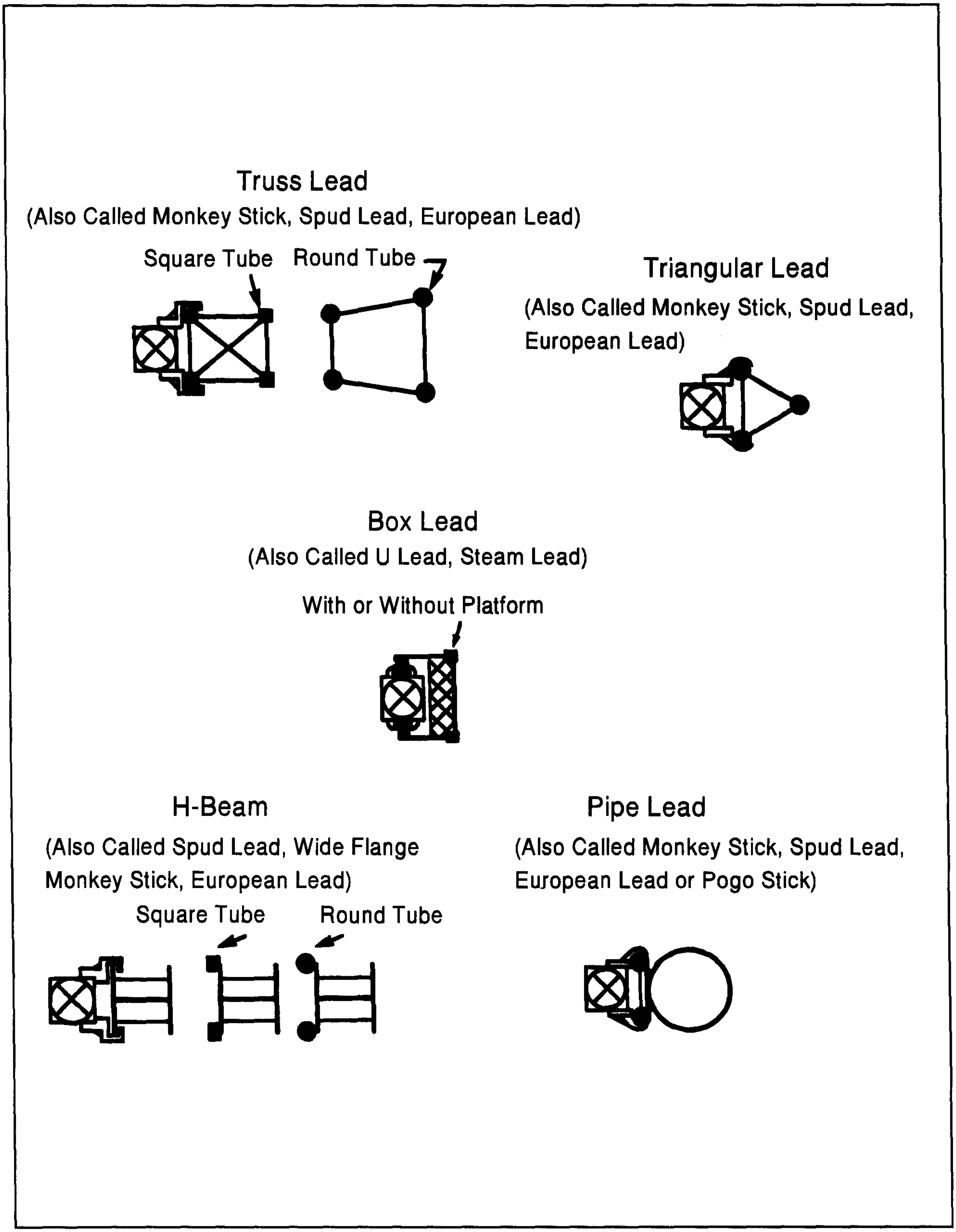


Figure 22.5 Typical Lead Types (after D.F.I. Publication, 1981)

Standard fixed leads shown in Figure 22.2 are slung from the boom point with a brace running from the bottom of the leads to the crane cab frame. A schematic of a typical fixed lead system is depicted in Figure 22.2(a). A variation of a fixed lead system is a semi-fixed or vertical travel lead as shown in Figure 22.2(b). The semi-fixed lead allows vertical lead movement at the lead connection points to the boom and brace which the standard fixed lead system does not. Figure 22.3(a) illustrates that a fixed lead is limited to plumb piles or batter piles in line with the leads and crane boom. To drive side batter piles, a moonbeam must be attached at the end of the brace as depicted in Figure 22.3(b). A fixed lead attempts to hold the pile in true alignment while driving but may require more set up time.

Offshore leads shown in Figure 22.4 are similar to swinging leads in that they are free to rotate sufficiently to align the hammer and head of the pile without precise alignment of the crane with the pile head. They generally consist of a short lead section of sufficient length to hold the hammer and axially align the hammer with the pile head. Offshore leads are used with a template that holds the pile in place.

Pile driving specifications have historically penalized or prohibited swinging leads. This general attitude is not justified based on currently available equipment. In fact, there are many cases where swinging leads are more desirable than fixed leads. For example, swinging leads are preferable for pile installation in excavations or over water. The function of a lead is to hold the pile in good alignment with the driving system in order to prevent damage, and to hold the pile in its proper position for driving. If a swinging lead is long enough so that the bottom is firmly embedded in the ground, and if the bottom of the lead is equipped with a gate, then bottom alignment of the pile will be maintained. In this situation, if the pile begins to move out of position during driving, it must move the bottom of the lead with it. Swinging leads should be of sufficient length so that the free line between the boom tip and the top of the leads is short, thus holding the top of the lead in good alignment. When batter piles are driven, pile alignment is more difficult to set with swinging leads. This problem is accentuated for diesel hammers since the hammer starting operation will tend to pull the pile out of line.

Regardless of lead type chosen, the pile must be kept in good alignment with the hammer to avoid eccentric impacts which could cause local stress concentrations and pile damage. The hammer and helmet, centered in the leads and on the pile head, keep the pile head in alignment. A pile gate at the bottom of the leads should be used to keep the lower portion of the pile centered in the leads.

22.2 TEMPLATES

Templates are required to hold piles in proper position and alignment when an offshore type or swinging lead system is used over water. The top of the template should be located within 1.5 m of the pile cutoff elevation or the water elevation, whichever is lower. The preferred elevation of the template is at or below the pile cutoff elevation so that final driving can occur without stopping for template removal. A photograph of a typical template is presented in Figure 22.6.

When positioning templates that include batter piles, it must be remembered that the correct template position of batter piles will vary depending upon the template elevation relative to the pile cutoff elevation. For example, consider a template located 1.5 meters above pile cutoff elevation. If the plan pile locations at cutoff are used at the template elevation, a 1H:4V batter pile would be 375 mm out of location at the pile cutoff elevation. This problem is illustrated in Figure 22.7. Template construction should also allow the pile to pass freely through the template without binding. Templates with rollers are preferable, particularly for batter piles.

22.3 HELMETS

Figure 22.8 shows the components of a typical helmet (also called a drive cap) and the nomenclature used for these components. The helmet configuration and size used depends upon the lead type, pile type and the type of hammer used for driving. Details on the proper helmet for a particular hammer can be obtained from hammer manufacturers, suppliers and contractors. To avoid the transmission of torsion or bending forces, the helmet should fit loosely, but not so loosely as to prevent the proper alignment of hammer and pile. Helmets should be approximately 2 to 5 mm larger than the pile diameter. Proper hammer-pile alignment is particularly critical for precast concrete piles. Figure 22.9 shows a helmet for a steel H-pile.

Most hammers use a hammer cushion between the hammer and the helmet to relieve the impact shock, thus protecting the pile hammer. However, some hammer models exist that do not require a hammer cushion, or utilize a direct drive option where the hammer cushion is replaced by a steel striker plate. Ineffective hammer cushions in hammers requiring a cushion can cause damage to hammer striking parts, anvil, helmet or pile. All cushion materials become compressed and stiffen as additional hammer impacts are applied. Therefore, hammer cushions eventually become ineffective, or may

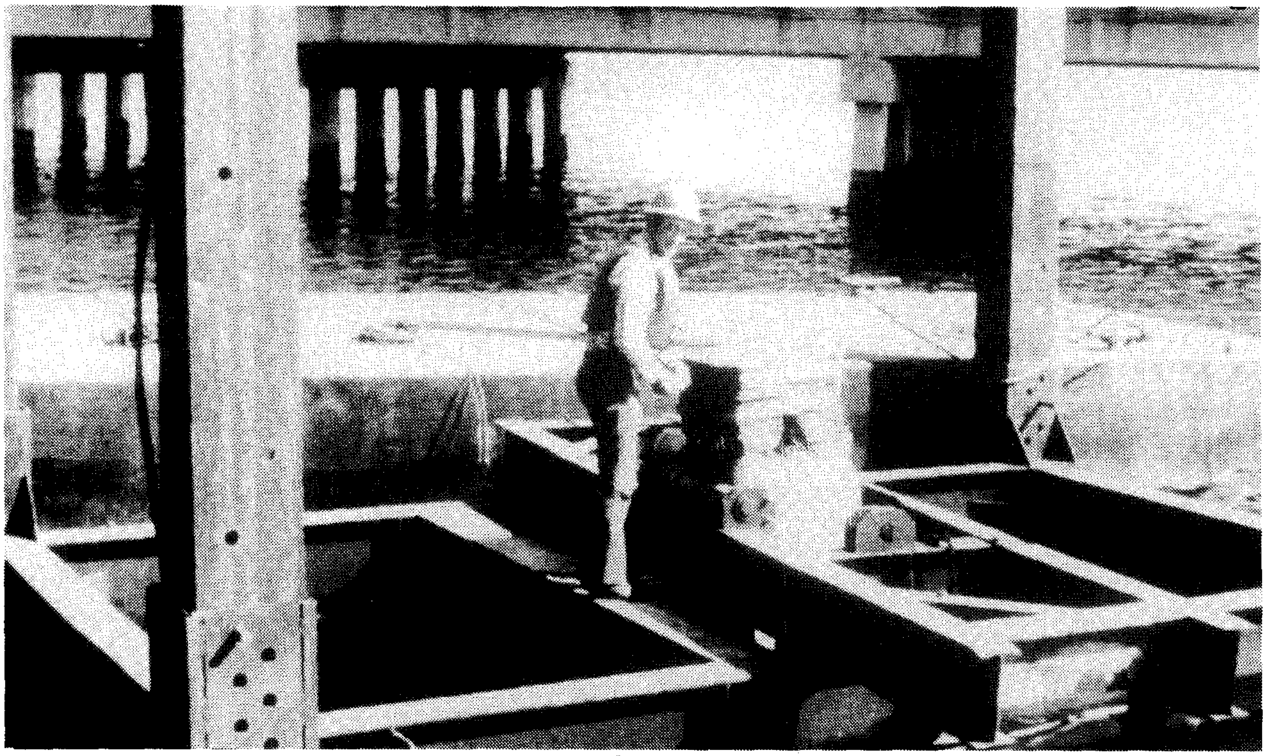


Figure 22.6 Typical Template Arrangement

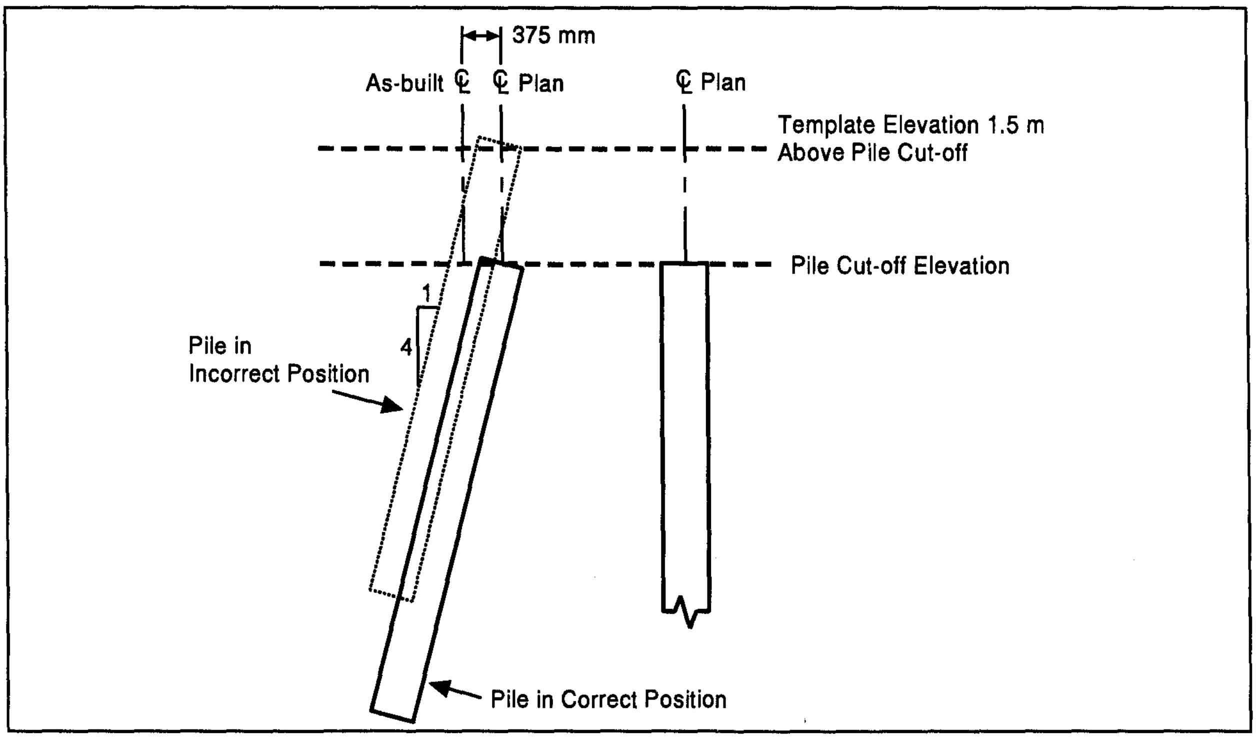
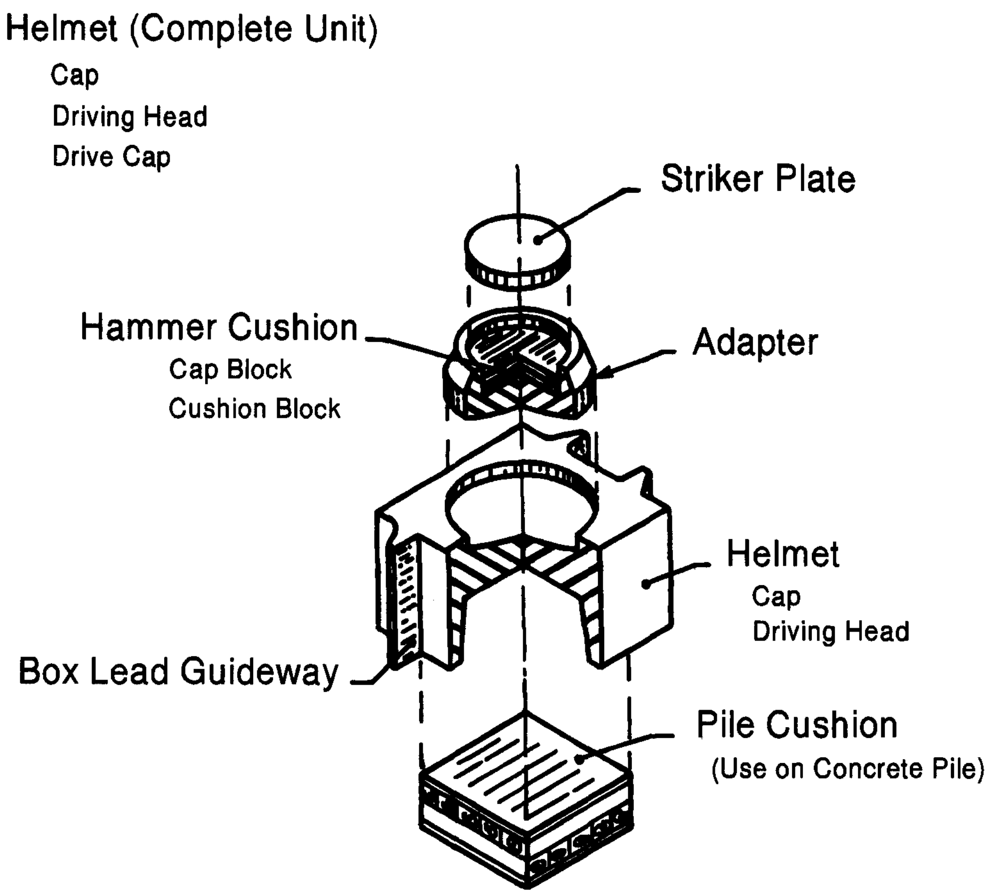


Figure 22.7 Template Elevation Effects on Batter Piles (after Passe 1994)

result in significant reduction in transferred energy or increased bending stress. Hammer cushion materials are usually proprietary man-made materials such as micarta, nylon, urethane or other polymers. In the past, a commonly used hammer cushion was made of hardwood (one piece), approximately 150 mm thick, with the wood grain parallel to the pile axis. This type of cushioning has the disadvantage of quickly becoming crushed and burned as well as having variable elastic properties during driving. With the widespread availability of manufactured hammer cushion materials, hardwood hammer cushions are no longer recommended.



Note: The helmet shown is for nomenclature only. Various sizes and types are available to drive H, pipe, concrete (shown) and timber piles. A system of inserts or adapters is utilized up inside of the helmet to change from size to size and shape to shape.

Figure 22.8 Helmet Components (after D.F.I. Publication, 1981)

The proprietary man-made hammer cushion materials have better energy transmission characteristics than a hardwood block, maintain more nearly constant elastic properties, and have a relatively long life. Their use results in more consistent transmission of hammer energy to the pile and more uniform driving. Since laminated cushioning materials have a long life, up to 200 hours of pile driving for some materials, it is often sufficient to inspect the cushion material only once before the driving operation begins for smaller projects. Periodic inspections of hammer cushion wear and thickness should be performed on larger projects. Many hammers require a specific cushion thickness for proper hammer timing. In these hammers, improper cushion thickness will result in poor hammer performance. Some man-made hammer cushions are laminated, such as aluminum and micarta, for example. The aluminum is used to transfer the heat generated during impact out of the cushion, thus prolonging its useful life. Hammer cushions consisting of small pieces of wood, coils or chunks of wire rope, or other highly elastic material should not be permitted. Cushion materials containing asbestos are not acceptable because of health hazards.

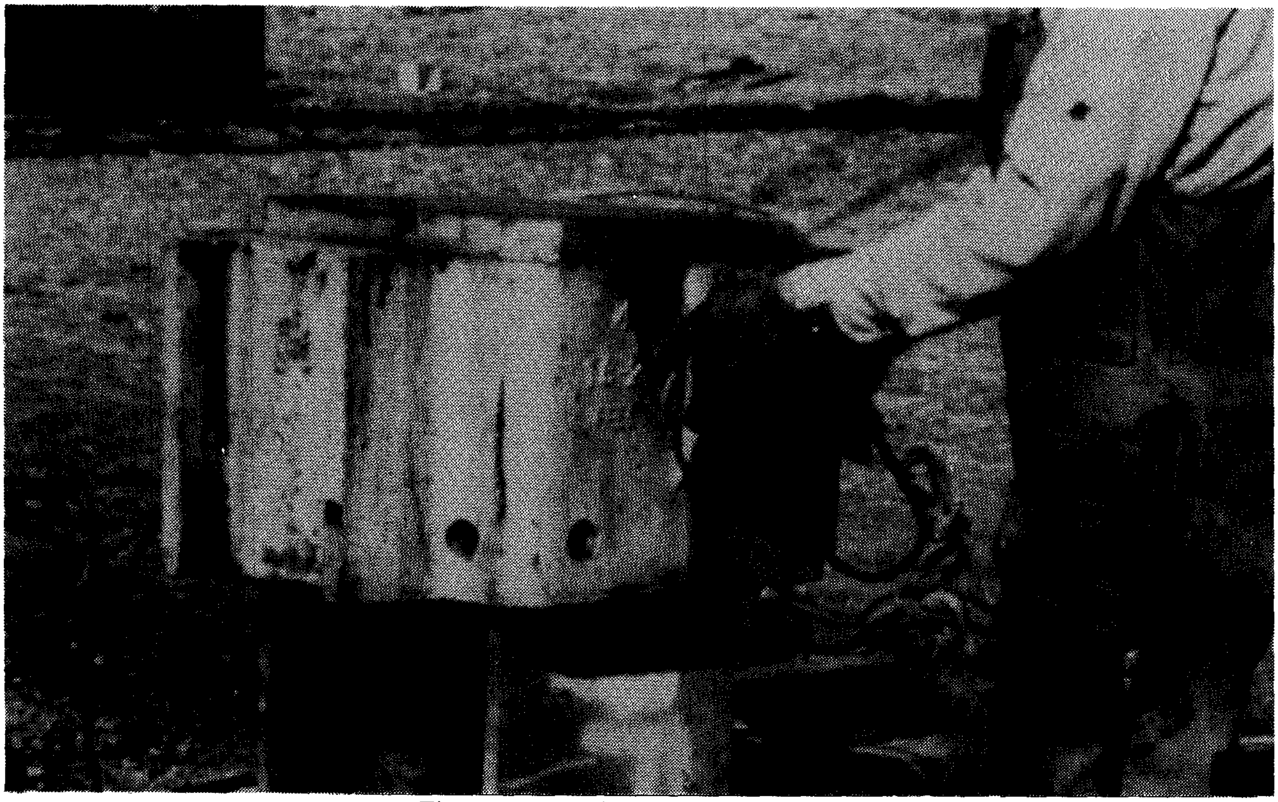


Figure 22.9 Helmet on H-pile

22.4 PILE CUSHIONS

To avoid damage to the head of a concrete pile as a result of direct impact from the helmet, a pile cushion should be placed between the helmet and the pile head. Typical pile cushions are made of compressible material such as plywood, hardwood, plywood and hardwood composites or other man made materials. Wood pile cushions should have a minimum thickness of 100 mm. Pile cushions should be checked periodically for damage and replaced before excessive compression or charring takes place. After replacing a cushion during driving, the blow count from the first 100 blows should not be used for pile acceptance as the cushion is still rapidly absorbing energy. The blow count will only be reliable after 100 blows of full energy application. The total number of blows which can be applied to a wood cushion is generally between 1000 and 2000. For wood pile cushions, it is recommended that a new, dry cushion be used for each pile. Old or water soaked cushions do not have good energy transfer, and will often deteriorate quickly. A photograph of a typical plywood pile cushion is presented in Figure 22.10.

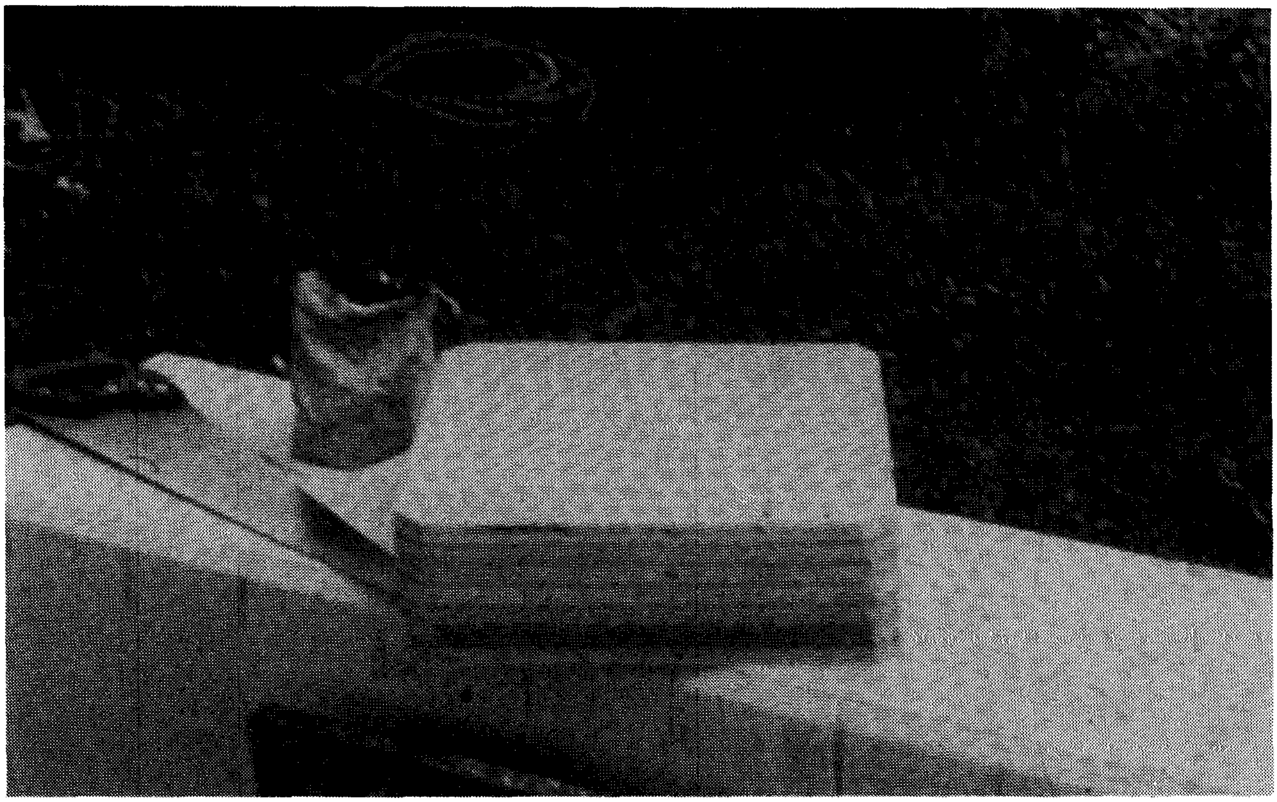


Figure 22.10 Plywood Pile Cushion

22.5 HAMMERS

Pile hammers can be categorized in two main types: impact hammers and vibratory hammers. There are numerous types of impact hammers having variations in the types of power source, configurations, and rated energies. Figure 22.11 shows a classification of hammers based on motivation and configuration factors. Table 22-1 presents characteristics and uses of several types of hammers. A discussion of various types of hammers follows in this chapter. Additional detailed descriptions of the operation of each hammer type and inspection guides are given in Chapter 24 of this manual, in Rausche *et al.* (1986), and in the Deep Foundation Institute Pile Inspector's Guide to Hammers (1995). Appendix D includes information on a majority of the currently available pile hammers.

22.5.1 Hammer Energy Concepts

Before the advent of computers and the availability of the wave equation to evaluate pile driving, driving criteria for a certain pile capacity was evaluated by concepts of work or energy. Work is done when the hammer forces the pile into the ground a certain distance. The hammer energy was equated with the work required, defined as the pile resistance times the final set. This simple idea led engineers to calculate energy ratings for pile hammers and resulted in numerous dynamic formulas which ranged from very simple to very complex. Dynamic formulas have since been widely discredited and replaced by the more accurate wave equation analysis. However, the energy rating legacy for pile hammers remains.

The energy rating of hammers operating by gravity principles only (drop, single acting air/steam or hydraulic hammers) was assigned based on their potential energy at full stroke (ram weight times stroke, h). Although single acting (open end) diesel hammers could also be rated this way, some manufacturers have used other principles for energy rating. Historically, these hammers have usually been rated by the manufacturer's rating, while the actual observed stroke was often ignored in using the dynamic formula. In current practice, the stroke is often measured electronically from the blow rate, which is an improvement over past practice. In the case of all double acting hammers (air/steam, hydraulic, or diesel), the net effect of the downward pressure on the ram during the downstroke is to increase the equivalent stroke and reduce time required per blow cycle. The equivalent stroke is defined as the stroke of the equivalent single acting hammer yielding the same impact velocity. The manufacturers generally calculate the potential energy equivalent for double acting hammers.

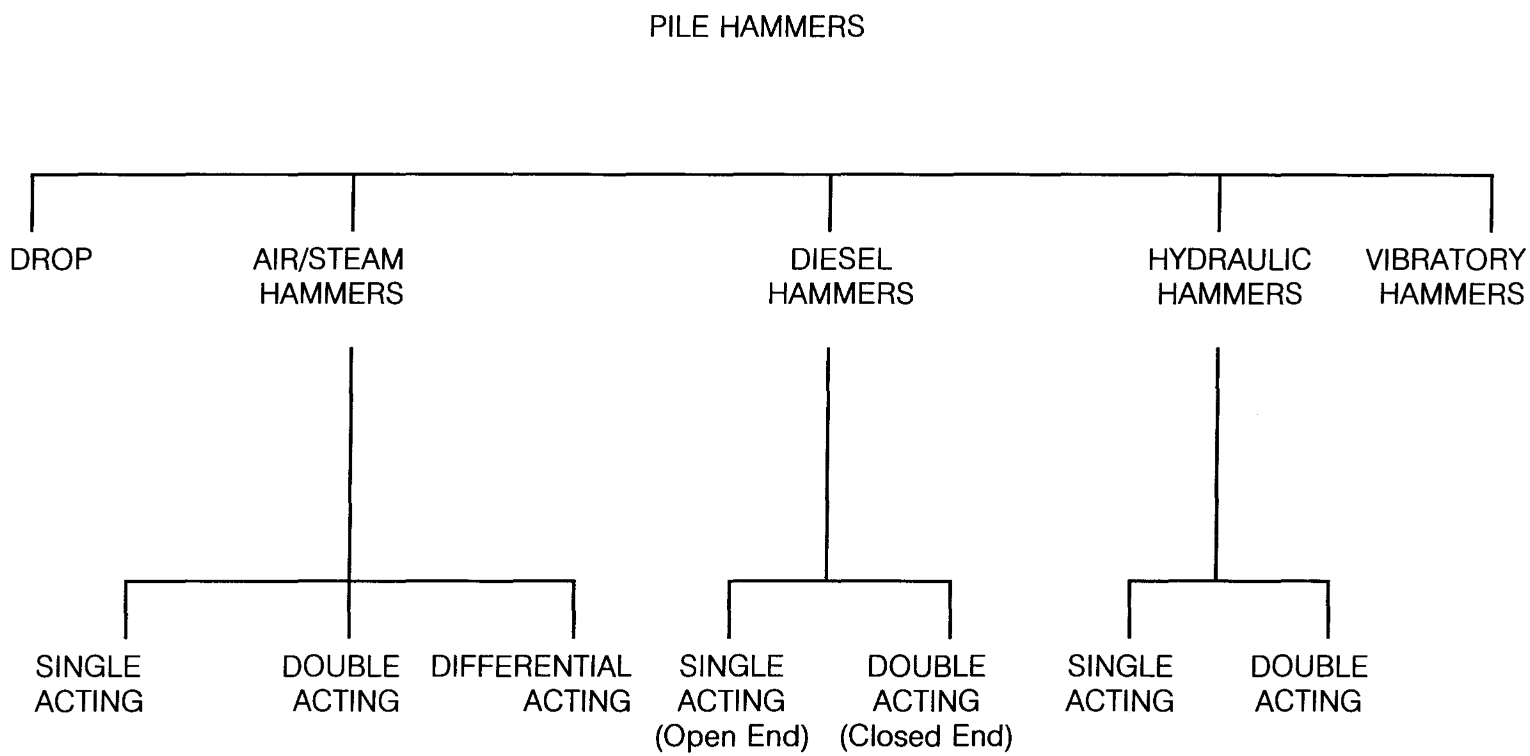


Figure 22.11 Pile Hammer Classification

TABLE 22-1 TYPICAL PILE HAMMER CHARACTERISTICS AND USES

Hammer Type	Drop	Steam or Air			Diesel		Hydraulic		Vibratory
		Single Acting	Double Acting	Differential	Single Acting (open end)	Double Acting (closed end)	Single Acting	Double Acting	
Rated energy range (kJ)	9 to 81	9 to 2440	5 to 225	20 to 68	5 to 380	11 to 88	35-2932	35-2984	----
Impact velocity (m/sec)	7 to 10	2.5 to 5	4.5 to 6	4 to 4.5	*	**	*	**	----
Blows/minute	4 to 8	35 to 60	95 to 300	98 to 303	40 to 60	80 to 105	30 to 50	40 to 90	750 to 2,000 pulses/minute
Energy (per blow)	Ram weight x height of fall.	Ram weight x ram stroke.	(Ram weight + effective piston head area x effective fluid pressure) x stroke.		Ram weight x stroke.	(Ram weight + chamber pressure) x stroke.	Ram weight x stroke.	(Ram weight + effective piston head area x effective fluid pressure) x stroke.	----
Lifting power	Provided by hoisting engine or a crane.	Steam or air.	Steam or air.		Provided by the explosion of injected diesel fluid.		Hydraulic	Hydraulic	Electricity or hydraulic power.
Maintenance	Simple	More complex than for drop hammer.	More complex than for single acting.		More complex than most air impact hammers.		More complex than other impact hammers.	More complex than other impact hammers.	Highest maintenance cost.
Hammer suitability for types of piles	All types except concrete piles.	Versatile for any pile, particularly large concrete and steel pipe.	Timber, steel H and pipe piles.		All types of piles.		All types of piles.	All types of piles.	Steel H and pipe end bearing piles. Very effective in granular soils.
Major advantages	Lowest initial cost equipment.	Relatively simple and moderate cost.	Fully enclosed and permit underwater operation. More productive than single acting. Generate lower dynamic forces. Differential hammer uses less volume of air or steam than double acting and has lower impact velocity.		Carry their own fuel from which power is internally generated. Stroke is a function of pile resistance.		Fully variable energy can be delivered.	Energy is variable over a wide range. Can be used for underwater driving.	Can be used for pulling or driving. Fastest operating installation tool.
Major disadvantages	Very high dynamic forces and danger of pile damage. Lowest pile productivity.	Need air compressor or steam plant. Heavy compared with most diesel hammers.	Costs more than single acting. Need air compressor or steam plant. Heavy compared to diesel hammer.		Pollutes air with exhaust. High cost hammer. Low blows per minute at higher strokes for single acting.		High initial cost.	High initial cost.	High investment and maintenance. Not recommend for friction pile installations.
Remarks	Becoming obsolete.	----	Ram accelerates downward under pressure.		Stroke variable in single acting diesel hammer. Becoming very popular.		New hammer type and may require additional field inspection and/or testing.	New hammer type and may require additional field inspection and/or testing.	----

* Depends on stroke
** Depends on chamber pressure

Ideally, the impact velocity, v_i, could be directly computed using basic laws of physics from the equivalent maximum stroke

$$v_i = \sqrt{2gh}$$

Where: g = Acceleration due to gravity, m/s^2.
h = Hammer stroke, m.

The kinetic energy could be computed from the equation

$$K.E. = ½\, m\, v_i^2$$

Where: m = Ram mass.

If there were no losses, the kinetic energy would equal the potential energy. In reality however, energy losses occur due to a variety of factors (friction, residual air pressures, preadmission, gas compression in the diesel combustion cylinder, preignition, *etc.*) which result in the kinetic energy being less than the potential energy. It is the inspector's task to minimize these losses when and where possible, or to at least identify and try to correct situations where losses are excessive. Some hammers, such as modern hydraulic hammers, measure the velocity near impact and hence can calculate the actual kinetic energy available.

Further losses occur in the transmission of energy to the pile. The hammer cushion, helmet, and pile cushion all have kinetic energy and store some strain energy. The pile head also has inelastic collision losses. The hammer transfers its energy to the pile with time. The energy delivered to the pile can be calculated from the work done as the integral of the product of force and velocity with time and is referred to as the transferred energy or ENTHRU.

The pile length, stiffness and capacity influence the energy delivered to the pile. The actual stroke (or potential energy) of diesel hammers depends on the pile resistance and the net transferred energy is also a variable. The stroke of single acting air/steam hammers is also somewhat dependent upon the pile capacity and rebound. The stroke of all double acting hammers is even more dependent on pile capacity due to lift-off considerations. Actually the transferred energy increases only when both the force and velocity are positive (compression forces; downward velocity). As resistance increases and/or the pile becomes shorter, the rebound or upward velocity occurs earlier and the pile then transfers energy back to the driving system. In fact, the energy returning to the hammer may occur before all the energy has been transferred into the pile.

22.6 DROP HAMMERS

The most rudimentary pile hammer still in use today is the drop hammer as shown in Figure 22.12. These hammers consist of a hoisting engine having a friction clutch, a hoist line, and a drop weight. The hammer stroke is widely variable and often not very precisely controlled. The hammer is operated by engaging the hoist clutch to raise the drop weight or ram. The hoist clutch is then disengaged, allowing the drop weight to fall as the hoist line pays out. The fall may not be very efficient since the ram attached by cable to the hoist must also overcome the rotational inertia of the hoist. Ideally, the crane operator engages the clutch immediately after impact to prevent excessive cable spooling. If the operator prematurely engages the clutch, or it is partially engaged during spooling, then the fall efficiency and hence impact energy is further reduced.

The hammer operating speed (blows per minute) depends upon the skill of the operator and the height of fall being used, but is generally very slow. One of the greatest risks in using a drop hammer is overstressing and damaging the pile. Pile stresses are generally increased with an increase in the impact velocity (hammer stroke) of the striking weight. Therefore, the maximum stroke should be limited to those strokes where pile damage is not expected to occur. In general, drop hammers are not as efficient as other impact hammers but are inexpensive and simple to operate and maintain. Current use of these hammers is generally limited to sheet pile installations where pile capacity is not an issue. Because of the uncertainties described above, drop hammers are not recommended for foundation piles.

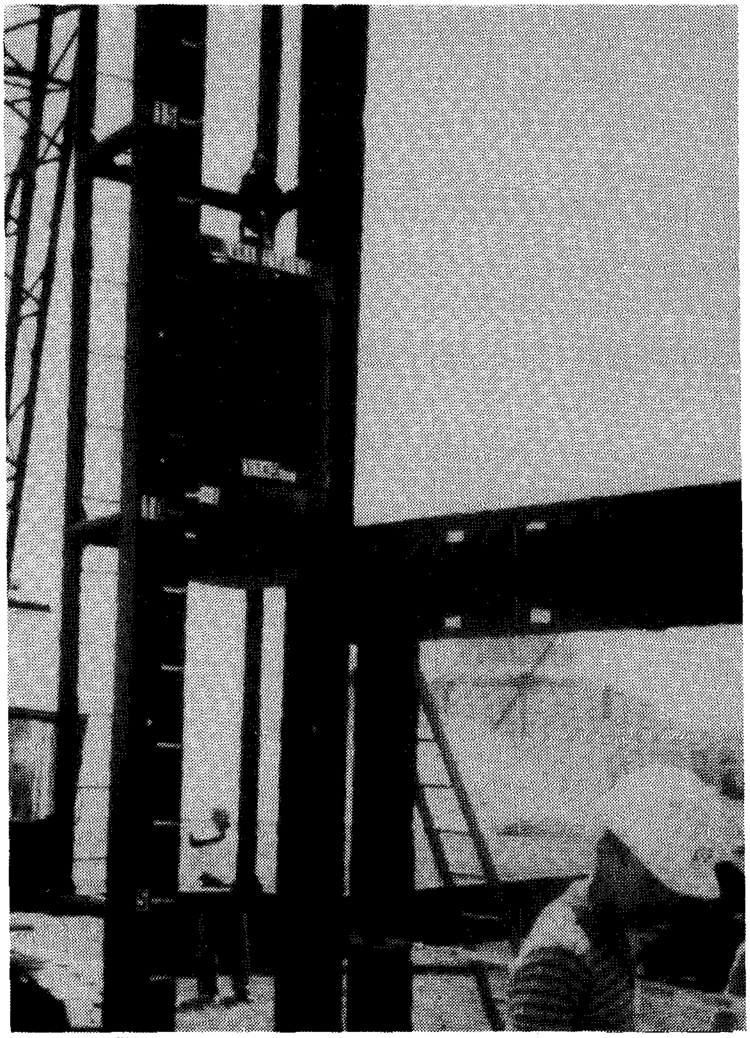
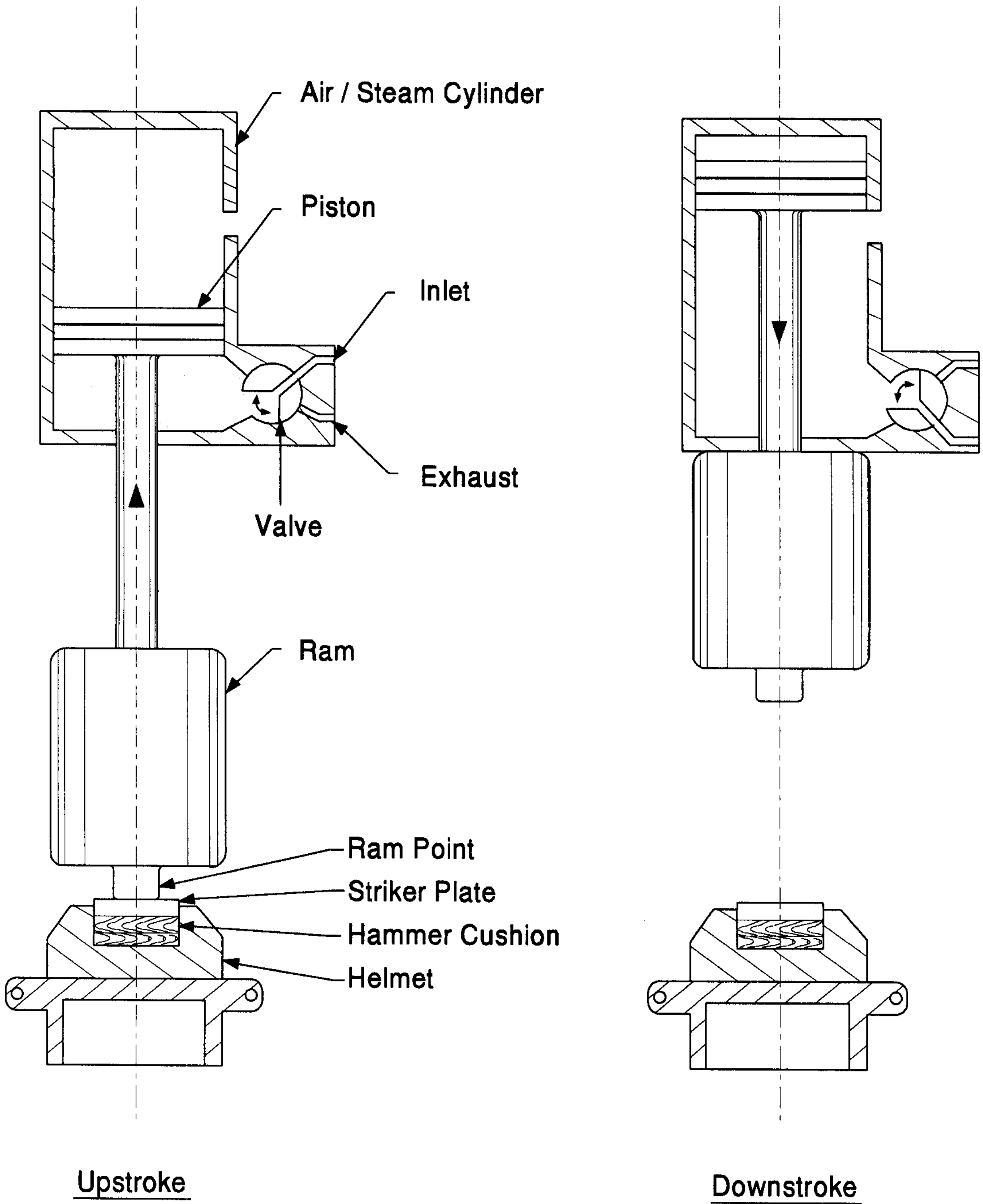


Figure 22.12 Typical Drop Hammer

22.7 SINGLE ACTING AIR/STEAM HAMMERS

Single acting air/steam hammers are essentially gravity, or drop hammers, for which the hoist line has been replaced by a pressurized medium, being either steam or air. While originally developed for steam power, most of these hammers today operate on compressed air. To lift the ram weight with motive pressure, a simple one-cylinder steam engine principle is used. The ram consists of a compact block with a so-called ram point attached at its base. The ram point strikes against a striker plate as illustrated in Figure 22.13. A photograph of a typical single acting air/steam hammer is presented in Figure 22.14.



Single Acting Air / Steam Hammer

Figure 22.13 Schematic of Single Acting Air/Steam Hammer

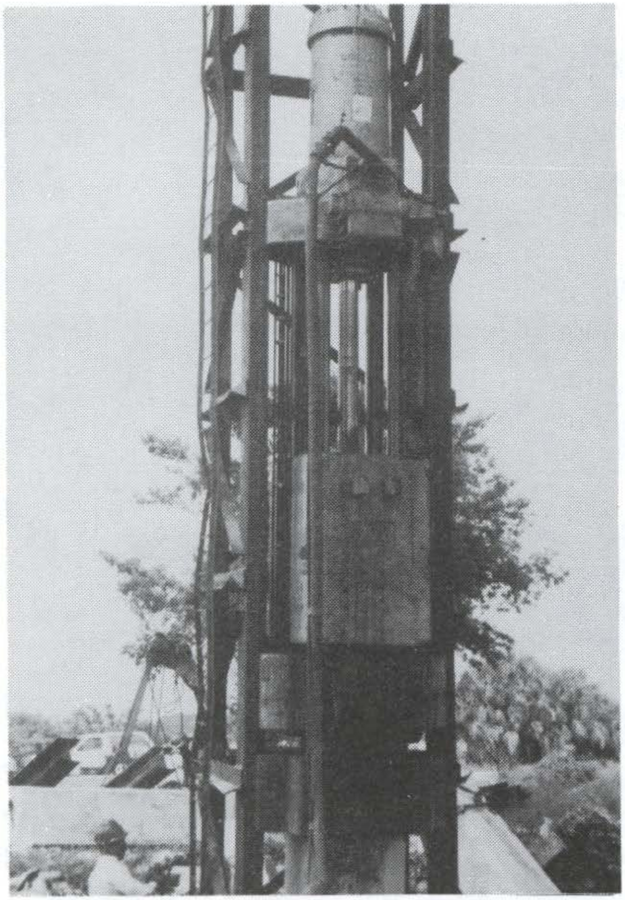


Figure 22.14 Single Acting Air Hammer

Figure 22.15 Double Acting Air Hammer

During the upstroke cycle, the ram is raised by externally produced air or steam pressure acting against a piston housed in the hammer cylinder. The piston in turn is connected to the ram by a rod. Once the ram is raised a certain distance, a valve is activated and the pressure in the chamber is released. At that time, the ram has some remaining upward velocity that depends upon the pile rebound, inlet air pressure, and volume of air within the hammer cylinder. Against the action of gravity and friction, the ram then "coasts" up to the maximum height (stroke). The maximum stroke, and hence hammer potential energy, is therefore not constant and depends upon the pressure and volume of air or steam supplied, as well as the amount of pile rebound due to pile resistance effects. During the downstroke cycle, the ram falls by gravity (less friction) to impact the striker plate and hammer cushion. Just before impact, the pressure valve is activated and pressure again enters the cylinder. This occurs approximately 50 mm before impact, but depends on having the correct hammer cushion thickness. If the hammer cushion height is too low, then the pressure is introduced too early, reducing the impact energy of the ram. This is referred to as "preadmission".

The dynamic forces exerted on a pile by a single acting air/steam hammer are of the same short-time duration as those exerted by a drop hammer. Because operating strokes are generally shorter, the accelerations generated by single acting air/steam hammers do not reach the magnitude of drop hammers. Some hammers may be equipped with two nominal strokes, one full stroke and another of lesser height. The hammer operator can switch between the two to better match the driving conditions and limit driving resistance or control tension driving stresses as needed. The maximum stroke of single acting air/steam hammers generally ranges from 0.9 to 1.5 meters. The weights of single acting air/steam hammer rams are usually considerably higher than drop hammer weights. Single acting air/steam hammers have the advantages of moderate cost and relatively simple operation and maintenance. They are versatile for many pile types, particularly large concrete and steel pipe piles.

22.8 DOUBLE ACTING AIR/STEAM HAMMERS

A photograph of an enclosed double acting air hammer is presented in Figure 22.15 and the working principle of a double acting hammer is illustrated in Figure 22.16. The ram of a double acting hammer is raised by pressurized steam or air during the upstroke. As the ram nears the maximum up stroke, the lower air valve opens, allowing the lower cylinder chamber to release the pressurized air. Once the ram reaches full stroke, the upper valve changes to admit pressurized steam or air to the upper cylinder. Gravity and the upper cylinder pressure accelerate the ram through its downward fall. As with the single acting hammer, the stroke is again not constant, due to variable lift pressure and volume as well as differing pile rebound. During hard driving with high pile rebound, the pressure may need to be reduced to prevent lift-off, with the hammer actually lifting up away from the pile. Since the maximum stroke is limited and the same lifting pressure is applied during downstroke, a pressure reduction may cause the kinetic energy at impact to be reduced during these hard driving situations. Just before impact, the valve positions are reversed and the cycle repeats.

The correct cushion thickness is extremely important for the proper operation of the hammer. If the hammer timing is off significantly, it is possible for the hammer to run with the ram moving properly, but with little or no impact force delivered to the pile. The kinetic energy of the ram at impact depends on the ram weight and stroke as well as the motive pressure effects. The overall result is that a properly operating double acting hammer with its shorter stroke delivers comparable impact energy per blow at up to about two times the blow rate of a single acting hammer of the same ram weight.

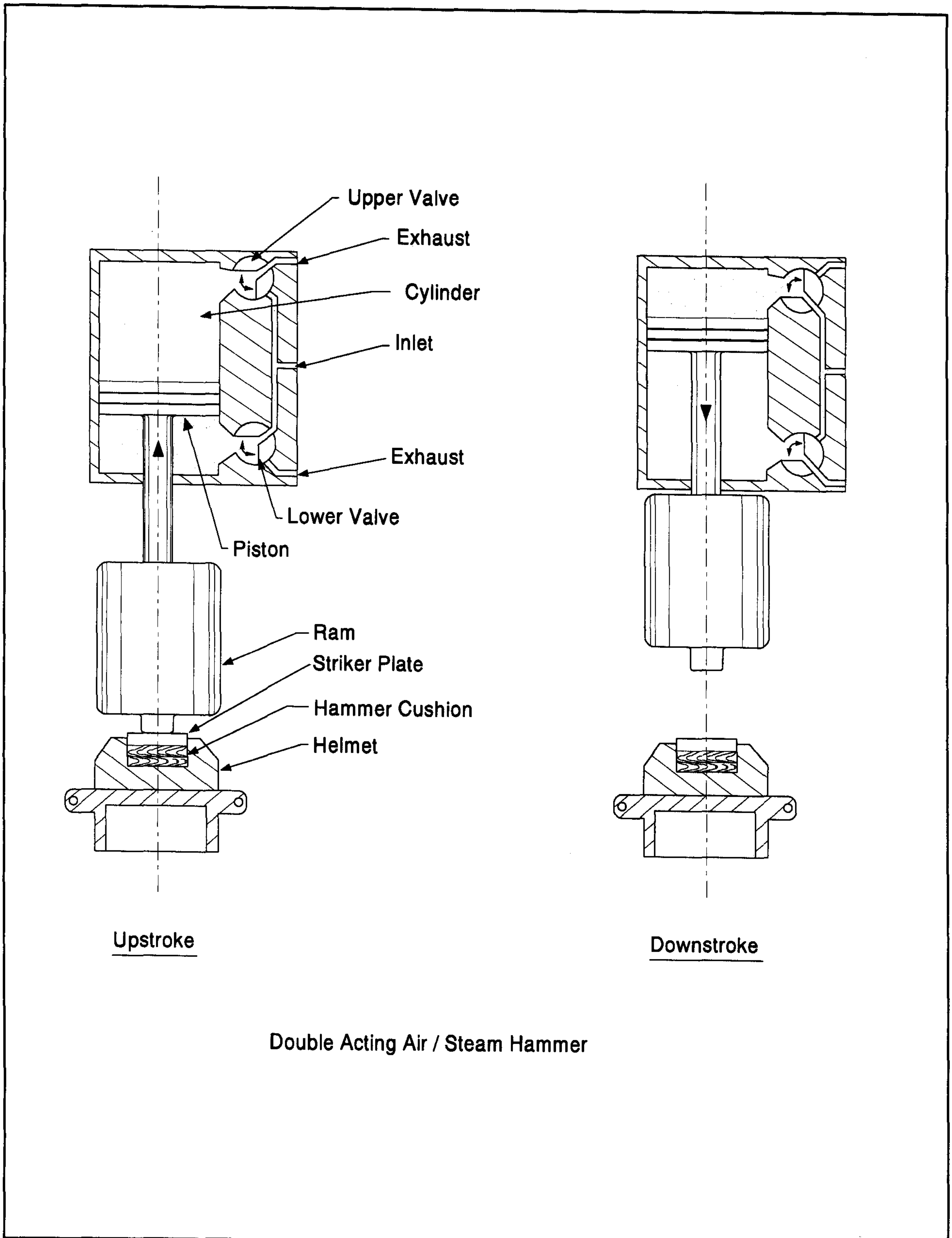


Figure 22.16 Schematic of Double Acting Air/Steam Hammer

Some double acting air/steam hammers are fully enclosed and can be operated underwater such as the one shown in Figure 22.15. They may be more productive than single acting hammers, but are more dependent upon the air pressure. Experience has shown that on average, they are slightly less efficient than equivalently rated single acting hammers. Double acting hammers generally cost more than single acting hammers and require additional maintenance. Similar to single acting air/steam hammers, they require an air compressor or a steam plant. However, double acting air/steam hammers consume more air and require greater air pressures than equivalent single acting hammers.

22.9 DIFFERENTIAL ACTING AIR/STEAM HAMMERS

A differential acting air/steam hammer is another type of double acting hammer with relatively short stroke and fast blow rates. The working principle of a differential hammer is illustrated in Figure 22.17. Operation is achieved by pressure acting on two different diameter pistons connected to the ram. At the start of the cycle, the single valve is positioned so that the upper chamber is open to atmospheric pressure only and the lower chamber is pressurized with the motive fluid. The pressure between the two pistons has a net upward effect due to the differing areas, thus raising the ram. The ram has an upward velocity when the valve position changes and applies air pressure into the upper chamber, causing the net force to change to the downward direction. Thus air pressure along with gravity and friction slows the ram, and after attaining the maximum stroke of the cycle, assists gravity during the downstroke to speed the ram.

As with the double acting hammers, the kinetic energy at impact may need to be reduced during hard driving since the pressure, which assists gravity during downstroke, must be reduced to prevent hammer lift-off. As with the other air/steam hammers, when the ram attains its maximum kinetic energy just before impact, the valve position is reversed and the cycle begins again. Therefore, the hammer cushion must be of the proper thickness to prevent preadmission which could cause reduced transferred energy. Very high air pressures between 820 and 970 kPa at the hammer inlet are required for proper operation. However, most air compressors only produce pressures of about 820 to 900 kPa at the compressor. As with the double acting hammer, the efficiency of a differential hammer is somewhat lower than the equivalent single acting air/steam hammer. The heavier ram of the differential acting hammer is lifted and driven downward with a lower volume of air or steam than is used by a double acting hammer.

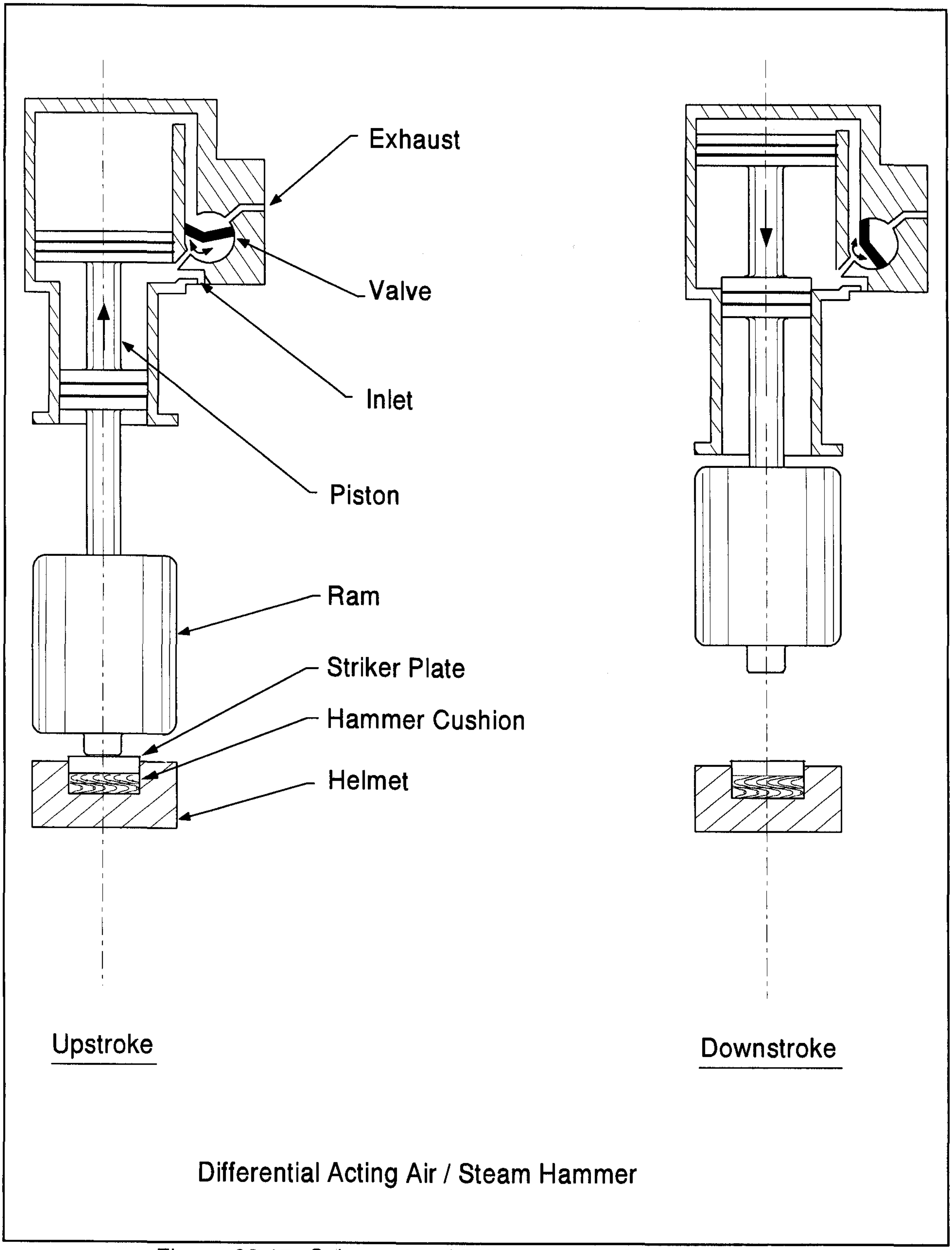


Figure 22.17 Schematic of Differential Air/Steam Hammer

22.10 SINGLE ACTING (OPEN END) DIESEL HAMMER

The basic distinction between all diesel hammers and all air/steam hammers is that, whereas air/steam hammers are one-cylinder engines requiring motive power from an external source, diesel hammers carry their own fuel from which they generate power internally. Figure 22.18 shows the working principle of a single acting diesel hammer. The initial power to lift the ram must be furnished by a hoist line or other source to lift the ram upward on a trip block. After the trip mechanism is released, the ram guided by the outer hammer cylinder falls under gravity. As the ram falls, diesel fuel is injected into the cylinder below the air/exhaust ports. Once the ram passes the air/exhaust ports the diesel fuel is compressed and heats the entrapped air. As the ram impacts the anvil the fuel explodes, increasing the gas pressure. In some hammers the fuel is injected in liquid form as shown in Figure 22.18(b), while in other hammers the fuel is atomized and injected later in the cycle and just prior to impact. In either case, the combination of ram impact and fuel explosion drives the pile downward, and the gas pressure and pile rebound propels the ram upward in the cylinder. On the upstroke, the ram passes the air ports and the spent gases are exhausted. Since the ram has a velocity at that time, the ram continues upward against gravity, and fresh air is pulled into the cylinder. The cycle then repeats until the fuel input is interrupted.

There is no consensus by the various hammer manufacturers on how a single acting diesel hammer should be rated. Many manufacturers use the maximum potential energy computed simply from maximum stroke times the ram weight. The actual hammer stroke achieved is a function of fuel charge, condition of piston rings containing the compressed gases, recoil dampener thickness, driving resistance, and pile length and stiffness. Therefore, the hammer stroke cannot be controlled. A set of conditions will generate a certain stroke which can only be adjusted within a certain range by the fuel charge. It may not be possible to achieve the manufacturer's maximum rated stroke under normal conditions. In normal conditions, part of the available potential energy is used to compress the gases as the ram proceeds downward after passing the air ports. The gases ignite when they attain a certain combination of pressure and temperature. Under continued operation, when the hammer's temperature increases due to the burning of the gases, the hammer fuel may ignite prematurely. This condition, called "preignition", reduces the effectiveness of the hammer, as the pressure increases dramatically before impact, causing the ram to do more work compressing the gases and leaving less energy available to be transferred into the pile.

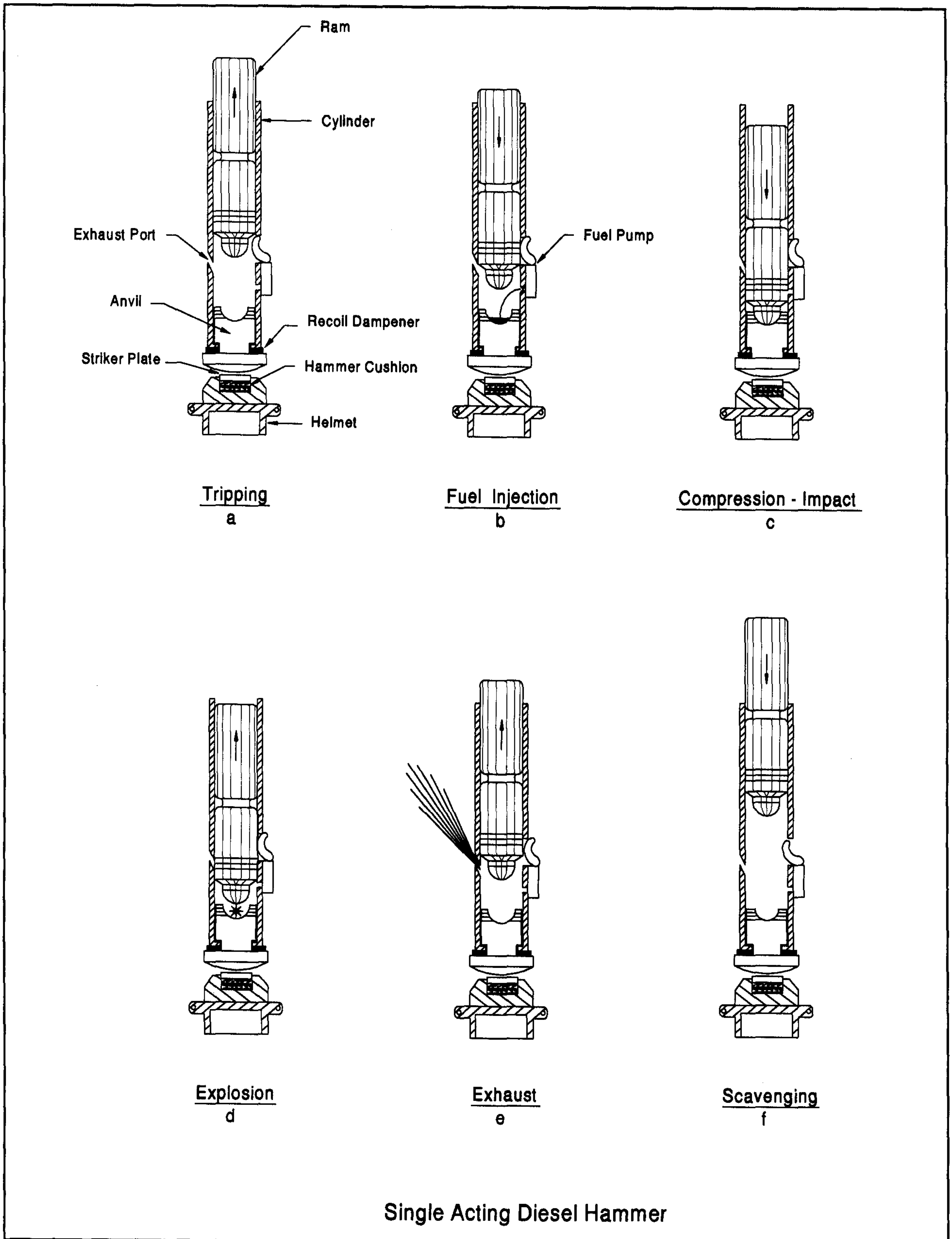


Figure 22.18 Schematic of Single Acting Diesel Hammer

When driving resistance is very low, the upward ram stroke may be insufficient to scavenge (or suction) the air into the cylinder and the hammer may not continue to operate. Thus, the ram must be manually lifted repeatedly until resistance increases. The stroke can be reduced for most hammers by reducing the amount of fuel injected. Some hammers have stepped fuel settings while others have continuously variable throttles. Other hammers use pressure to maintain fuel flow by connecting a hand operated fuel pump to the hammer, which is operated at the ground. By adjusting the fuel pump pressure, hammer strokes may be reduced. Using the hammer on reduced fuel can be useful for limiting driving stresses. For single acting diesel hammers, the stroke is also a function of pile resistance, which also helps in limiting driving stresses. This feature is very useful in controlling tensile stresses in concrete piles during easy driving conditions. The actual stroke can and should be monitored. The stroke of a single acting diesel hammer can be calculated from the following formula:

$$h = \left[4400/[bpm^2]\right] - 0.09$$

Where: h = Hammer stroke in meters.
bpm = Blows per minute.

Diesel hammers may be expensive and their maintenance more complex. Concerns over air pollution from the hammer exhaust have also arisen, causing some areas to require a switch to kerosene fuel. However, it should be noted that diesel hammers burn far less fuel to operate than the air compressor required for an air/steam hammer. Diesel hammers are also considerably lighter than air/steam hammers with similar energy ratings, allowing a larger crane operating radius and/or a lighter crane to be used. A photograph of a typical single acting diesel hammer is shown in Figure 22.19.

22.11 DOUBLE ACTING (CLOSED END) DIESEL HAMMER

The double acting diesel hammer works very much in principle like the single acting diesel hammer. The main change consists of a closed cylinder top. When the ram moves upward, air is being compressed at the top of the ram in the so called "bounce chamber" which causes a shorter stroke and therefore a higher blow rate. A photograph of a typical double acting diesel hammer is provided in Figure 22.20.

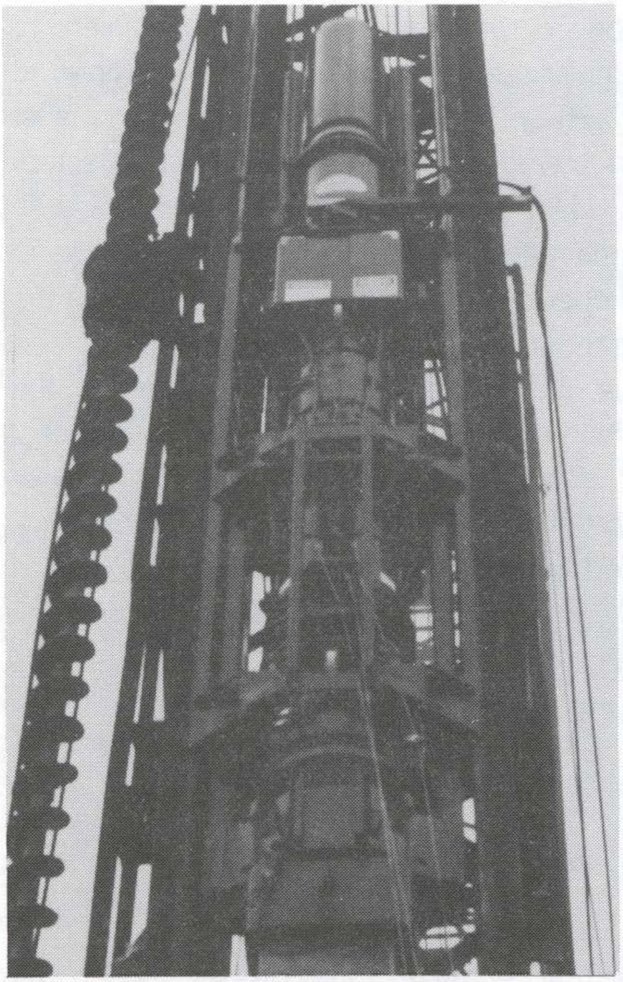


Figure 22.19 Single Acting Diesel Hammer (courtesy of Pileco)

Figure 22.20 Double Acting Diesel Hammer

The bounce chamber has ports so that atmospheric pressure exists as long as the ram top is below these ports, as shown in Figure 22.21. Operationally, as the ram passes the bounce chamber port and moves toward the cylinder top, it creates a pressure which effectively reduces the stroke and stores energy, which in turn will be used on the downstroke. Like the single acting hammer, the actual stroke depends on fuel charge, pile length and stiffness, soil resistance, and condition of piston rings. As the stroke increases, the chamber pressure also increases until the total upward force is in balance with the weight of the cylinder itself. Further compression beyond this maximum stroke is not possible, and if the ram still has an upward velocity, uplift of the hammer will result. This uplift should be avoided as it can lead both to an unstable driving condition and to hammer damage. For this reason, the fuel amount, and hence maximum

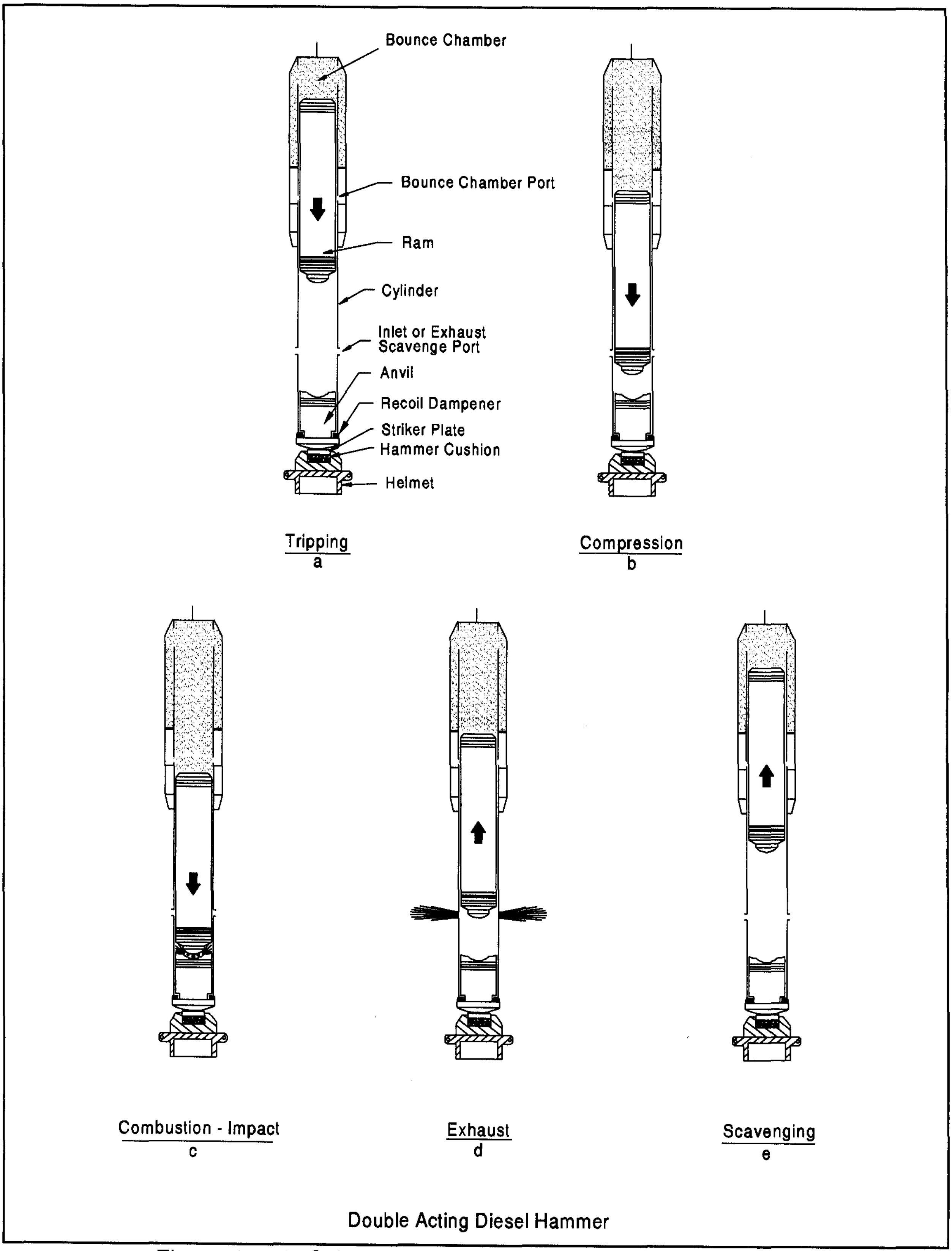


Figure 22.21 Schematic of Double Acting Diesel Hammer

combustion chamber pressure, has to be reduced so that there is only a very slight lift-off or none at all. Most of these hammers have hand held fuel pumps connected by rubber hose to control the fuel flow. Hammer strokes, and therefore hammer energy, may be increased or decreased by the fuel pump pressure.

To determine the energy provided by the hammer, the peak bounce chamber pressure in the hammer is read from a bounce chamber pressure gage. The hammer manufacturer should supply a chart which correlates the bounce chamber pressure gage reading as a function of hose length with the energy provided by the hammer.

22.12 HYDRAULIC HAMMERS

There are many different types of hydraulic hammers. However, all hydraulic hammers use an external hydraulic power source to lift the ram, as illustrated in Figure 22.22. The ram drop may be due to gravity only, or may be hydraulically assisted. They can be perhaps thought of as a modern, although more complicated, version of air/steam hammers in that the ram weights and maximum strokes are similar in sizes and the ram is lifted by an external power source. The simplest version lifts the ram with hydraulic cylinders which then retract quickly, fully releasing the ram, which then falls under gravity. The ram impacts the striker plate and hammer cushion located in the helmet. The hydraulic cylinder then lifts the ram again and the cycle is repeated. Other models employ hydraulic accumulators during the downstroke to store a volume of hydraulic fluid used to speed up the ram lifting operation after impact. Similar to air/steam hammers, hydraulic hammers are also made in both single and double acting versions. The above models with hydraulic accumulators often have a relatively small double acting component. Other more complicated models have nitrogen charged accumulator systems, which store significant energy allowing a shortened stroke and increased blow rate. Photographs of single acting and double acting hydraulic hammers are provided in Figures 22.23 and 22.24, respectively.

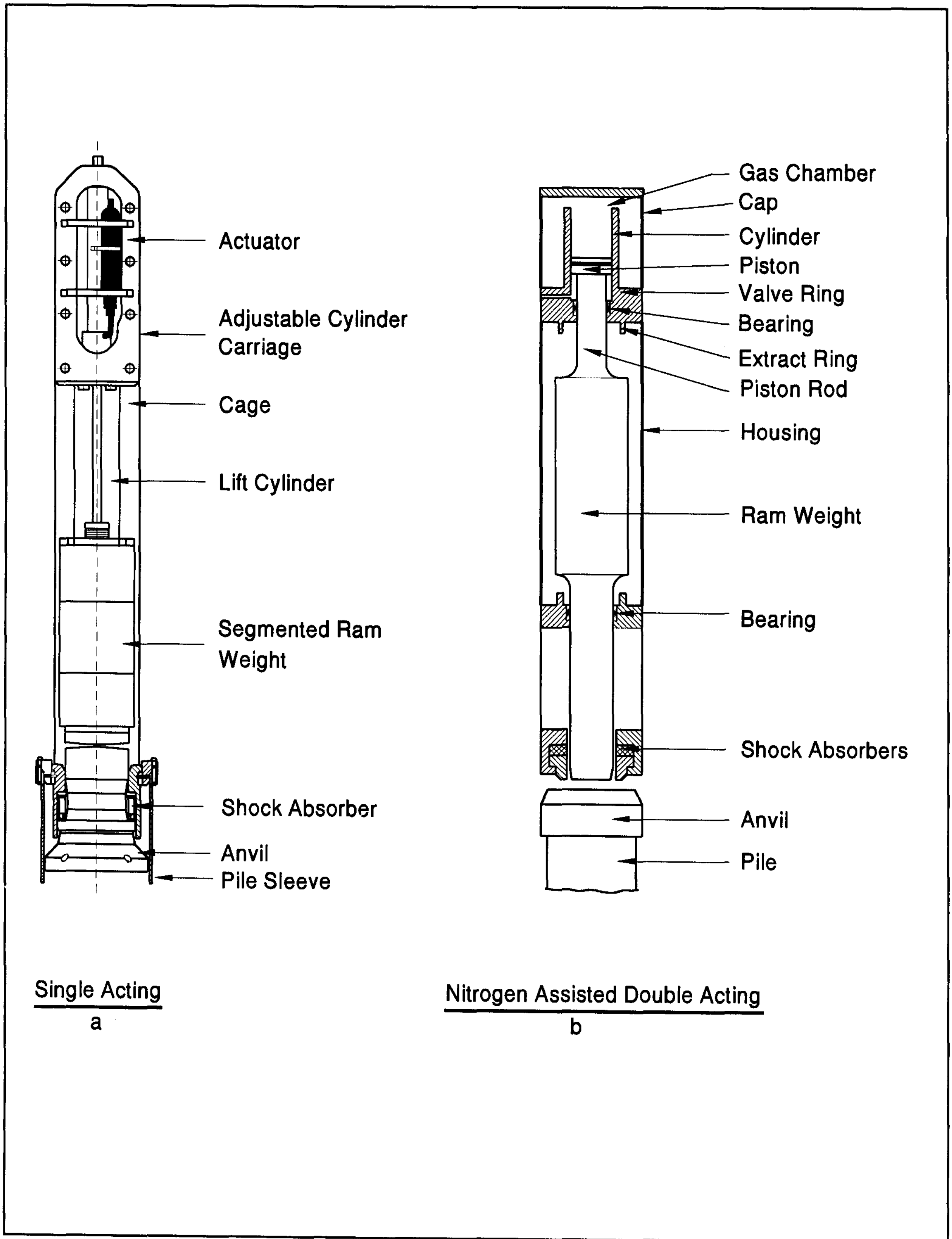


Figure 22.22 Schematics of Single and Double Acting Hydraulic Hammers

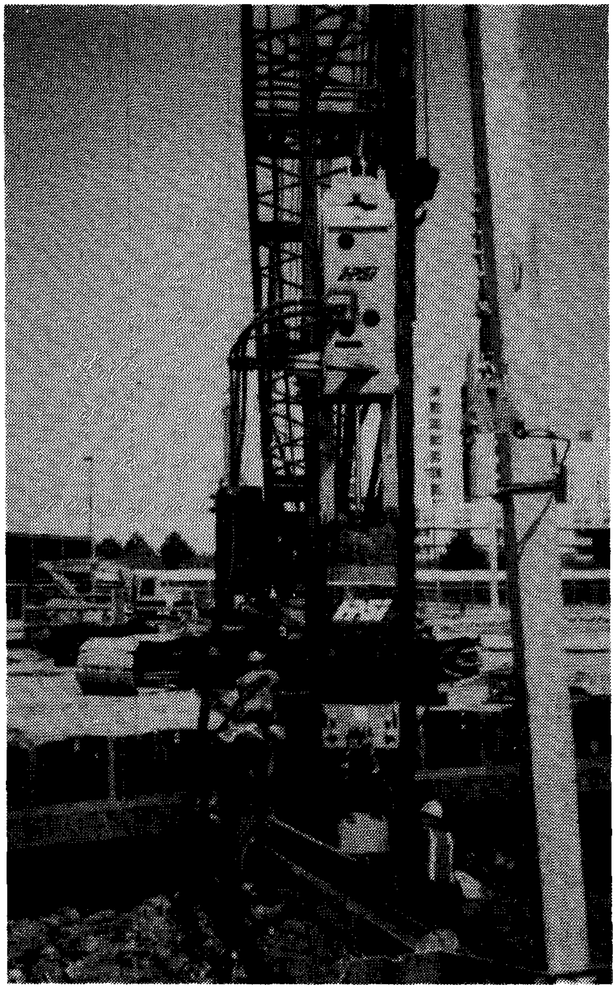


Figure 22.23
Single Acting Hydraulic Hammer

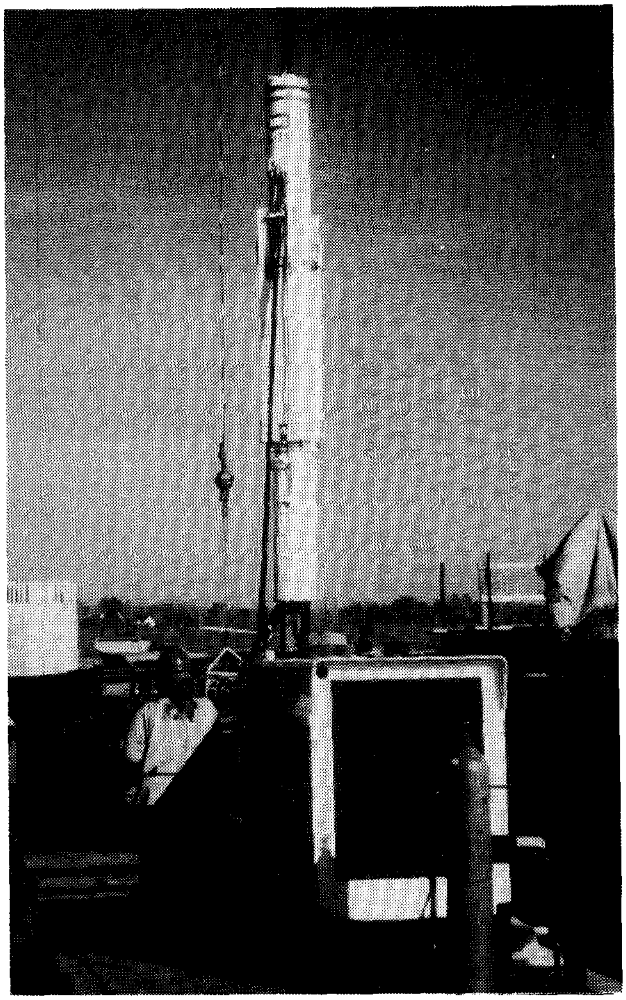


Figure 22.24
Double Acting Hydraulic Hammer

All hydraulic hammers allow the ram stroke to be continuously variable and controlled to adapt to the driving conditions. Very short strokes for easy driving may be used to prevent pile run or to minimize tension stresses in concrete piles. Higher strokes are available for hard driving conditions. On many hydraulic hammers, the stroke can be visually estimated. However, most hydraulic hammers include a built-in monitoring system which determines the ram velocity just before impact. The ram velocity can be converted to kinetic energy or equivalent stroke. Because of the variability of stroke, this hammer monitor should be required as part of the hammer system. The monitor results should be observed during pile driving with appropriate hammer performance notes recorded on the driving log.

Some hydraulic hammers can be equipped with extra noise abatement panels. A significant advantage of some hydraulic hammers is that they are fully enclosed and can operate underwater. This allows piles to be driven without using a follower or extra length pile. Some hydraulic hammers do not have hammer cushions and thus generate steel to steel impacts with high hammer efficiencies. Therefore, hydraulic hammers are often not used at their full energy potential. Hydraulic hammers require a dedicated hydraulic power pack, and can be more complex to operate and maintain compared to other hammers.

22.13 VIBRATORY HAMMERS

Vibratory hammers use paired counter-rotating eccentric weights to impart a sinusoidal vibrating axial force to the pile (the horizontal components of the paired eccentors cancel). A schematic of a vibratory hammer is presented in Figure 22.25(a) and a photograph is included in Figure 22.25(b). Most common hammers operate at about 1000 Hz. These hammers are rigidly connected by hydraulic clamps to the pile head and may be used for either pile installation or extraction. These hammers typically do not require leads, although templates are often required for sheet pile cells. Vibratory hammers are not rated by impact energy delivered per blow, but instead are classified by energy developed per second and/or by the driving force they deliver to the pile. The power source to operate a vibratory hammer is usually a hydraulic power pack.

Vibratory hammers are commonly used for driving/extracting sheet piles and can also be used for installing non-displacement H-piles and open end pipe piles. However, it is often difficult to install closed end pipes and other displacement piles due to difficulty in displacing the soil laterally at the toe. Vibratory hammers should not be used for precast concrete piles because of possible pile damage due to tensile and bending stress considerations. Vibratory hammers are most effective in granular soils, particularly if submerged. They also may work in silty or softer clays, but most experience suggests they are less effective in stiff to hard clays.

Some wave equation analysis programs can simulate vibratory driving. Dynamic measurements have also been made on vibratory hammer installed piles. However, a reliable technique for estimating pile capacity during vibratory hammer installation has not yet been developed. Hence, if a vibratory hammer is used for installation, a confirmation test of pile capacity by some method is still necessary.

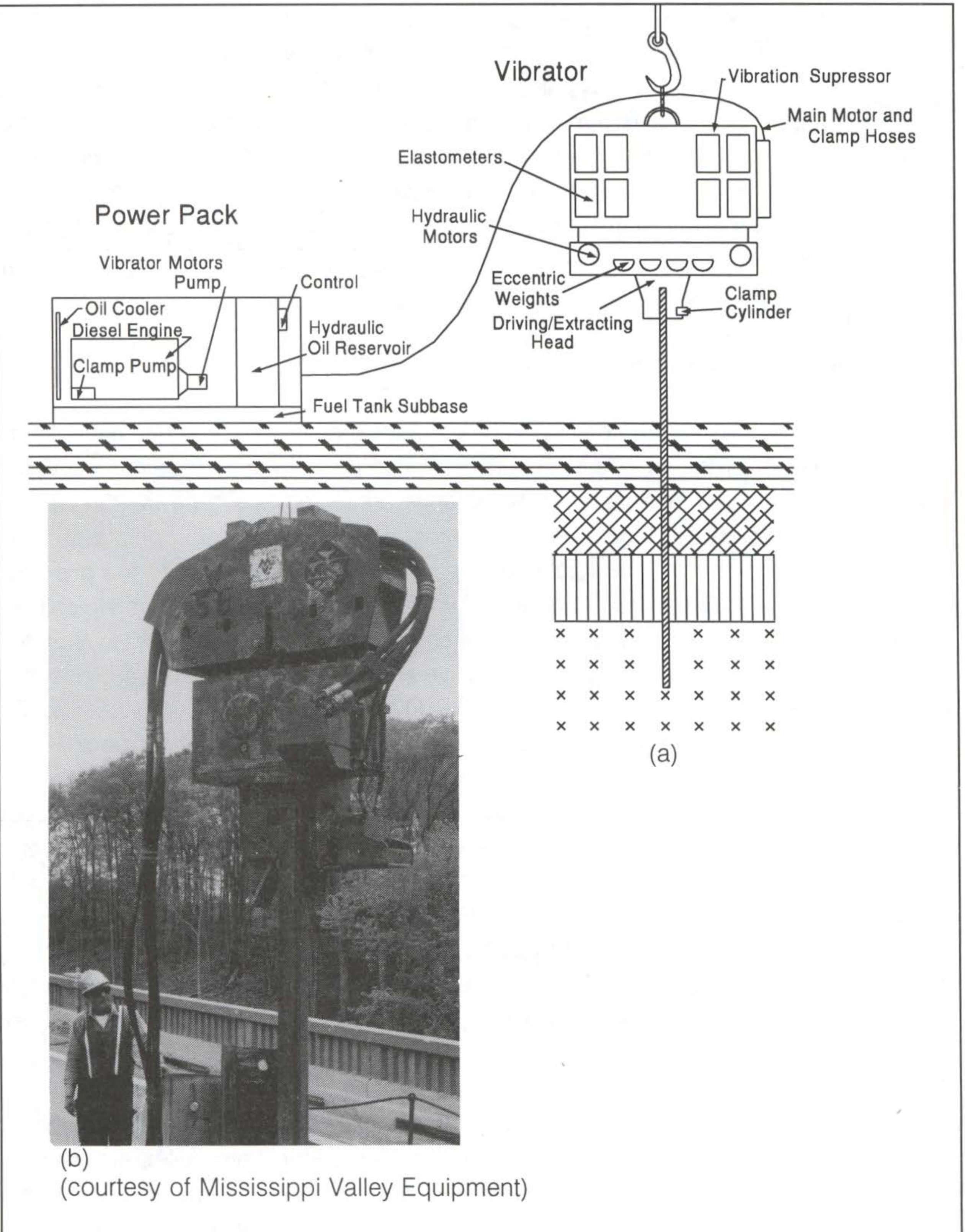


Figure 22.25 Vibratory Hammer

22.14 HAMMER SIZE SELECTION

It is important that the contractor and the engineer choose the proper hammer for efficient use on a given project. A hammer which is too small may not be able to drive the pile to the required capacity, or may require an excessive number of blows. On the other hand, a hammer which is too large may damage the pile. The use of empirical dynamic pile formulas to select a hammer energy should be discontinued because this approach incorrectly assumes these formulas result in the desired pile capacities. Results from these formulas become progressively worse as the complexity of the hammers increase.

A wave equation analysis, which considers the hammer cushion-pile-soil system, is the recommended method to determine the optimum hammer size. For preliminary equipment evaluation, Table 22-2 provides approximate minimum hammer energy sizes for ranges of ultimate pile capacities. This is a generalization of equipment size requirements that should be modified based on pile type, pile loads, pile lengths, and local soil conditions. In some cases, such as short piles to rock, a smaller hammer than indicated may be more suitable to control driving stresses. This generalized table should not be used in a specification. Guidance on developing a minimum energy table for use in a specification is provided in Chapter 12.

TABLE 22-2 PRELIMINARY HAMMER ENERGY REQUIREMENTS

Ultimate Pile Capacity (kN)	Minimum Manufacturers Rated Hammer Energy (Joules)
800 and under	16,500
800 to 1350	28,500
1351 to 1850	39,000
1851 to 2400	51,000
2401 to 2650	57,000

22.15 FOLLOWERS

A follower is a structural member interposed between the pile hammer and the pile, to transmit hammer blows to the pile head when the pile head is below the reach of the hammer. This occurs when the pile head is below the bottom of leads. Followers are sometimes used for driving piles below the deck of existing bridges, for driving piles underwater, or for driving the pile head below grade.

Maintaining pile alignment, particularly for batter piles, is a problem when a follower is used while driving below the bottom of the leads. The use of a follower is accompanied by a loss of effective energy delivered to the pile due to compression of the follower and losses in the connection. This loss of effective energy delivered to the pile affects the necessary driving resistance for the ultimate pile capacity. These losses can be estimated by an extensive and thorough wave equation analysis, or field evaluated by dynamic measurements. A properly designed follower should have about the same stiffness (per unit length) as the equivalent length of pile to be driven. Followers with significantly less stiffness should be avoided. Followers often require considerable maintenance. In view of the difficulties that can be associated with followers, their use should be avoided when possible. For piles to be driven underwater, one alternative is to use a hammer suitable for underwater driving. A photograph of a follower used to drive steel H-piles underwater is presented in Figure 22.26.

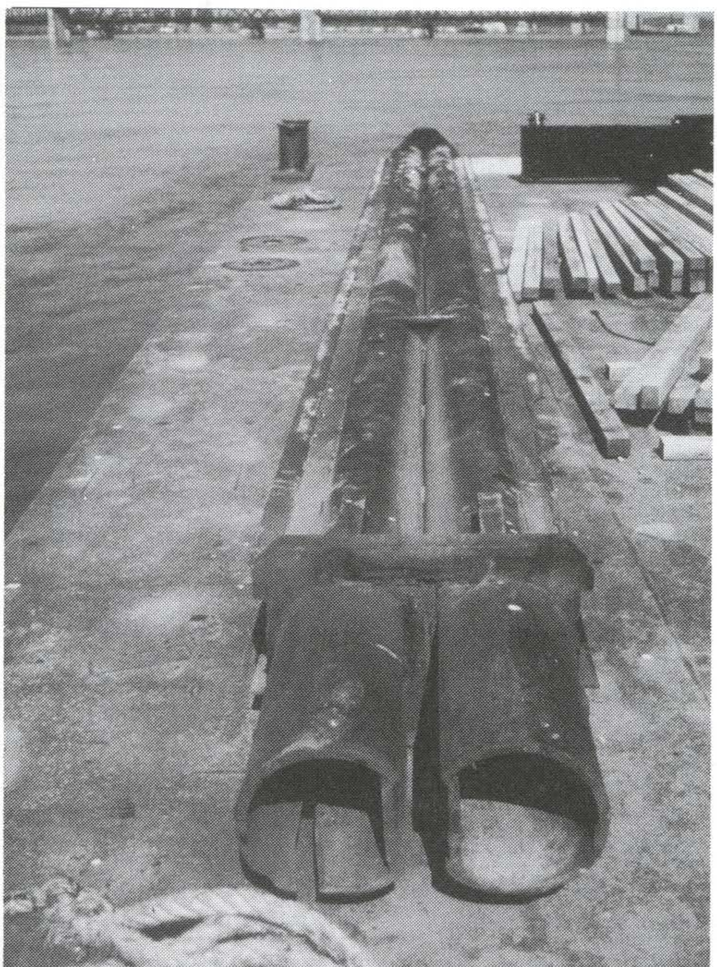


Figure 22.26 Follower used for Driving H-piles

22.16 JETTING

Jetting is the use of water or air to facilitate pile penetration by displacing the soil. In some cases, a high pressure air jet may be used in combination with water. Jets may be used to create a pilot hole prior to or simultaneously with pile placement. Jetting pipes may be located either inside or outside the pile. Jetting is usually most effective in loose to medium dense granular soils.

Jetting is not recommended for friction piles because the frictional resistance is reduced by jetting. Jetting should also be avoided if the piles are designed to provide substantial lateral resistance. For end bearing piles, the final required resistance must be obtained by driving (without jetting). Backfilling should be required if the jetted hole remains open after the pile installation. A separate pay item for jetting should be included in the contract documents when jetting is anticipated. Alternatives to jetting include predrilling and spudding.

The use of jetting has been greatly reduced due to environmental restrictions. Hence, jetting is rarely used unless containment of the jetted materials can be provided. Photographs of a dual jet system mounted on a concrete pile and a jet/punch system are presented in Figures 22.27 and 22.28, respectively.

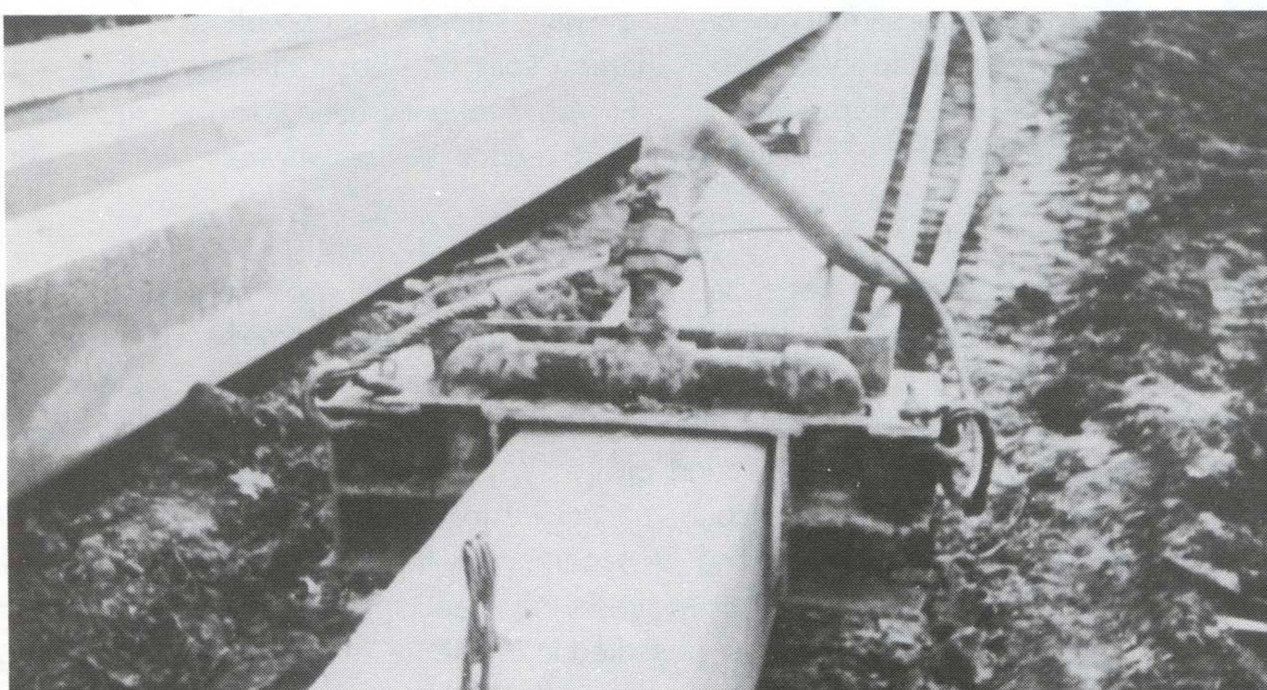


Figure 22.27 Dual Jet System Mounted on a Concrete Pile (courtesy of Florida DOT)

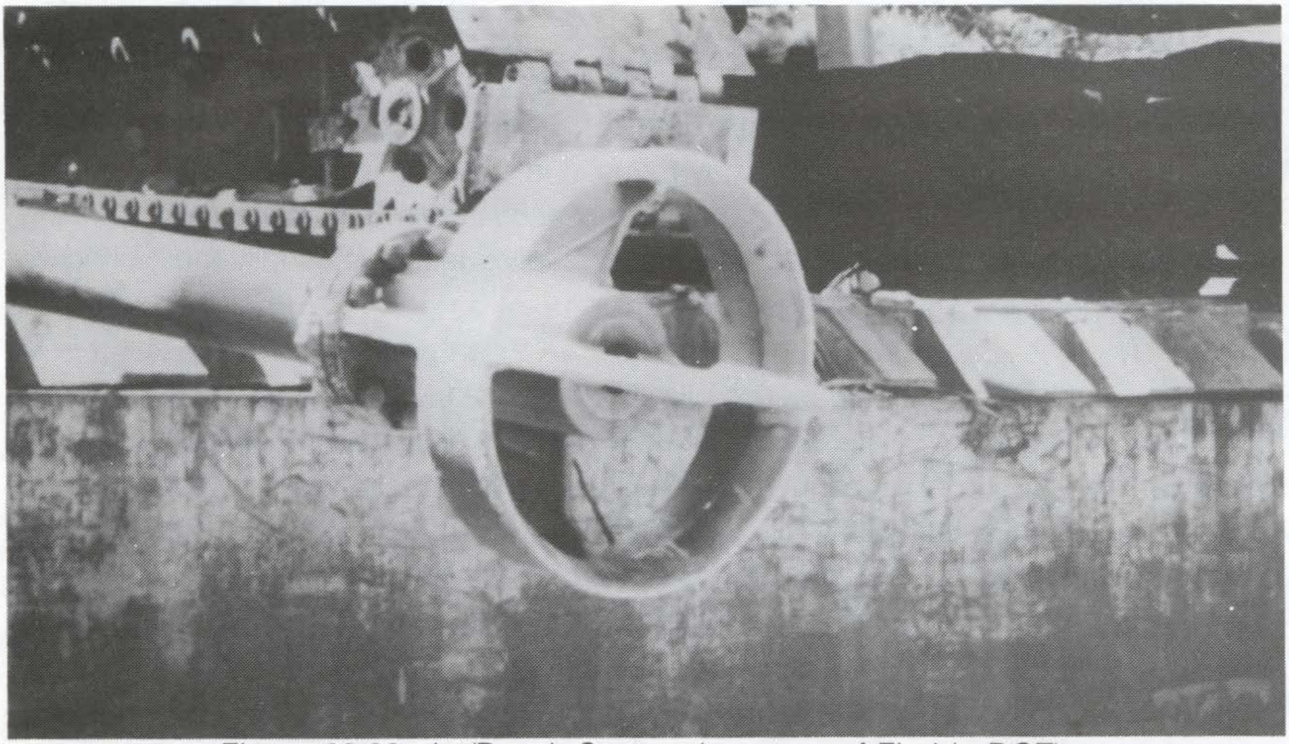


Figure 22.28 Jet/Punch System (courtesy of Florida DOT)

22.17 PREDRILLING

Soil augers or drills may sometimes be used where jetting is inappropriate. Predrilling is sometimes necessary to install a pile through soils with obstructions, such as old timbers, boulders, and riprap. Predrilling is also frequently used for pile placement through soil embankments and may be helpful to reduce pile heave when displacement piles are driven at close spacings.

The predrilled hole diameter depends upon the size and shape of the pile, and soil conditions. The hole should be large enough to permit driving but small enough so the pile will be supported against lateral movement. Under most conditions, the predrilled hole diameter should be 100 mm less than the diagonal of square or steel-H piling, and 25 mm less than the diameter of round piling. Where piles must penetrate into or through very hard material, it is usually necessary to use a diameter equal to the diagonal width or diameter of the piling. A separate pay item for predrilling should be included in the contract documents when predrilling is anticipated. A photograph of a solid flight auger predrilling system is presented in Figure 22.29.

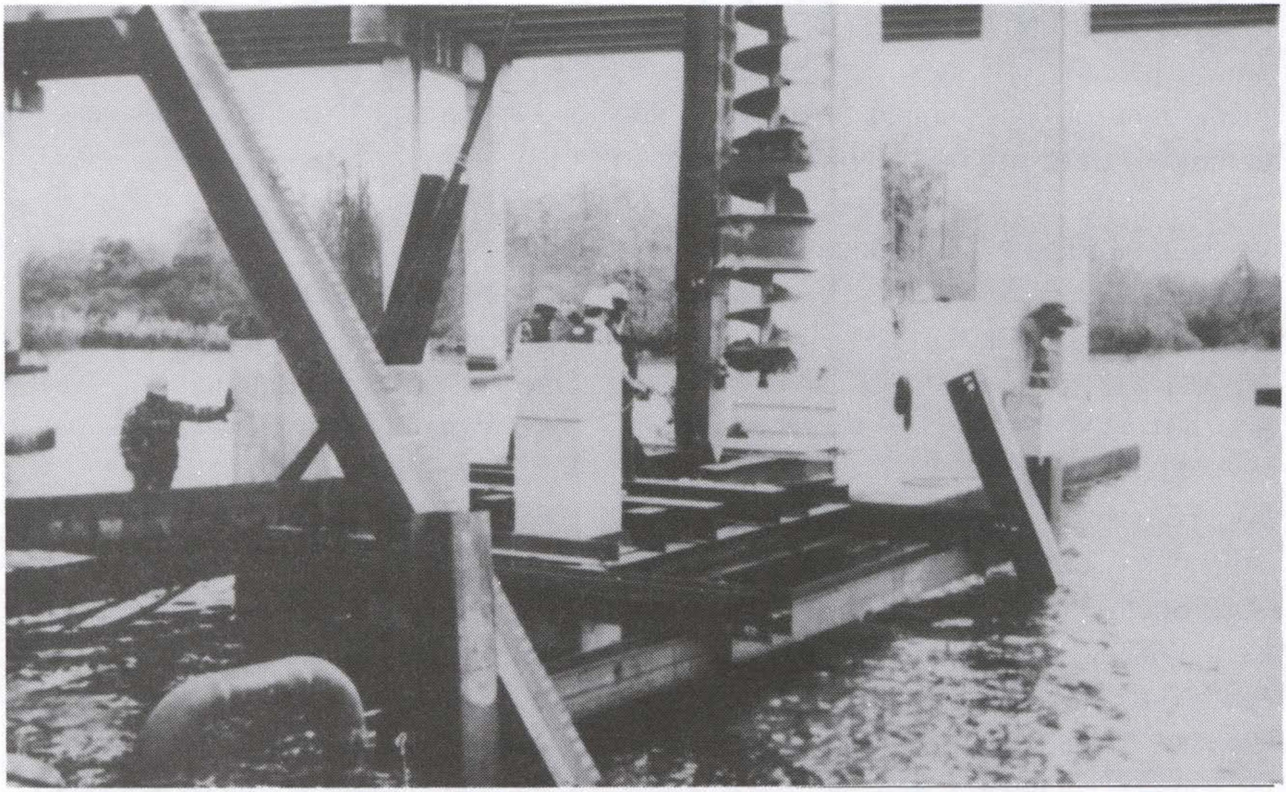


Figure 22.29 Solid Flight Auger Predrilling System (courtesy of Florida DOT)

22.18 SPUDDING

Spudding is the act of opening a hole through dense material by driving or dropping a short and strong member and then removing it. The contractor may resort to spudding in lieu of jetting or predrilling when the upper soils consist of miscellaneous fill and debris. A potential difficulty of spudding is that a spud may not be able to be pulled when driven too deep. However, an advantage of spudding is that soil cuttings and groundwater are not brought to the ground surface, which could then require disposal due to environmental concerns.

22.19 REPRESENTATIVE LIST OF U.S.A. HAMMER MANUFACTURERS AND SUPPLIERS

At the time of final printing this manual, the following manufacturers or suppliers of commonly used pile hammers were identified:

American Equipment & Fabricating Corp.
100 Water St.
East Providence, RI 02914
Ph: 401-438-2626
Fax: 401-438-0764

Supplier:
- Berminghammer Diesel Hammers
- Dawson Hydraulic Hammers
- Dawson Vibratory Hammers
- H&M Vibratory Hammers
- Vulcan Air Hammers

American Piledriving Equipment, Inc.
7032 South 196th
Kent, WA 98032
Ph: 206-872-1041 or 800-248-8498
Fax: 206-872-8710

Manufacturer:
- APE Hydraulic Hammers
- APE Vibratory Hammers

Berminghammer Foundation Equipment
Wellington Street Marine Terminal
Hamilton, Ontario L8L 4Z9
Ph: 905-528-0425 or 800-668-9432
Fax: 905-528-6187

Manufacturer:
- Berminghammer Diesel Hammers

Continental Machine Co., Inc.
1602 Engineers Road
Belle Chasse, LA 70037
Ph: 504-394-7330 or 800-259-7330
Fax: 504-393-8715

Manufacturer:
- Conmaco Air Hammers

Supplier:
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers
- MKT Diesel Hammers
- MKT Vibratory Hammers
- PTC Vibratory Hammers

Drive-Con, Inc.
8225 Washington Blvd.
Jessup, MD 20794
Ph: 410-799-8963 or 800-255-8963
Fax: 410-799-5264

Supplier:
ICE Diesel Hammers
ICE Hydraulic Hammers
ICE Vibratory Hammers
MKT Air Hammers

Equipment Corporation of America
P.O. Box 306
Coraopolis, PA 15108-0306
Ph: 412-264-4480
Fax: 412-264-1158

Supplier:
Delmag Diesel Hammers
HPSI Hydraulic Hammers
HPSI Vibratory Hammers
MKT Air Hammers
Tunkers Vibratory Hammers
Vulcan Air Hammers

L. B. Foster
415 Holiday Drive
Pittsburgh, PA 15220
Ph: 412-928-5625
Fax: 412-928-7891

Supplier:
IHC Hydraulic Hammers

Foundation Equipment Corporation
P.O. Box 566
270 S. Tuscarosa
Dover, OH 44622
Ph: 330-364-7521
Fax: 330-364-7524

Supplier:
FEC Diesel Hammers

Gardella Equipment Corporation
111 Harbor Avenue
Norwalk, CT 06850
Ph: 203-855-8160
Fax: 203-853-0342

Supplier:
Junttan Hydraulic Hammers

Geoquip, Inc.
1201 Cavalier Blvd.
Chesapeake, VA 23323
Ph: 757-485-2500
Fax: 757-485-5631

Supplier:
- Delmag Diesel Hammers
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers
- Menck Hydraulic Hammers
- Vulcan Air Hammers
- Vulcan Vibratory Hammers

Hammer and Steel, Inc.
11912 Missouri Bottom Road
St. Louis, MO 63042
Ph: 314-895-4600
Fax: 314-895-4070

Supplier:
- Dawson Hydraulic Hammers
- Delmag Diesel Hammers
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers

Hercules Machinery
Mid-America-Foundation Supply, Inc.
P.O. Box 5198
3101 New Haven Avenue
Fort Wayne, IN 46803
Ph: 219-424-0405 or 800-348-1890
Fax: 219-422-2040

Supplier:
- H&M Vibratory Hammers
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers
- ICE/Linkbelt Diesel Hammers
- ICE Hydraulic Hammers
- ICE Vibratory Hammers
- Vulcan Air Hammers
- Vulcan Vibratory Hammers

Hercules Machinery Corporation
8 Bryant Court
Sterling, VA 20166
Ph: 800-223-8427
Fax: 703-435-4530

Supplier:
- APE Vibratory Hammers
- Berminghammer Diesel Hammers
- ICE/Linkbelt Diesel Hammers
- Kobe Diesel Hammers
- MKT Air Hammers

Hydraulic Power Systems, Inc.
1203 Ozark
North Kansas City, MO 64116
Ph: 816-221-4774
Fax: 816-221-4591

Manufacturer:
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers

International Construction Equipment, Inc.
301 Warehouse Drive
Matthews, NC 28105
Ph: 704-821-8200 or 800-438-9281
Fax: 704-821-6448

Supplier:
- ICE Diesel Hammers
- ICE Hydraulic Hammers
- ICE Vibratory Hammers

Midwest Vibro Inc.
3715-28th Street S.W.
P.O. Box 224
Grandville, MI 49468-0224
Ph: 616-532-7670
Fax: 616-532-8505

Supplier:
- H&M Vibratory Hammers
- Dawson Vibratory Hammers
  (Dawson for Michigan only)

MKT Manufacturing, Inc.
1198 Pershall Road
St. Louis, MO 63137
Ph: 314-869-8600
Fax: 314-869-6862

Manufacturer:
- MKT Air Hammers
- MKT Diesel Hammers
- MKT Vibratory Hammers

New England Construction Products, Inc.
P.O. Box 1124
Taunton, MA 02780
Ph: 508-821-4450
Fax: 508-821-4438

Supplier:
- Conmaco Air Hammers
- Delmag Diesel Hammers
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers
- Menck Hydraulic Hammers

Pacific American Commercial Company
7400 Second Avenue South
P.O. Box 3742
Seattle, WA 98124
Ph: 206-762-3550 or 800-678-6379
Fax: 206-763-4232

Supplier:
- BSP Hydraulic Hammers
- Delmag Diesel Hammers
- HPSI Hydraulic Hammers
- HPSI Vibratory Hammers
- MKT Air Hammers
- MKT Diesel Hammers
- MKT Vibratory Hammers
- Tunkers Vibratory Hammers
- Vulcan Air Hammers
- Vulcan Vibratory Hammers

Pile Equipment, Inc.
1058 Roland Avenue
Green Cove Springs, FL 32043
Ph: 800-367-9416
Fax: 904-284-2588

Supplier:
- Delmag Diesel Hammers
- HPSI Hydarulic Hammers
- HPSI Vibratory Hammers
- Menck Hydraulic Hammers
- Vulcan Air Hammers
- Vulcan Vibratory Hammers

Pileco, Inc.
P.O. Box 16099
Houston, TX 77222
Ph: 713-691-3000
Fax: 713-691-0089

Supplier:
- Delmag Diesel Hammers
- Menck Hydraulic Hammers
- Tunkers Vibratory Hammers

Seaboard Steel Corporation
P.O. Box 3408
Sarasota, FL 34230
Ph: 941-355-9773 or 800-533-2736
Fax: 941-351-7064

Supplier:
- MKT Air Hammers
- MKT Diesel Hammers
- MKT Vibratory Hammers

Uddcomb Equipment A B
U. S. Representative - Sullivan Services
P.O. Box 385
San Andreas, CA 95249
Ph: 209-286-1290
Fax: 209-286-1290

Supplier:
- Uddcomb Hydraulic Hammers

Vulcan Iron Works
P.O. Box 5402
2909 Riverside Dr.
Chattanooga, TN 37406
Ph: 423-698-1581
Fax: 423-698-1587

Supplier:
- Vulcan Air Hammers
- Vulcan Diesel Hammers
- Vulcan Vibratory Hammers

REFERENCES

Associated Pile and Fitting Corporation. Design and Installation of Driven Pile Foundations.

Chellis, R.D. (1961). Pile Foundations. McGraw-Hill Book Company.

Compton, G.R. (1977). Piletalk Panel on Hammers and Equipment, APF Piletalk Seminar, San Francisco.

Davisson, M.T. (1975). Pile Load Capacity. Design, Construction and Performance of Deep Foundations, ASCE and University of California, Berkeley.

Davisson, M.T. (1980). Pile Hammer Selection. Presentation at the APF Piletalk Seminar in Carlsbad, California.

Deep Foundations Institute (1981). A Pile Inspector's Guide to Hammers, First Edition, Deep Foundations Institute.

Deep Foundations Institute (1995). A Pile Inspector's Guide to Hammers. Second Edition, Deep Foundations Institute.

Fuller, F.M. (1983). Engineering of Pile Installations. McGraw-Hill Book Company.

Goble, G.G. and Rausche F. (1981). WEAP Program, Vol. I Background. Implementation Package Prepared for the Federal Highway Administration.

Passe, Paul D. (1994). Pile Driving Inspector's Manual. State of Florida Department of Transportation.

Rausche, F., Likins, G.E., Goble, G.G. and Hussein, M. (1986). The Performance of Pile Driving Systems. Inspection Report, FHWA/RD-86/160, U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development, Washington, D.C., 92.

STUDENT EXERCISE #14 - EQUIPMENT SUBMITTAL REVIEW

Project specifications require the contractor to use a pile driving hammer having a minimum rated energy of 20.0 kJ to install the 20 m long, 305 mm square, prestressed concrete piles on this project. The piles have a required ultimate pile capacity of 1200 kN. Soil conditions consist of 15 m of soft clay over 20 meters of medium dense to dense sands. Static analyses indicate the piles should develop the required ultimate capacity at a penetration depth of 19 m. The Gates dynamic formula will be used for construction control.

The following pages contain the contractor's submittal package on this project. Based on the submittal, the final driving resistance required by the Gates formula is 56 blows per 0.25 m for the 1200 kN ultimate capacity. Review the submittal information and decide if the submittal should be approved. Do you have any questions or concerns ?

STEP 1 Check if hammer meets minimum energy requirements.

STEP 2 Determine line pressure loss in air hose between compressor and hammer by entering hose detail table on page 22-49 at compressor air delivery of 28 m^3/min. (Note, this table indicates the line loss in 15.2 m of hose.)

STEP 3 Check if the pressure at the hammer meets manufacturer's requirements.

STEP 4 Determine the rated energy based on the pressure at the hammer using the following manufacturer's formula for a differential hammer:

$$E_r = \left(W + A_{np}\,(p_h)\right) h$$

Where:
- E_r = rated energy (kJ).
- W = ram weight (kN).
- A_{np} = net area of piston (m^2).
- p_h = pressure at hammer (kPa).
- h = hammer stroke (m).

Equipment Submittal

Hammer: Vulcan 50-C differential acting air hammer.
Rated energy = 20.5 kJ at 0.39 m stroke.
(additional hammer details on page 22-49)

Hammer Cushion: 152 mm of Aluminum and Micarta.
Hammer Cushion Area = 641 cm^2.

Helmet: 4.6 kN

Pile Cushion: 100 mm of Plywood.
Pile Cushion Area = 930 cm^2.

Air Compressor: Model 1000
Rated Delivery: 28.3 m^3 / min.
Rated Pressure: 827 kPa.

Hose: 61 m of 51 mm I.D. (additional details on page 22-49).

Pile: 20 m long, 305 mm square precast, prestressed concrete
Compressive Strength: 40 MPa.
Effective Prestress after losses: 6 MPa.

Equipment Submittal

Hammer Details:

Ram Weight: 22.25 kN
Normal Stroke: 0.39 m
Rated Operating Pressure at Hammer: 827 kPa
Air Consumption: 24.9 m^3 / min
Required Air Compressor Size: 25.5 m^3 / min
Net Area of Piston: 0.036 m^2

Hose Details:

Hose		Pressure Loss in Hose (kPa)						
Inside Dia. (mm)	Length (m)	Air Delivery (m^3 / min)	Line Pressure (kPa) 414	552	690	827	1034	1378
51	15.2	16.8	13.1	------	------	------	------	------
		22.4	22.1	17.2	14.5	------	------	------
		28.0	34.5	26.9	22.1	18.6	12.2	11.7
		33.6	48.3	37.9	31.0	26.2	21.4	16.5
		39.2	64.1	51.0	42.1	35.9	29.0	22.1
		44.8	------	66.2	54.5	46.2	37.9	29.0
		50.4	------	83.4	69.3	57.9	47.6	36.5
		56.0	------	------	84.1	71.7	58.6	44.8

23. ACCESSORIES FOR PILE INSTALLATION

Pile accessories are sometimes used for pile toe protection and for splicing. Accessories available for driven piles can make installation easier and faster. They can also reduce the possibility of pile damage and help provide a more dependable permanent support for any structure. Heavier loading on piles, pile installation in sloping rock surfaces or into soils with obstructions, and longer pile length, are project situations where the use of pile shoes and splice accessories are often cost effective and sometimes necessary for a successful installation. However, pile accessories may add significant cost to the project and should not be used unless specifically needed. Pile toe attachments and splices for timber, steel, concrete and composite piles are discussed in this chapter. A list of the manufacturers and suppliers of pile accessories is provided at the end of this chapter.

During driving and in service, pile toe attachments and splices should develop the required strength in compression, bending, tension, shear, and torsion at the point of the toe attachment or splice. The current AASHTO Bridge Specifications require that a splice must provide the full strength of a pile. Some of the manufactured splices do not satisfy this AASHTO requirement.

23.1 TIMBER PILES

The potential problems associated with driving timber piles are splitting and brooming of the pile toe and pile head, splitting or bowing of the pile body, and breaking of the pile during driving. Protective attachments at the pile toe and at the pile head can minimize these problems.

23.1.1 Pile Toe Attachments

A timber pile toe can be protected by a metal boot or a point. The trend toward heavier hammers and heavier design loading may result in greater risk of damage for timber piles if obstructions are encountered. The pile toe attachment shown in Figure 23.1(a) and (c) covers the entire pile toe without the need for trimming. Figure 23.1(b) shows another type of pile toe protection attachment, which requires trimming of the pile toe.

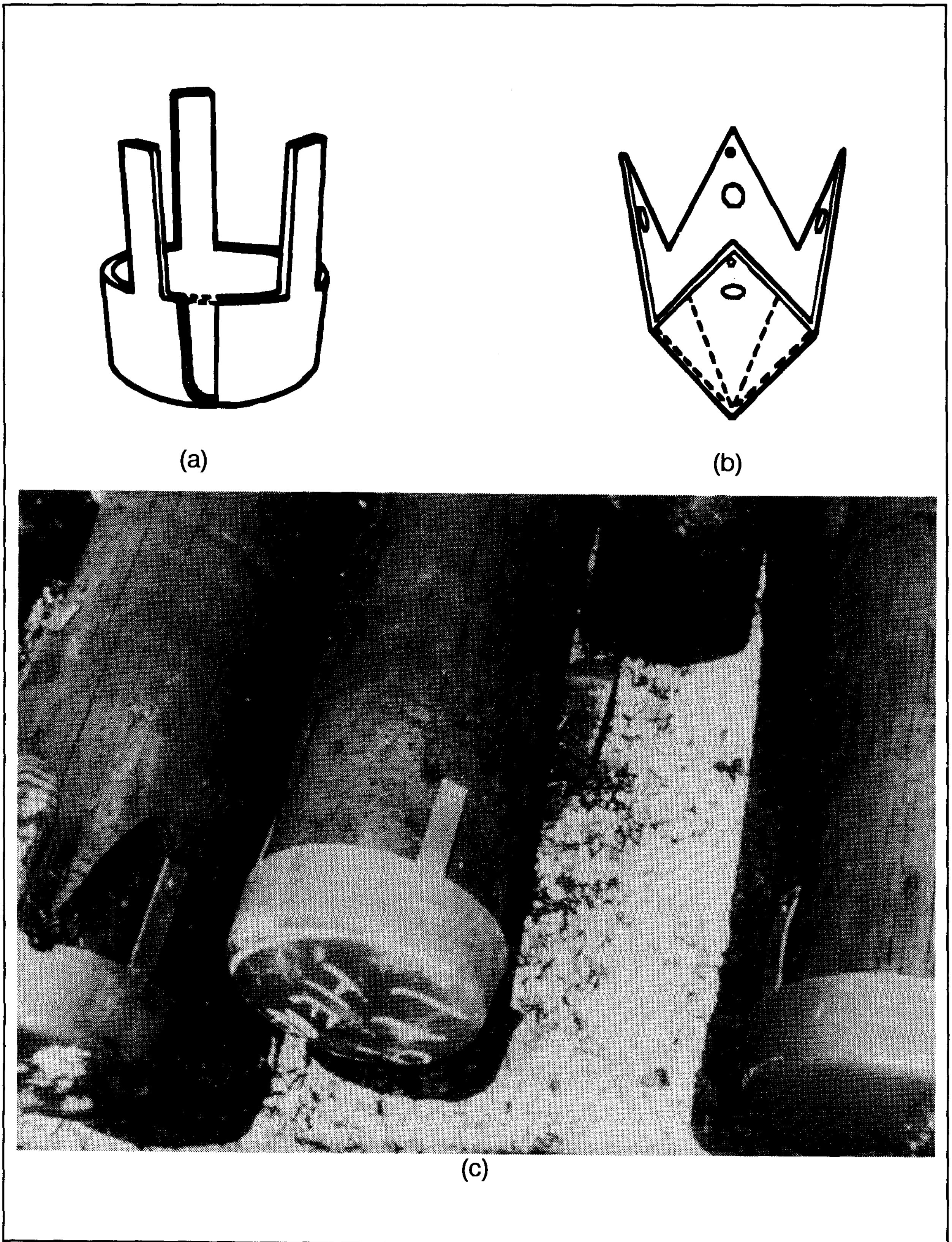


Figure 23.1 Timber Pile Toe Attachments

23.1.2 Attachment at Pile Head

The American Wood Preservers Institute (AWPI) recommends banding timber piles with heavy metal strapping at the pile head prior to driving to prevent splitting. A photograph of a banded timber pile head is shown in Figure 23.2

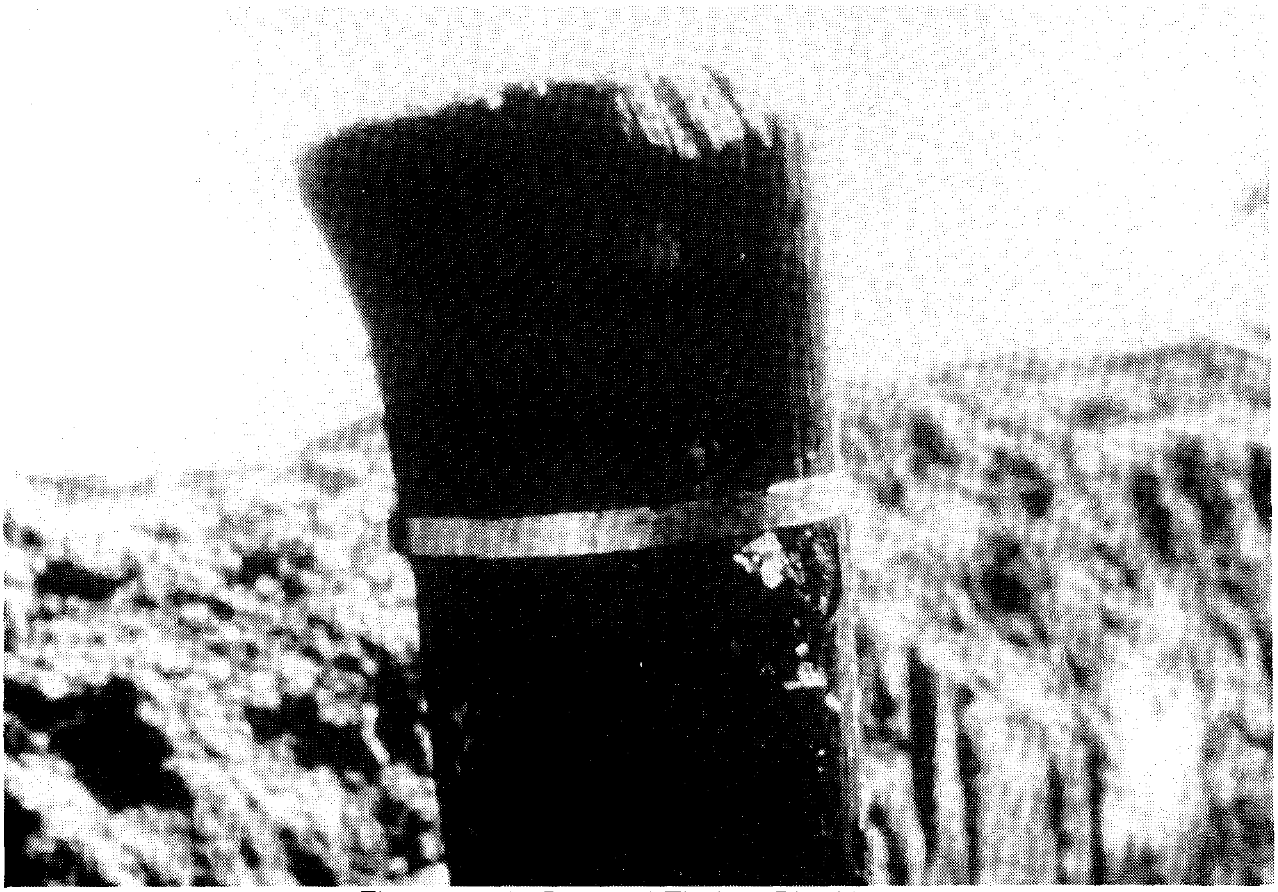


Figure 23.2 Banded Timber Pile Head

23.1.3 Splices

Timber pile splices are undesirable. It is virtually impossible to develop the full bending strength of the piling through simple splices such as those shown in Figure 23.3(a through c). In order to develop full bending strength, a detail similar to that shown in Figure 23.3(d) is required.

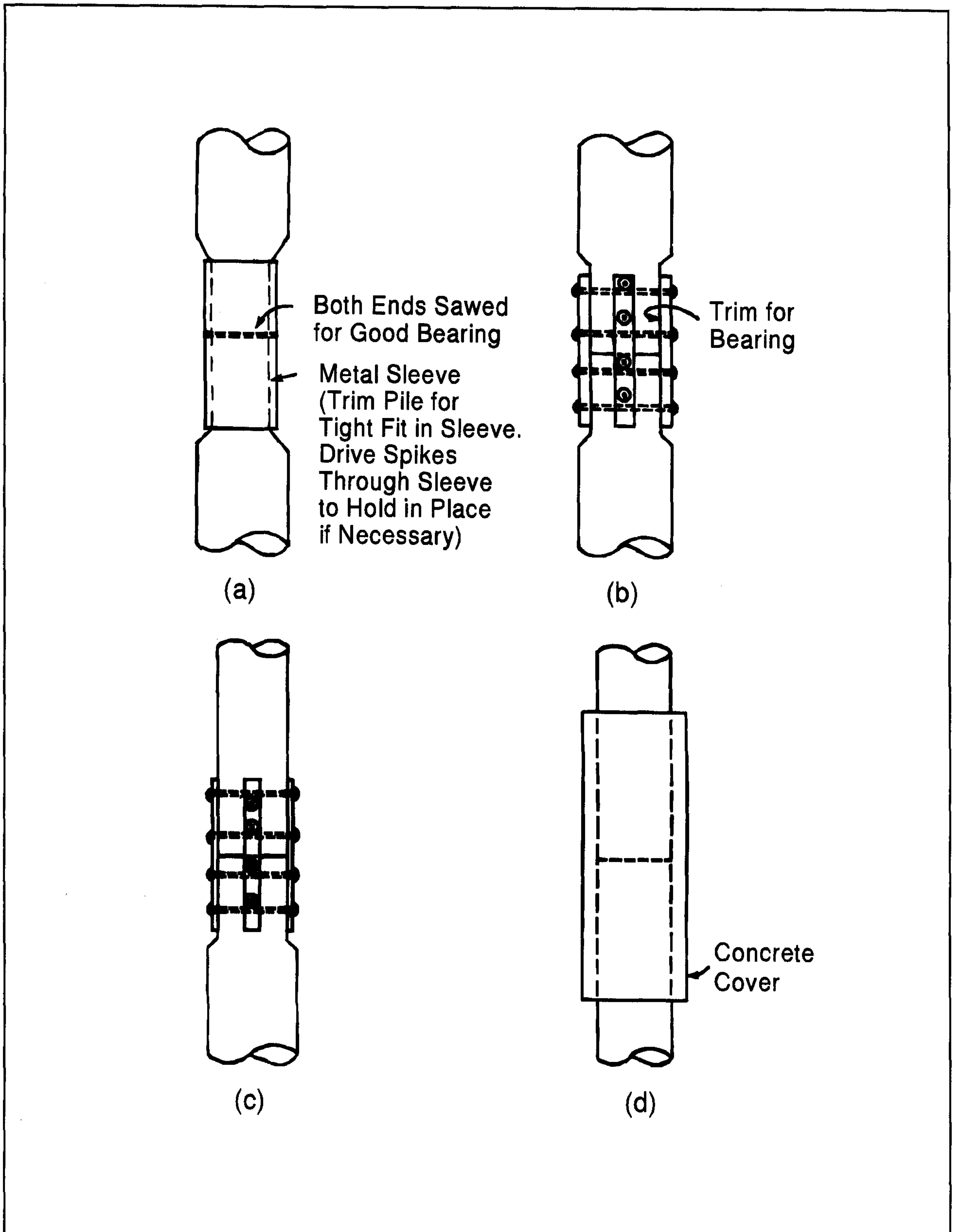


Figure 23.3 Splices for Timber Piles

23.2 STEEL H-PILES

23.2.1 Pile Toe Attachments

Steel H-piles are generally easy to install due to the non-displacement character of the pile. Problems arise when driving H-piles through man-made fills, very dense gravel or deposits containing boulders. If left unprotected under these conditions, the pile toe may deform to an unacceptable extent and separation of the flanges and web may occur (Figure 23.4). Pile toe attachments can help prevent these problems. Such attachments are also desirable for H-piles driven to rock, particularly on sloping rock surfaces.

Pile toe reinforcement consisting of steel plates welded to the flanges and web are not recommended because the reinforcement provides neither protection nor increased strength at the critical area of the flange-to-web connection. Several manufactured driving shoes are available, as shown in Figure 23.5(a through d). These shoes are attached to the H-piles with fillet welds along the outside of each flange. Pile shoes fabricated from cast steel (ASTM A 27) are recommended because of their strength and durability.

Prefabricated H-pile shoes come in various shapes and sizes. Manufacturers also recommend different shapes for various applications. It is recommended that for a given set of subsurface conditions, pile shoes from different manufacturers should be considered as equivalent if they are manufactured from similar materials and by similar fabrication techniques. Minor variations in configuration should be given minimum importance, except in specific subsurface conditions where a certain shape would give a definite advantage.

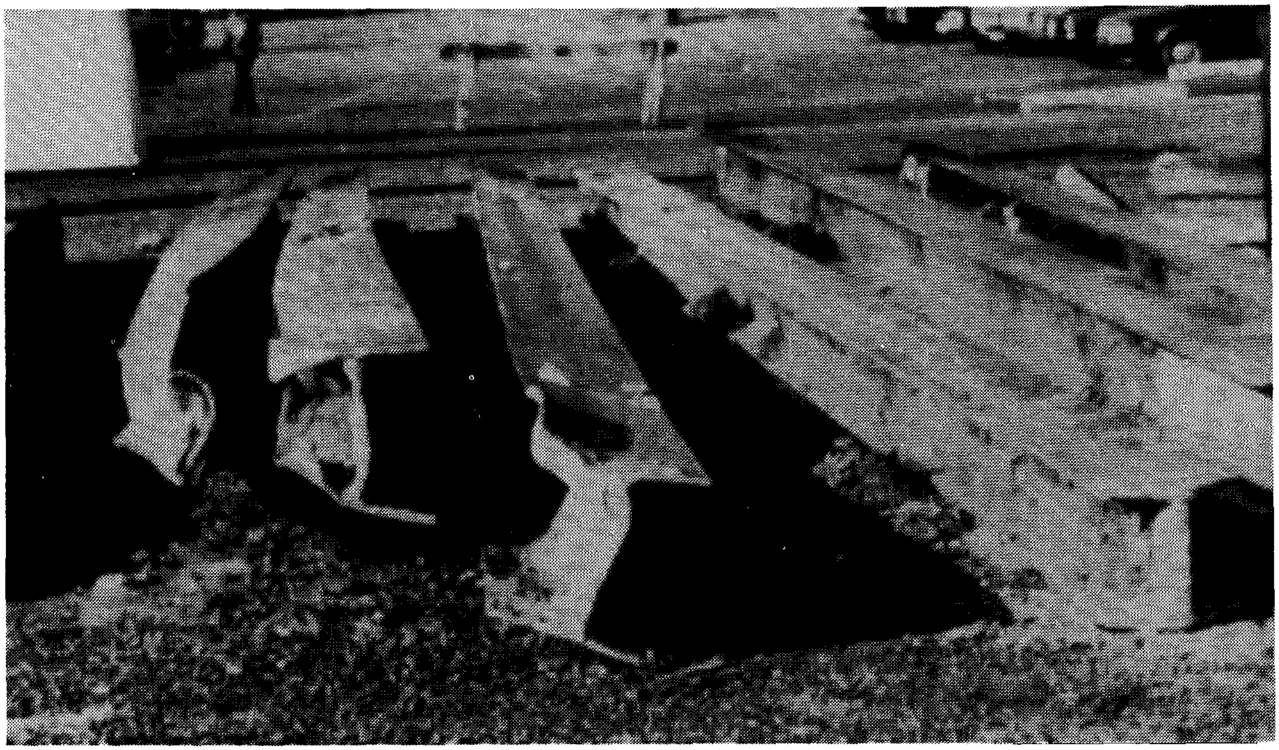


Figure 23.4 Damaged H-piles without Pile Toe Protection

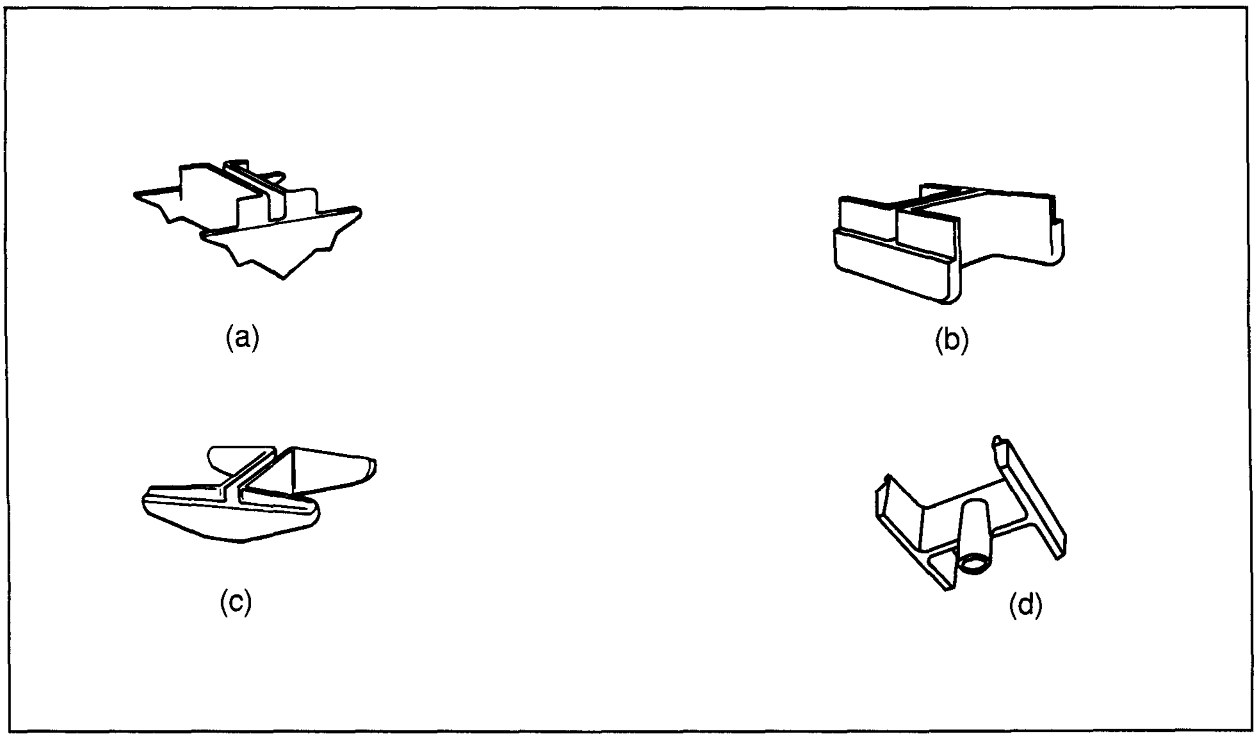


Figure 23.5 Driving Shoes for Protection of H-pile Toes

23.2.2 Splices

H-pile splices are routinely made by full penetration groove welding along the web and both flanges, or with manufactured splicers such as the ones shown in Figures 23.6(a) and 23.6(b). For the manufactured splicer shown, a notch is cut into the web of the driven section of pile and the splicer is slipped over the pile. Short welds are then made to the flanges near the corners of the splicer. The top section must have the flanges chamfered to achieve effective welding. Typically the section of pile to be added is positioned and held while welds across flanges are made. H-pile splicers are fabricated from ASTM A 36 steel. These splicers have been tested in the laboratory and the results have shown they provide full strength in bending as required by the AASHTO Bridge Specifications.

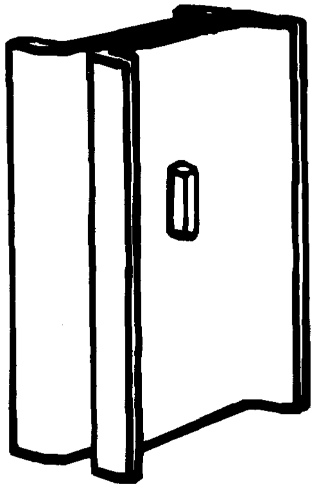
23.3 ACCESSORIES FOR STEEL PIPE PILES

23.3.1 Pile Toe Attachments

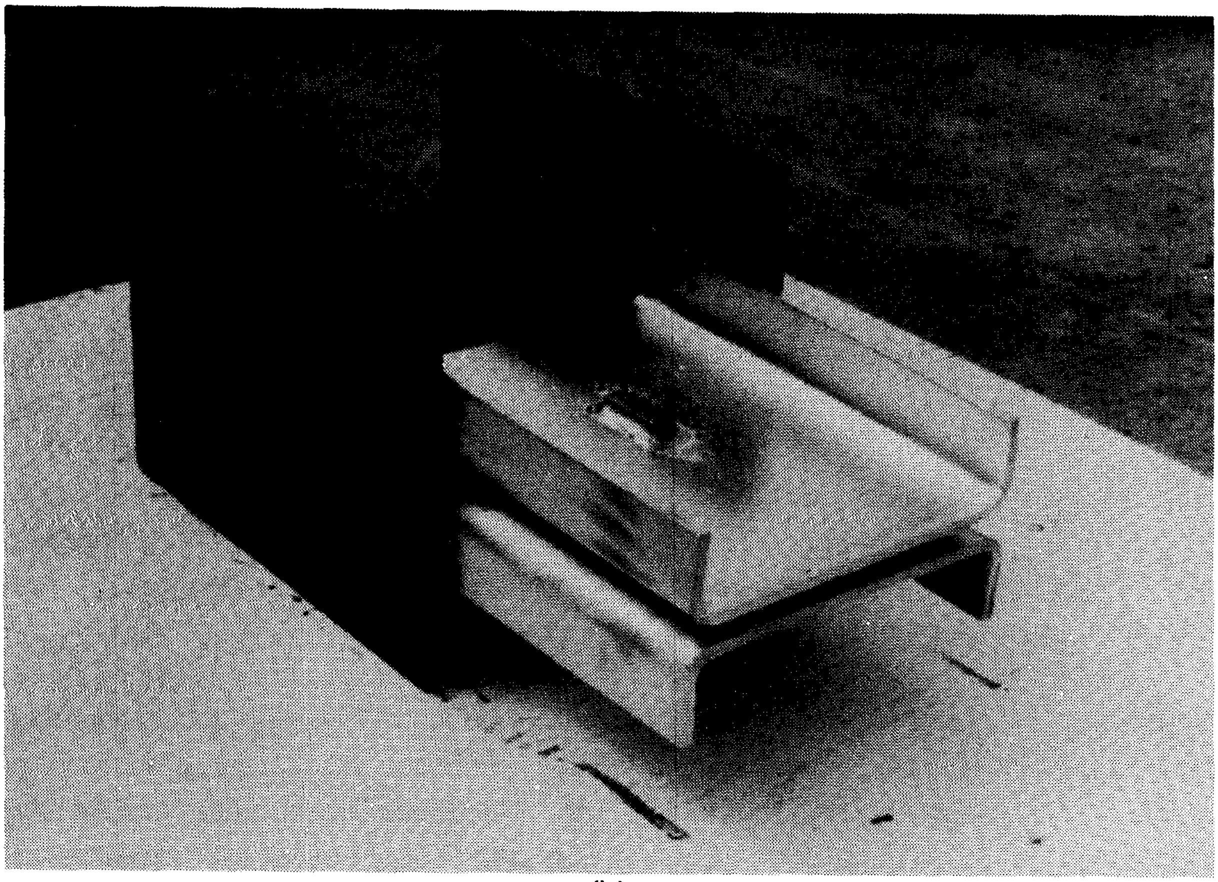
Problems during installation of closed end pipe piles arise when driving through materials containing obstructions. In this case, piles may deflect and deviate from their design alignment to an unacceptable extent. In case of driving open end pipe piles through or into very dense materials, the toe of the pile may be deformed. Pile toe attachments on closed end and open end piles are used to reduce the possibilities of damage and excessive deflection.

When pipe piles are installed with a closed end, a 12 to 25 mm thick flat plate is usually used as a form of toe protection. Conical toe attachments as shown in Figures 23.7(a) and 23.7(b) are also available as end-closures for pipe piles, although they generally cost more than flat plate type protection.

Generally, conical attachments have sixty degree configurations and are available with either an inside flange connection as shown in Figure 23.7(b) or outside flange connection as illustrated in Figure 23.7(a). The outside flange attachment can be driven with a press fit, so welding is not required. This additional benefit can save time and money.

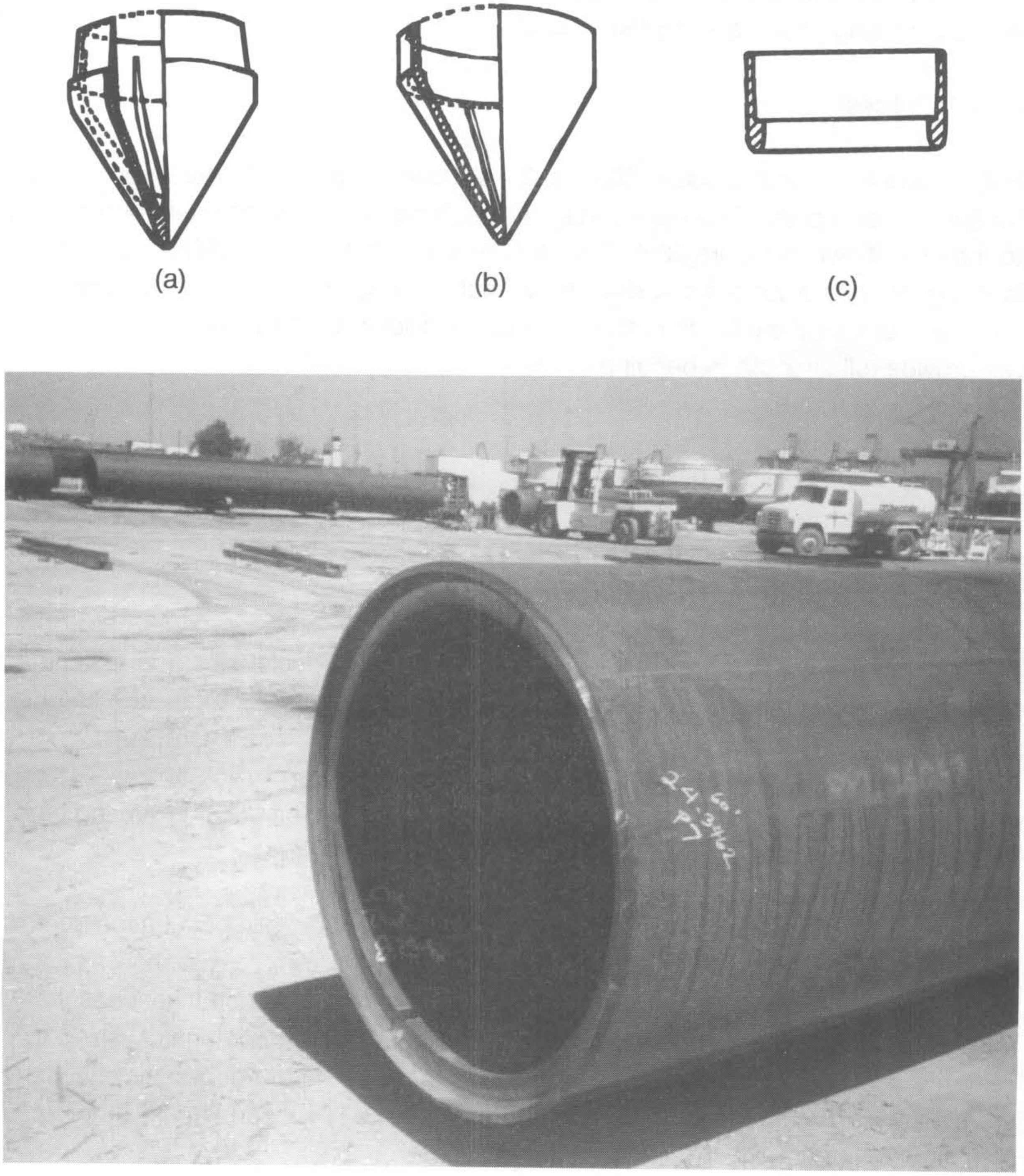


(a)



(b)

Figure 23.6 Typical H-pile Splicer



(a) (b) (c)

(d)

Figure 23.7 Pile Toe Attachments for Pipe Piles

When installing open end piles in dense gravel or to rock, the use of cutting shoes will help protect the piles and may make it possible to use thinner wall pipe. Cutting shoes are made from cast steel with a ridge for pile shoe bearing, as shown in Figure 23.7(c and d). Cutting shoes are welded to piles.

23.3.2 Splices

Full penetration groove welds or fillet welds as shown in Figure 23.8 are commonly used for splicing pipe piles. Pipe piles can also be spliced with manufactured splicers similar to the one shown in Figure 23.9. This splicer is fabricated from ASTM A 36 steel and is designed with a taper for a drive fit without welding so no advance preparation is required. Unless the drive fit or friction splicer is fillet welded to the pile, the splice will not provide full strength in bending.

23.4 PRECAST CONCRETE PILES

23.4.1 Pile Toe Attachments

The toe of precast concrete piles may be crushed in compression under hard driving. For hard driving conditions, or for end bearing on rock, special steel toe attachments can be used. Cast iron or steel shoes as depicted in Figure 23.10(a), or "Oslo Point" shown in Figure 23.10(b), are also used for toe protection. The characteristics of the Oslo Point are such that it can be chiseled into any type of rock to ensure proper seating. All toe attachments to precast concrete piles must be attached during casting of the piles and not in the field.

Another common type of toe attachment to increase concrete pile penetration depths in hard materials is a structural H sectional embedded in the pile, as shown in Figure 23.11. The H section extension is most often used to obtain additional penetration when uplift and scour are a concern. The H section should be proportionately sized to the concrete section to prevent overstressing and must be embedded sufficiently far for proper bonding and to develop bending strength. The H section should be protected by a H-pile toe attachment as discussed previously.

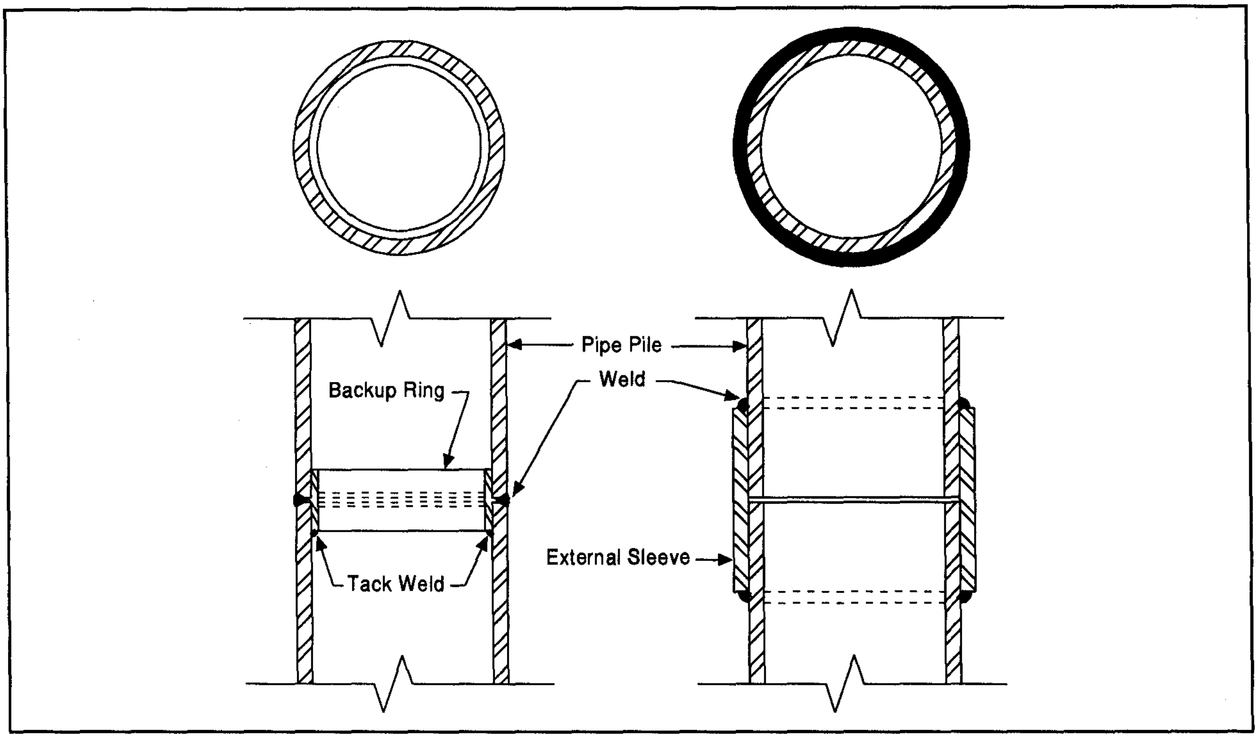


Figure 23.8 Splices for Pipe Piles

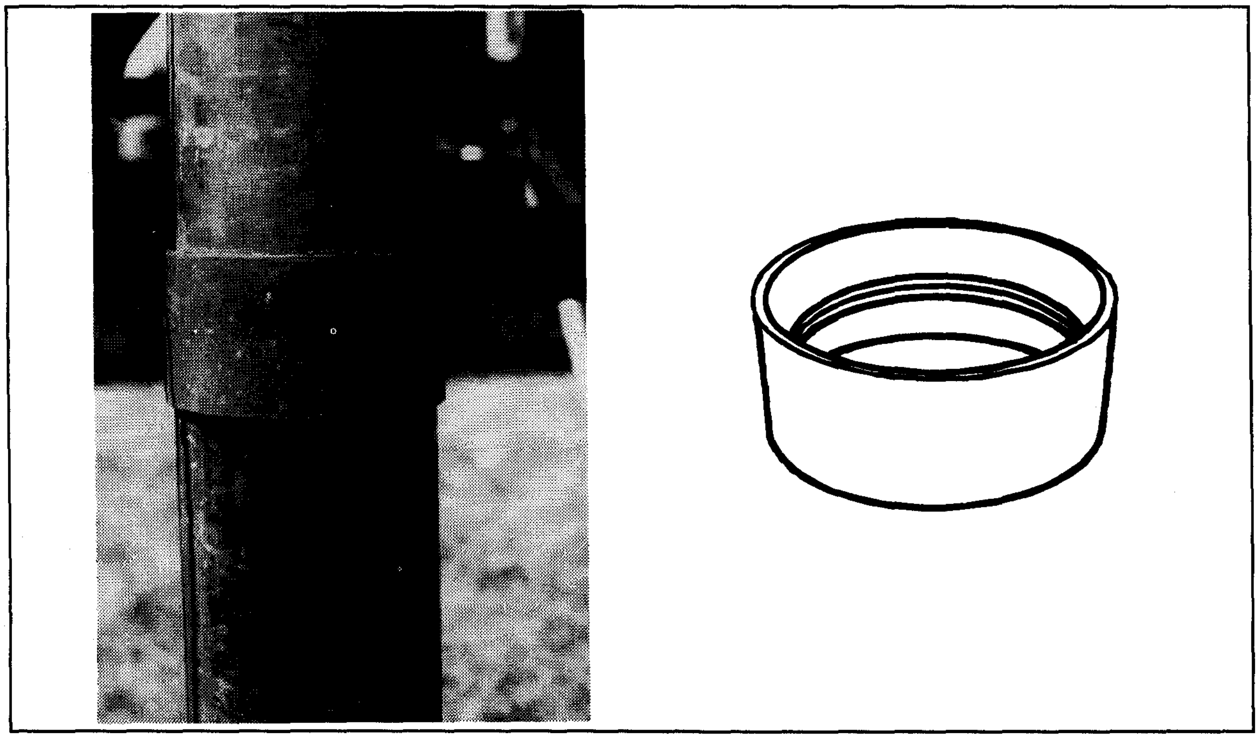


Figure 23.9 Splicer for Pipe Pile

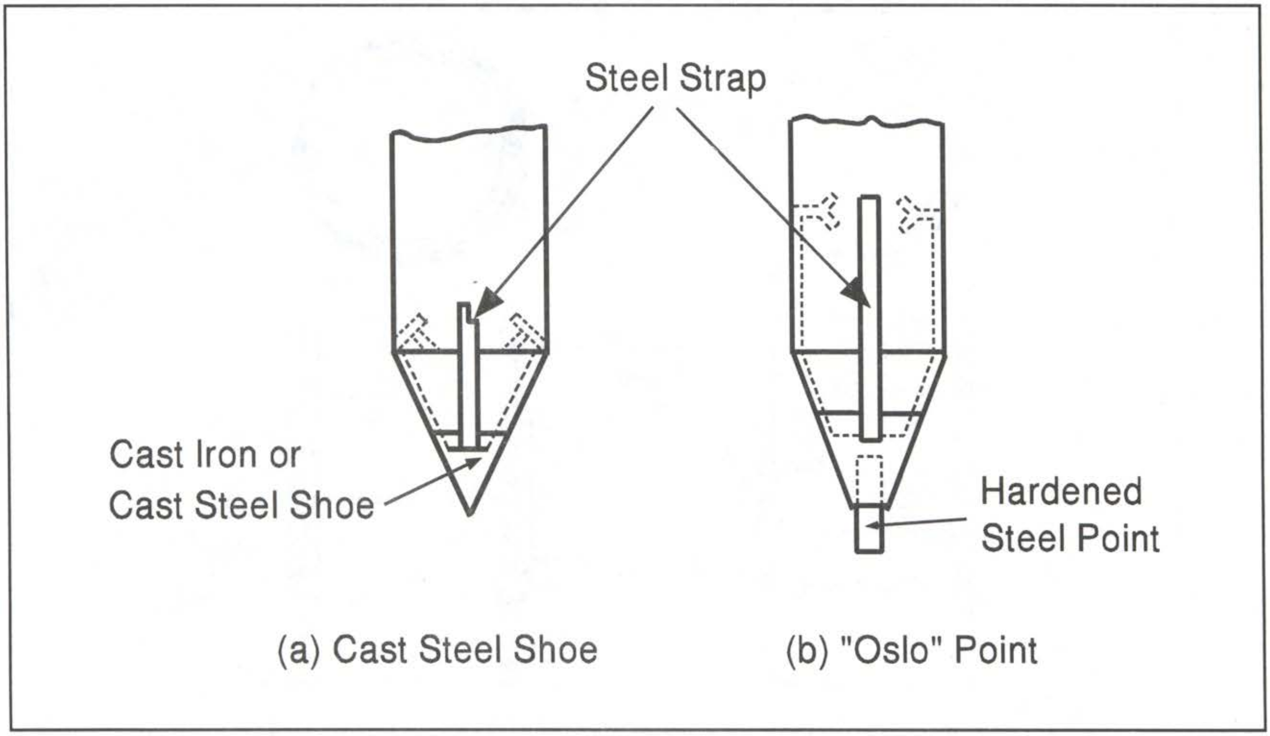


Figure 23.10 Pile Toe Attachments for Precast Concrete Piles

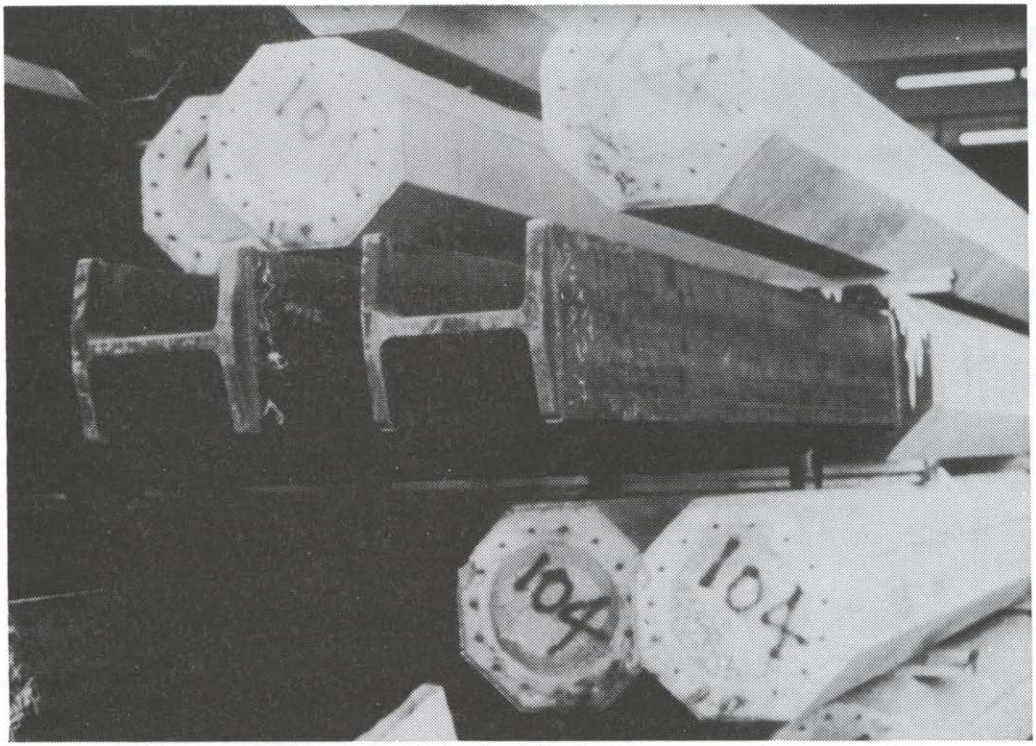


Figure 23.11 Steel H-pile Tip for Precast Concrete Pile

23.4.2 Splices

Most concrete piles driven in the United States are prestressed to minimize potential problems associated with handling and tension stresses during driving. However, the ends of prestressed concrete piles are not effectively prestressed due to development length, and thus special precautions must be taken when splicing prestressed concrete piles.

Table 23-1 from Bruce and Hebert (1974) shows a summary of splices for precast concrete piles. While this information is 20 years old, it still adequately summarizes the state of concrete pile splices. The table also provides guidelines concerning the compressive, tensile and flexural strength of the splice mechanisms. However, the actual performance of this and other splices should be evaluated on a project by project basis.

Whenever possible, concrete piles should be ordered with sufficient length to avoid splicing. However, if splicing is required, the splices available can be divided into four types: Dowel, Welded, Mechanical, and Sleeve. An overview of these splice types is given in Figure 23.12.

The generic epoxy dowel splice shown in Figure 23.13 can be used on prestressed and conventionally reinforced concrete piles. The bottom pile section to be spliced has holes which receive the dowels. These holes may be cast into the pile when splicing is planned, or drilled in the field when splicing is needed, but was unexpected. The bottom section is driven with no special consideration and the top section is cast with the dowel bars in the end of the pile. When spliced together in the field, the top section with the protruding dowels is guided and set in position and a thin sheet metal form is placed around the splice. Epoxy is then poured, filling the holes of the bottom section and the small space between the piles. The form can be removed after 15 minutes and driving resumed after curing of the epoxy. Dowel splices may be time consuming but are comparatively inexpensive. These splices have been proven reliable if dowel bars are of sufficient length and strength, and if proper application of the epoxy is provided. The number, length, and location of the dowel holes, as well as the dowel bar size, must be designed.

TABLE 23-1 SUMMARY OF PRECAST CONCRETE PILE SPLICES*							
					Strength		
Name of Splice	Type	Origin	Approximate Size Range (mm)	Approximate Field Time minutes	Percent Compressive	Percent Tensile	Percent Flexural Cracking
Marrier	Mechanical	Canada	254-330	30	100	100	100
Herkules	Mechanical	Sweden	254-508	20	100	100	100
ABB	Mechanical	Sweden	254-305	20	100	100	100
NCS	Welded	Japan	305-1195	60	100	100	100
Tokyu	Welded	Japan	305-1195	60	100	100	100
Raymond cyl.	Welded	USA	914-1372	90	100	100	100
Bolognesi-Mor.	Welded	Argentina	Varied	60	100	55	100
Japanese bolted	Bolted	Japan	Varied	30	100	90	90
Brunsplice	Connect-ring	USA	305-355	20	100	20	50
Anderson	Sleeve	USA	Varied	20	100	0	100
Fuentes	Weld-sleeve	Puerto Rico	254-305	30	100	100	100
Hamilton form	Sleeve	USA	Varied	90	100	75	100
Cement dowel	Dowel	USA	Varied	45	100	40	65
Macalloy	Post-tension	England	Varied	120	100	100	100
Mouton	Combination	USA	254-355	20	100	40	100
Raymond wedge	Welded wedge	USA	Varied	40	100	100	100
Pile coupler	Connect-ring	USA	305-1372	20	100	100	100
Nilsson	Mechanical	Sweden	Varied	20	100	100	100
Wennstrom	Wedge	Sweden	Varied	20	100	100	100
Pogonowski	Mechanical	USA	Varied	20	100	100	100

* (after Bruce and Herbert, 1974)

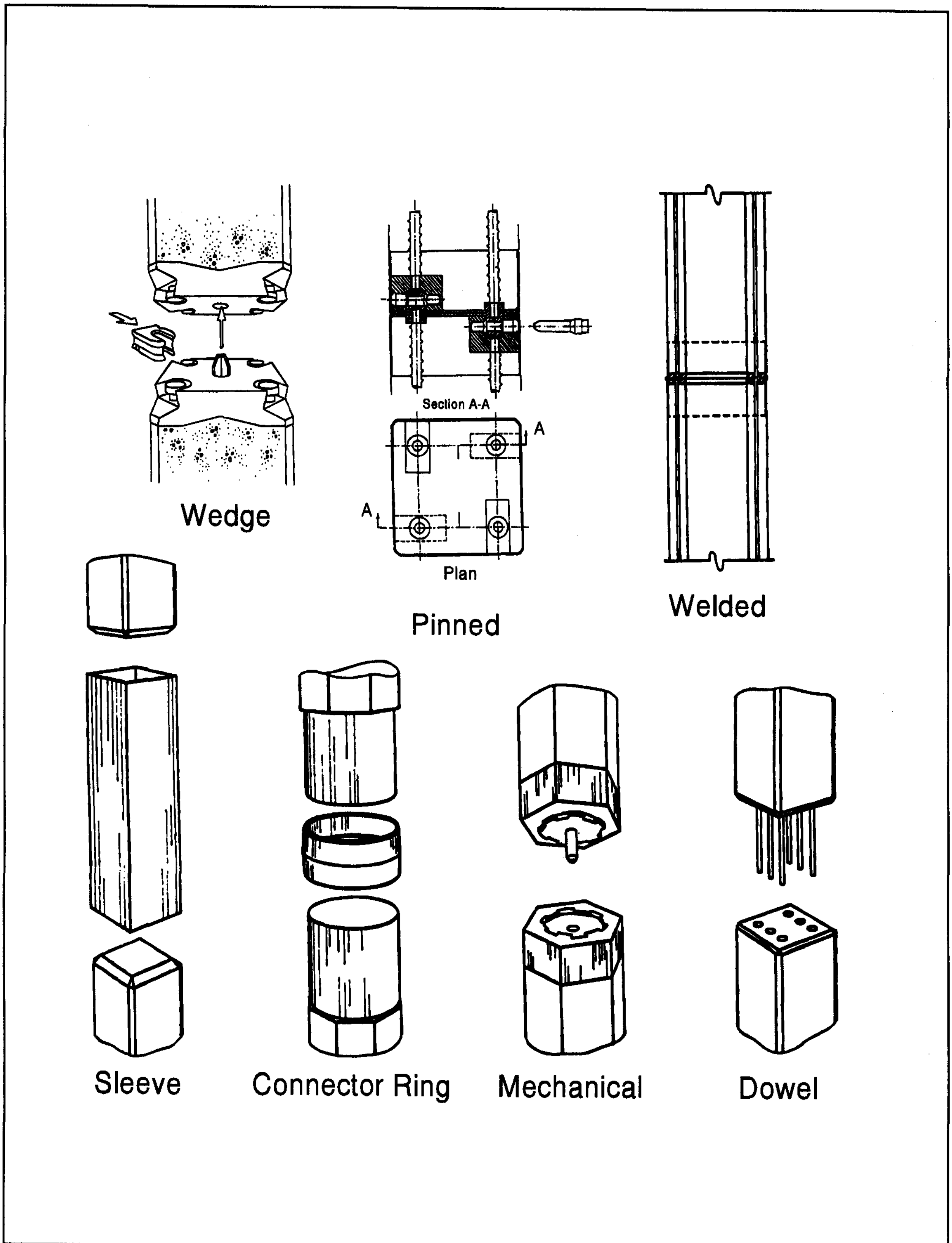


Figure 23.12 Commonly used Prestressed Concrete Pile Splices (after PCI, 1993)

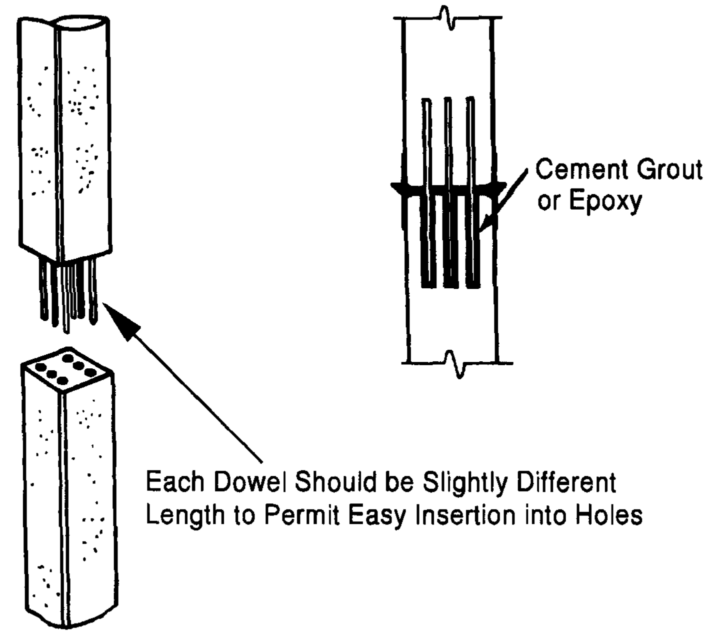


Figure 23.13 Cement-Dowel Splice (after Bruce and Herbert, 1974)

Welded splices require steel fittings be cast at the end of the sections to be spliced. The two sections are then welded around the entire perimeter. Most mechanical splices, such as the Herkules, Harddrive, Sure Lock, ABB, and Dyn-A-Splice, among others, are made of steel castings and are available for square, octagonal, hexagonal, and/or round sectional shapes. They can be used either for reinforced or prestressed concrete piles and are cast into the pile at the time of manufacture. The Herkules splice requires mating both male and female castings, while most other mechanical splices are gender neutral. All mechanical splices are then locked by inserting wedges, pins, keys, or other mechanical connections after aligning the sections. Although mechanical splices can be expensive, they do save considerable time and they have been designed to properly account for all loading conditions, including tension.

Sleeve type concrete splices can also be rapidly applied and are very effective in reducing tension driving stresses, but they cannot be used where static uplift loading will be required. The sleeve must have sufficient length and strength if lateral or bending loads are anticipated. The shorter connector ring design has limited tensile and flexural strength and is generally not recommended.

If a specific splice is specified based on previous experience, then an option for substituting some other concrete splice should not be allowed unless the substitute splicer is field tested. The alternative splice should be required to have equivalent compressive, tensile, and flexural strength to the originally specified splice. The substitute splicer can be tested by driving a number of spliced test piles and observing the performance.

23.5 A LIST OF MANUFACTURERS AND SUPPLIERS OF PILE ACCESSORIES

1. A-Joint Corporation (concrete splices)
   P.O. Box 317
   Voorhees, NJ 08043
   Ph: 609-767-0609; Fax: 609-767-7458

2. National Ventures, Inc., Division of Agra Industries Ltd. (concrete splices)
   198 Union Boulevard, Suite 200
   Lakewood, CO 80238
   Ph: 303-989-2800; Fax: 303-989-0667

3. Associated Pile and Fitting Corporation (shoes and splices)
   P.O. Box 1048
   Clifton, NJ 07014-1048
   Ph: 800-526-9047, 201-773-8400; Fax: 201-773-8442

4. Dougherty Foundation Products (shoes and splicers)
   P.O. Box 688
   Franklin Lakes, NJ 07417
   Ph: 201-337-5748; Fax: 201-337-9022

5. International Construction Equipment, Inc. (ICE)
   301 Warehouse Drive
   Matthews, NC 28105
   Ph: 800-438-9281; Fax: 704-821-6448

6. International Pipe Products
P.O. Box 546
Ambridge, PA 15003
Ph: 412-266-8110; Fax: 412-266-4766

7. Mid-America Foundation Supply, Inc. (shoes)
P.O. Box 5198
Fort Wayne, IN 46895
Ph: 800-348-1890, 219-422-8767; Fax: 219-422-2040

8. Versabite Foundations Accessories (shoes and splices)
19600 S.W. Cipole Road
Tualatin, OR 97062
Ph: 800-678-8772; Fax: 503-692-5939

REFERENCES

American Wood Preserves Institute (1981). Splicing Treated Timber Piling. AWPI Technical Guidelines for Pressure Treated Wood, 2.

American Wood Preservers Institute (1981). Specification for Strapping Timber Piling, PH.

Bruce, R.N. and Hebert, D.C. (1974). Splicing of Precast Prestressed Concrete Piles, Parts I and II, PCI Journal, Volume 19, No. 5 and 6, September-October and November-December, 1974.

Dougherty, J.J. (1978). Accessories for Pile Installation. Presentation made at the ASCE Metropolitan Section Seminar on Pile Driving and Installation.

PCI (1993). Precast/Prestressed Concrete Institute Journal. Volume 38, No. 2, March-April, 1993.

24. INSPECTION OF PILE INSTALLATION

Knowledgeable supervision and inspection play a very important role in the proper installation of pile foundations. The present trend in pile foundation design and construction is to use larger piles with higher load capacities, installed by larger equipment to achieve cost savings, made possible by advances in the state-of-the-art of design and construction methods. The inspection of these higher capacity pile installations becomes critical because of less redundancy (fewer piles required), and the smaller tolerances and factors of safety.

Inspection is only as good as the knowledge, experience and qualifications of the inspector. The inspector must understand the operation of the hammer and its accessories, the pile behavior, the soil conditions, and how these three components interact. Most pile installation problems are avoidable if a competent inspector uses systematic inspection procedures coupled with good communication and cooperation with the contractor. The inspector must be more than just a "blow counter". The inspector is the "eyes and ears" for the engineer and the owner. Timely observations, suggestions, reporting, and correction advice can ultimately assure the success of the project. **The earlier a problem or unusual condition is detected and reported by the inspector, the earlier a solution or correction in procedures can be applied, and hence a potentially negative situation can be limited to a manageable size.** If the same problem is left unattended, the number of piles affected increases, as do the cost of remediation and the potential for claims or project delays. Thus, early detection and reporting of any problem may be critical to keep the project on schedule and within budget.

An outline of inspection procedures and maintenance of pile driving records is provided in this chapter. Procedures and record keeping methods should be refined periodically as more experience is gathered by those responsible for construction operations.

24.1 ITEMS TO BE INSPECTED

There are several items to be checked by the inspector on every pile foundation project for test piles and/or production piles. Test piles may be driven for establishing order lengths or for load testing. Each of these items can be grouped under one of the following areas:

1. Review of the foundation design report, project plans and specifications prior to the arrival at the project site.

2. Inspection of piles prior to installation.

3. Inspection of pile driving equipment both before and during operation.

4. Inspection of test or indicator piles.

5. Inspection during production pile driving and maintenance of driving records.

A flow chart identifying the key components of the pile inspection process is presented in Figure 24.1.

24.2 REVIEW OF PROJECT PLANS AND SPECIFICATIONS

The first task of an inspector is to thoroughly review the project plans and specifications as they pertain to pile foundations. All equipment and procedures specified, including any indicator or test program of static and/or dynamic testing, should be clearly understood. If questions arise, clarification should be obtained from the originator of the specifications. The preliminary driving criteria should be known, as well as methods for using the test program results to adjust this criteria to site specific hammer performance and soil conditions. At this stage, the pile inspector should also determine the responsibility of his/her organization and should have answers to the following questions:

1. Is the inspector on the project in an observational capacity reporting to the foundation designer?, or

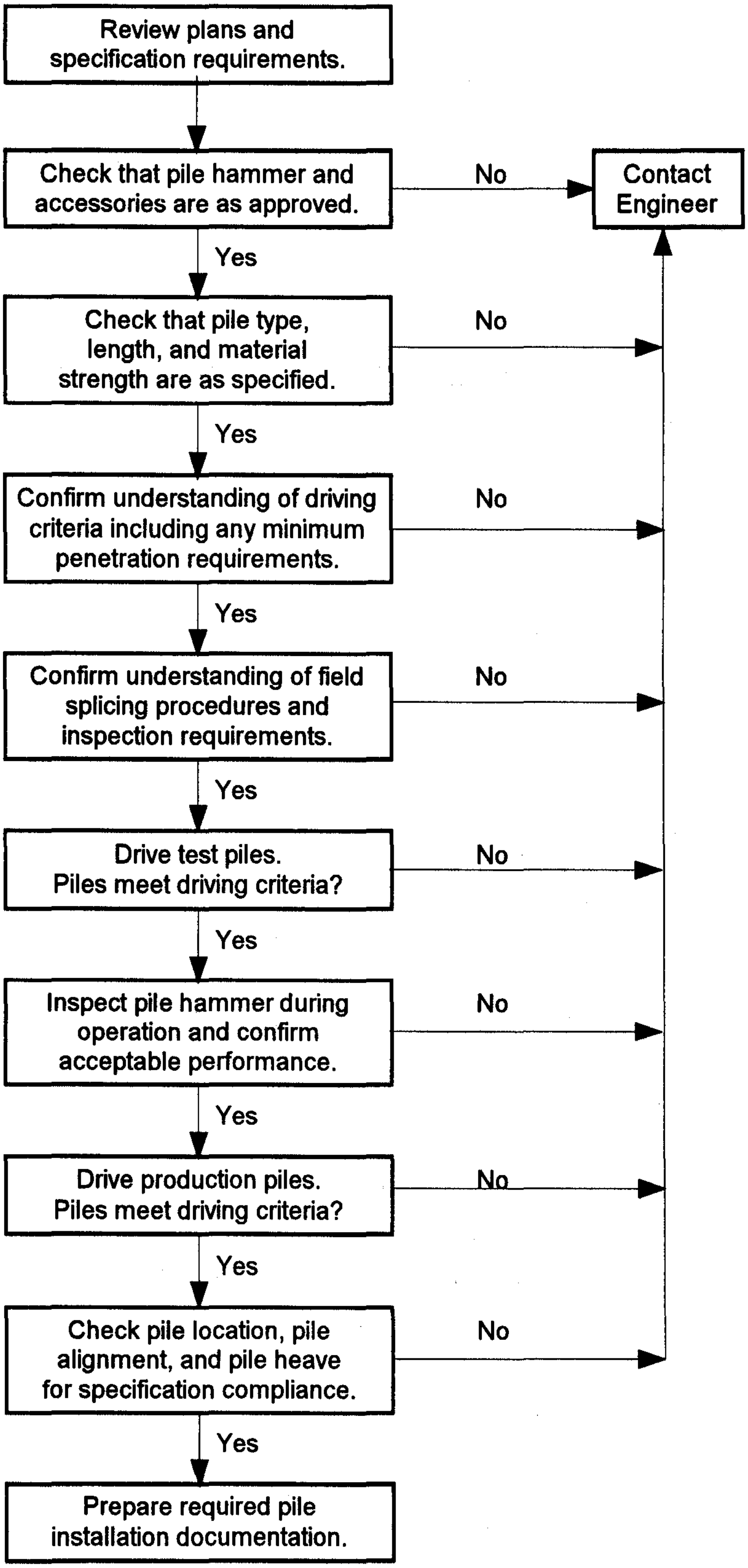


Figure 24.1 Pile Inspection Flow Chart

2. Does his/her organization have the direct responsibility to make decisions during driving of the test pile(s) and/or the production piles?

The inspector should also know:

1. Whom to contact if something goes wrong, and/or where to seek advice.

2. Whom to send copies of driving records and daily inspection reports.

3. What is required in the report during driving and at the completion of the project.

24.3 INSPECTOR'S TOOLS

The following check list, modified from Williams Earth Science (1995) summarizes the tools a pile inspector should have readily available to perform their job.

Approved Job Information
- ☐ Project Plans and Specifications with Revisions
- ☐ Special Provisions
- ☐ Pile Installation Plan
- ☐ Driving Criteria
- ☐ Casting/Ordered Lengths
- ☐ Approved Splice Detail

Daily Essentials
- ☐ Hard Hat
- ☐ Boots
- ☐ Ear Protection
- ☐ Pen/Pencil (and spare)
- ☐ Scale
- ☐ Measuring Tape
- ☐ Builder's Square
- ☐ Level
- ☐ Watch
- ☐ Calculator
- ☐ Camera

Indexed Notebook of Driven Piles
- ☐ Test Pile Program
- ☐ Production
- ☐ Construction Daily

Blank Forms
- ☐ Pile Driving Log
- ☐ Daily Inspection Reports
- ☐ Personal Diary

References
- ☐ State Standard Specifications
- ☐ Design and Construction of Driven Pile Foundations (Vol. II)
- ☐ Performance of Pile Driving Systems Inspectors Manual (FHWA/RD-86/160)

24.4 INSPECTION OF PILES PRIOR TO AND DURING INSTALLATION

The inspection check list will be different for each type of pile, but some items will be the same. A certificate of compliance for the piles is generally required by the specifications. The inspector should obtain this certificate from the contractor and compare the specification requirements with the information provided on the certificate. The following sections contain specific guidance for each major pile type.

24.4.1 Timber Piles

Physical details for round timber piles are sometimes referred to in the ASTM pile specification, ASTM D25. Regardless of the referenced specifications, the following items should be checked for compliance:

a. The timber should be of the specified species.

b. The piles should have the specified minimum length, and have the correct pile toe and butt sizes. The pile butt must be cut squarely with the pile axis.

c. The twist of spiral grain and the number and distribution of knots should be acceptable.

d The piles should be acceptably straight.

e. The piles must be pressure treated as specified.

f. The pile butts and/or toe may require banding as detailed in Chapter 23.

g. Steel shoes which may be specified must be properly attached. Details are provided in Chapter 23.

h. Pile splices, if allowed by plans and specifications, must meet the project requirements.

24.4.2 Precast Concrete Piles

On many projects, inspection and supervision of casting operations for precast concrete piles is provided by the State transportation department. Frequently, in lieu of this inspection, a certificate of compliance is required from the contractor. The following checklist provides items to be inspected at the casting yard (when applicable):

a. Geometry and other characteristics of the forms.

b. Dimensions, quantity, and quality of spiral reinforcing and prestressing steel strands, including a certificate indicating that the prestressing steel meets specifications.

c. If the pile is to have mechanical or welded splices, or embedded toe protection, the splice or toe protection connection details including number, size and lengths of dowel bars should be checked for compliance with the approved details and for the required alignment tolerance. They should be cast within tolerance of the true axial alignment.

d. Quality of the concrete (mix, slump, strength, *etc.*) and curing conditions.

e. Prestressing forces and procedures, including time of release of tension, which is related to concrete strength at time of transfer.

f. Handling and storage procedures, including minimum curing time for concrete strength before removal of piles from forms.

The following is a list of items for prestressed concrete piles to be inspected at the construction site:

a. The piles should be of the specified length and section. Many specifications require a minimum waiting period after casting before driving is allowed. Alternatively, the inspector must be assured that a minimum concrete strength has been obtained. If the piles are to be spliced on the site, the splices should meet the specified requirements (type, alignment, *etc.)*.

b. There should be no evidence that any pile has been damaged during shipping to the site, or during unloading of piles at the site. Lifting hooks are generally cast into the piling at pick-up points. Piles should be unloaded by properly sized and tensioned slings attached to each lifting hook. Piles should be inspected for cracks or spalling.

c. The piles should be stored properly. When piles are being placed in storage, they should be stored above ground on adequate blocking in a manner which keeps them straight and prevents undue bending stresses.

d. The contractor should lift the piles into the leads properly and safely. Cables looped around the pile are satisfactory for lifting. Chain slings should never be permitted. Cables should be of sufficient strength and be in good condition. Frayed cables are unacceptable and should be replaced. For shorter piles, a single pick-up point may be acceptable. The pick-up point locations should be as specified by the casting yard. For longer piles, two or more pick-up points at designated locations may be required.

e. The pile should be free to twist and move laterally in the helmet.

f. Piles should have no noticeable cracks when placed in leads or during installation. Spalling of the concrete at the top or near splices should not be evident.

24.4.3 Steel H-Piles

The following should be inspected at the construction site:

a. The piles should be of the specified steel grade, length, or section/weight.

b. Pile shoes, if required for pile toe protection, should be as specified. Pile shoe details are provided in Chapter 23.

c. Splices should be either proprietary splices or full penetration groove welds as specified. The top and bottom pile sections should be in good alignment before splicing. Pile splice details are discussed in Chapter 23.

d. Pile shoe attachments and splices must be welded properly.

e. The piles being driven must be oriented with flanges in the correct direction as shown on the plans. Because the lateral resistance to bending of H-piles is considerably more in the direction perpendicular to flanges, the correct orientation of H-piles is very important.

f. There should be no observable pile damage, including deformations at the pile head.

24.4.4 Steel Pipe Piles

The following should be inspected at the construction site:

a. The piles should be of specified steel grade, length, or minimum section/weight (wall thickness) and either seamless or spiral welded as specified.

b. Piles should be driven either open-ended or closed-ended. Closed-ended pipe piles should have bottom closure plates or conical points of the correct size (diameter and thickness) and be welded on properly, as specified. Open end pipe piles should have cutting shoes that are welded on properly.

c. The top and bottom pile sections should be in good alignment before splicing. Splices or full penetration groove welds should be installed as specified. Pile splice details are discussed in Chapter 23.

d. There should be no observable pile damage, including deformations at the pile head. After installation, closed-end pipes should be visually inspected for damage or water prior to filling with concrete.

24.5 INSPECTION OF DRIVING EQUIPMENT

A typical driving system consists of crane, leads, hammer, hammer cushion, helmet, and in the case of concrete piles, a pile cushion. As discussed in Chapter 22, each component of the drive system has a specific function and plays an important role in the pile installation. The project plans and specifications may specify or restrict certain items of driving equipment. The inspector must check the contractor's driving equipment and obtain necessary information to determine conformity with the plans and specifications prior to the commencement of installation operations.

The following checklist will be useful in the inspection of driving equipment before driving:

1. The pile driving hammer should be the specified type/size.

   Usually the specifications require certain hammer types and/or specify minimum and/or maximum energy ratings. A listing of hammer energy ratings is provided in Appendix D. The inspector should make sure for single acting air/steam or hydraulic hammers that the contractor uses the proper size external power source and that, for adjustable stroke hammers, the stroke necessary for the required energy be obtained. For double acting or differential air/steam or hydraulic hammers, the contractor must again obtain the proper size external power source and the operating pressure and volume must meet the hammer manufacturer's specification. For open end diesel hammers, the inspector should obtain a chart for determining stroke from visual observation, or alternatively have available a device for electronically estimating the stroke from the blow rate. For closed end diesel hammers, the contractor should supply the inspector with a calibration certificate for the bounce chamber pressure gauge and a chart which correlates the bounce chamber pressure with the energy developed by the hammer. The bounce chamber pressure gauge should be provided by the contractor.

2. The hammer cushion being used should be checked to confirm it is of the approved material type, size and thickness.

   The main function of the hammer cushion is to protect the hammer itself from fatigue and high frequency accelerations which would result from steel to steel impact with the helmet and/or pile. The hammer cushion should have the proper material and same shape/area to snugly fit inside the helmet (drive cap). If the cushion diameter is too small, the cushion will break or badly deform during hammer blows and become ineffective. The hammer cushion must not be excessively deformed or compressed. Some air/steam hammers rely upon a certain total thickness (of cushion plus striker plate) for proper valve timing. Hammers with incorrect hammer cushion thickness may not operate, or will have improper kinetic energy at impact. Since it is difficult to inspect this item once the driving operation begins, it should be checked before the contractor starts pile driving on a project as well as periodically during production driving on larger projects. A photograph of a hammer cushion check is presented in Figure 24.2.

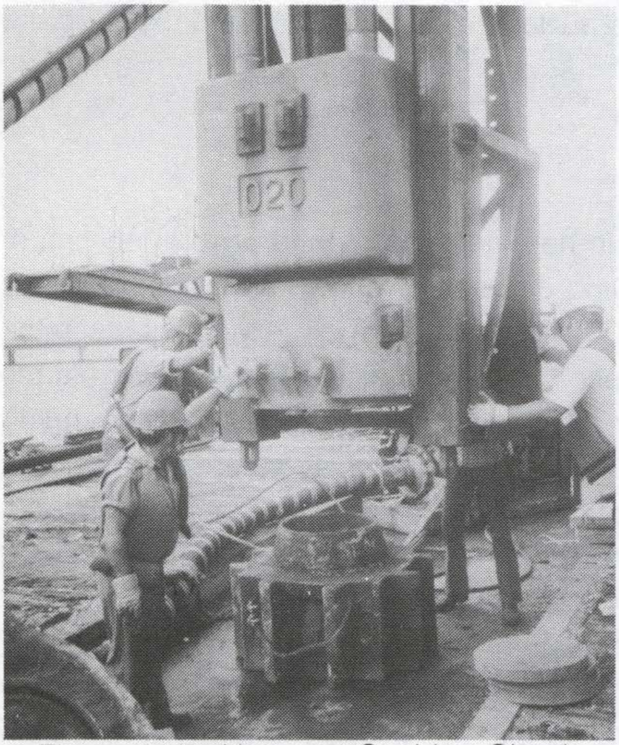


Figure 24.2 Hammer Cushion Check

Figure 24.3 Damaged Hammer Cushion

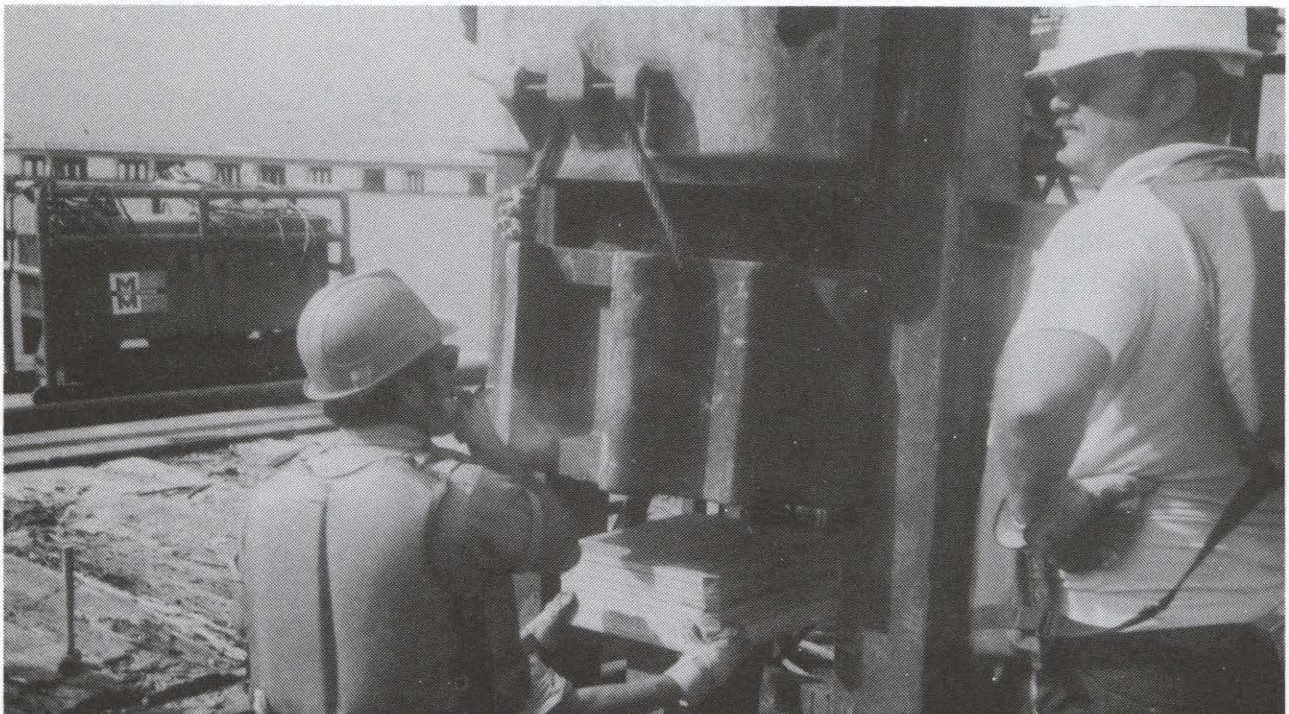


Figure 24.4 Pile Cushion Replacement

The hammer cushion material disks are shown in the lower right corner of the photograph. A damaged hammer cushion detected by a hammer cushion check is shown in Figure 24.3.

3. The helmet (drive cap) should properly fit the pile.

   The purpose of the helmet is to hold the pile head in alignment and transfer the impact concentrically from the hammer to the pile. The helmet also houses the hammer cushion, and must accommodate the pile cushion thickness for concrete piles. The helmet should fit loosely to avoid transmission of torsion or bending forces, but not so loosely as to prevent the proper alignment of hammer and pile. Helmets should ideally be of roughly similar size to the pile diameter. Although generally discouraged, spacers may be used to adapt an oversize helmet, provided the pile will still be held concentrically with the hammer. A properly fitting helmet is important for all pile types, but is particularly critical for precast concrete piles. <u>A poorly fitting helmet often results in pile head damage</u>. Check and record the helmet weight for conformance to wave equation analysis or for future wave equation analysis. Larger weights will reduce the energy transfer to the pile.

4. The pile cushion should be of correct type material and thickness for concrete piles.

   <u>The purpose of the pile cushion is to reduce high compression stresses, to evenly distribute the applied forces to protect the concrete pile head from damage, and to reduce the tension stresses in easy driving</u>. Pile cushions for concrete piles should have the required thickness determined from a wave equation analysis but not less than 100 mm. A new plywood, hardwood, or composite wood pile cushion, which is not water soaked, should be used for every pile. The cushion material should be checked periodically for damage and replaced before excessive compression (more than half the original thickness), burning, or charring occurs. Wood cushions may take only about 1,000 to 2,000 blows before they deteriorate. During hard driving, more than one cushion may be necessary for a single pile. Longer piles or piles driven with larger hammers may require thicker pile cushions. A photograph of a pile cushion being replaced is presented in Figure 24.4.

5. Predrilling, jetting or spudding equipment, if specified or permitted, should be available for use and meet the requirements. <u>The depth of predrilling, jetting or spudding should be very carefully controlled so that it does not exceed the</u>

allowable limits. Predrilling, jetting, or spudding below the allowed depths will generally result in a reduced pile capacity, and the pile acceptance may become questionable. Additional details on predrilling, jetting, and spudding are presented in Chapter 22.

6. The lead system being used must conform to the requirements, if any, in the specifications. Lead system details are presented in Chapter 22.

   The leads perform the very important function of holding the hammer and pile in good alignment with each other. Poor alignment reduces energy transfer as some energy is then imparted into horizontal motion. Poor alignment also generally results in higher bending stresses and higher local contact stresses which can cause pile damage. This is particularly important at end of driving when driving resistance is highest and driving stresses are generally increased. Sometimes the specifications do not allow certain lead systems or may require a certain type system. A pile gate at the lead bottom which properly centers the pile should be required, as it helps maintain good alignment.

Note: On most projects, a wave equation analysis is used to determine preliminary driving criteria for design and/or construction control. The contractor is usually required to provide a pile and driving equipment data form similar to Figure 17.3 and obtain prior approval from the State transportation agency. Even if wave equation analysis is not required, this form should be included in the project files so a wave equation analysis could be performed in the future. This form can also function as a check list for the inspector to compare the proposed equipment with the actual equipment on-site.

24.6 INSPECTION OF DRIVING EQUIPMENT DURING INSTALLATION

The main purpose of inspection is to assure that piles are installed so that they meet the driving criteria and the pile remains undamaged. The driving criteria is often defined as a minimum driving resistance as measured by the blow count in blows per 0.25 meter. The driving criteria is to assure that piles have the desired capacity. However, the driving resistance is also dependent upon the performance of the pile driving hammer. The driving resistance will generally be lower when the hammer imparts higher energy and force to the pile, and the driving resistance will be higher if the hammer imparts lower energy and force to the pile. High driving resistances can be due either to soil resistance or to a poorly performing hammer. Thus, for the inspector to assure that the minimum driving criteria has been met and therefore the capacity is adequate, the inspector must evaluate if the hammer is performing properly.

Each hammer has its own operating characteristics; the inspector should not blindly assume that the hammer on the project is in good working condition. In fact, two different types of hammers with identical energy rating will not drive the same pile in the same soil with the same driving resistance. In fact, two supposedly identical hammers (same make and model) may not have similar driving capability due to several factors including differing friction losses, valve timing, air supply hose type-length-condition, fuel type and intake amount, and other maintenance status items. The inspector should become familiar with the proper operation of the hammer(s) used on site. The inspector may wish to contact the hammer manufacturer or supplier who generally will welcome the opportunity to supply further information. The inspector should review the operating characteristics for the hammer which are included in Chapter 22. The following checklists briefly summarize key hammer inspection issues.

24.6.1 Drop Hammers

a. Determine/confirm the ram weight. Ram weight can be calculated from the ram volume and steel density of 78.5 kN/m^3 if necessary.

b. The leads should have sufficient tolerance and/or the guides greased to allow the ram to fall without obstruction or binding.

c. Make sure the desired stroke is maintained. Low strokes will reduce energy. Excessively high strokes increase pile stresses and could cause pile damage.

d. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

e. Make sure the hammer hoist line is spooling out freely during the drop and at impact. If the hoist line drags, less energy will be delivered. If the crane operator catches the ram too early, not only is less energy delivered, but energy is transmitted into the hoist line, crane boom, and hoist, which could cause maintenance and/or safety problems.

24.6.2 Single Acting Air/Steam Hammers

a. Determine/confirm the ram weight. Ram weight can be calculated from the ram volume and steel density of 78.5 kN/m^3 if necessary. Check for and record any identifying labels as to hammer make, model and serial number.

b. Check the air or steam supply and confirm it is of adequate capacity to provide the required pressure and flow volume. Also check the number, length, diameter, and condition of the air/steam hoses. Manufacturers provide guidelines for proper compressors and supply hoses. Air should be blown through the hose before attaching it to the hammer. The motive fluid lubricator should occasionally be filled with the appropriate lubricant as specified by the manufacturer. During operation, check that the pressure at the compressor or boiler is equal to the rated pressure plus hose losses. The pressure should not vary significantly during driving. The photograph of an air compressor display panel in Figure 24.5 illustrates the discharge pressure dial that should be checked.

c. Visually inspect the slide bar and its cams for excessive wear. Some hammers can be equipped with a slide bar with dual set of cams to offer two different strokes. The stroke can be changed with a valve, usually operated from the ground. Measure the stroke being attained and confirm it meets specification.

d. Check that the columns or ram guides, piston rod, and slide bar are well greased.

e. For most air/steam hammers, the total thickness of hammer cushion and striker plate must match the hammer manufacturer's recommendation and the hammer cushion cavity in the helmet for proper valve timing and hammer operation. This thickness must be maintained and should be checked before placing the helmet into the leads, and thereafter by comparison of cam to valve position and/or gap between ram and hammer base when the ram is at rest on the pile top.

f. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

g. The ram and column keys used to fasten together hammer components should all be tight.

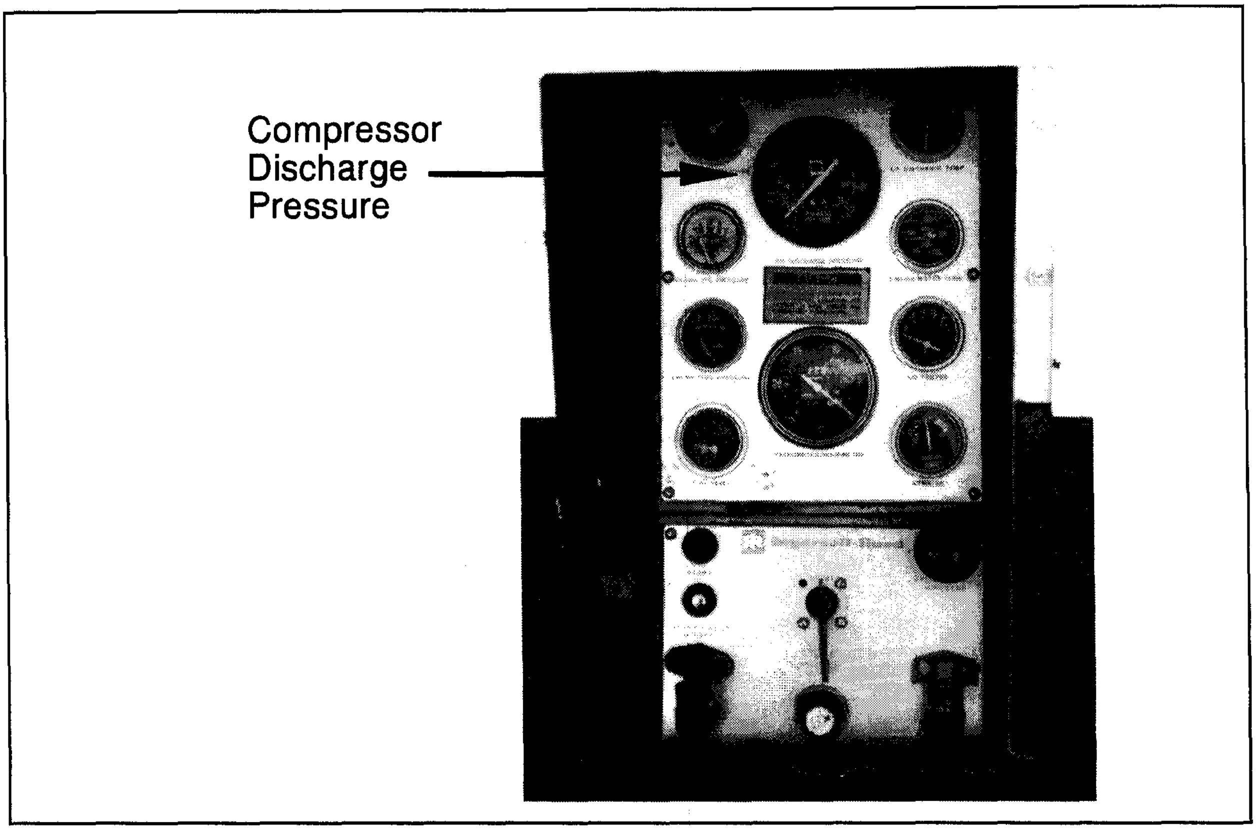


Figure 24.5 Air Compressor Display Panel

h. The hammer hoist line should always be slack, with the hammer's weight fully carried by the pile. Excessive tension in the hammer hoist line is a safety hazard and will reduce energy to the pile. Leads should always be used.

i. Compare the observed hammer speed in blows per minute near end of driving with the manufacturer's specifications. Blows per minute can be timed with a stopwatch or a saximeter. Slower operating rates may imply a short stroke (from inadequate pressure or volume, restricted or undersized hose, or inadequate lubrication) or improper valve timing (possibly from incorrect cushion thickness or worn parts). Erratic hammer operation, such as skipping blows, can result from improper cushion thickness, poor lubrication, foreign material in a valve, faulty valve/cam system, or loose hammer fasteners or keys.

j. As the driving resistance increases, the ram stroke may also increase, causing it to strike the upper hammer assembly and lifting the hammer ("racking") from the pile. If this behavior is detected, the air pressure flow should be reduced gradually until racking stops. The flow should not be overly restricted so that the stroke is reduced.

k. Some manufacturers void their warranty if the hammer is consistently operated above 100 blows per 250 mm of penetration beyond short periods such as required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

l. Common problems and problem indicators for air/steam hammers are summarized in Table 24-1.

TABLE 24-1 COMMON PROBLEMS AND PROBLEM INDICATORS FOR AIR/STEAM HAMMERS (from Williams Earth Sciences, 1995)

Common Problems	Indicators
Air trip mechanism on hammer malfunctioning.	Erratic operation rates or air valve sticking open or close.
Cushion stack height not correct (affects timing of trip mechanism air valve).	Erratic operation rates.
Compressor not supplying correct pressure and volume of air to hammer.	Blows per minute rate is varying either faster or slower than the manufacturer specified.
Air supply line kinked or tangled in leads, boom or other.	Visually evident.
Moisture in air ices up hammer.	Ice crystals exiting exhaust ports of hammer.
Lack of lubricant in air supply lines.	Erratic operation rates.
Packing around air chest worn, allowing air blow by.	Ram raises slowly - blows per minute rate slower than manufacturer specifications - air leaking around piston shaft and air chest.
Nylon slide bar worn.	Visually evident.
Ram columns not sufficiently greased.	Visually evident.

An inspection form for single and differential acting air/steam hammers is provided in Figure 24.6. The primary feature of this form is the three column area in the middle of the form. The left column illustrates the key objects of the driving system. The middle column contains the manufacturer's requirements for key objects and the right column is used to record the observed condition of those objects. This format allows the inspector to quickly identify potential problems and an immediate correction may be possible. The hammer inspection form is intended to be used periodically during the course of the project as a complement to the pile driving log.

The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

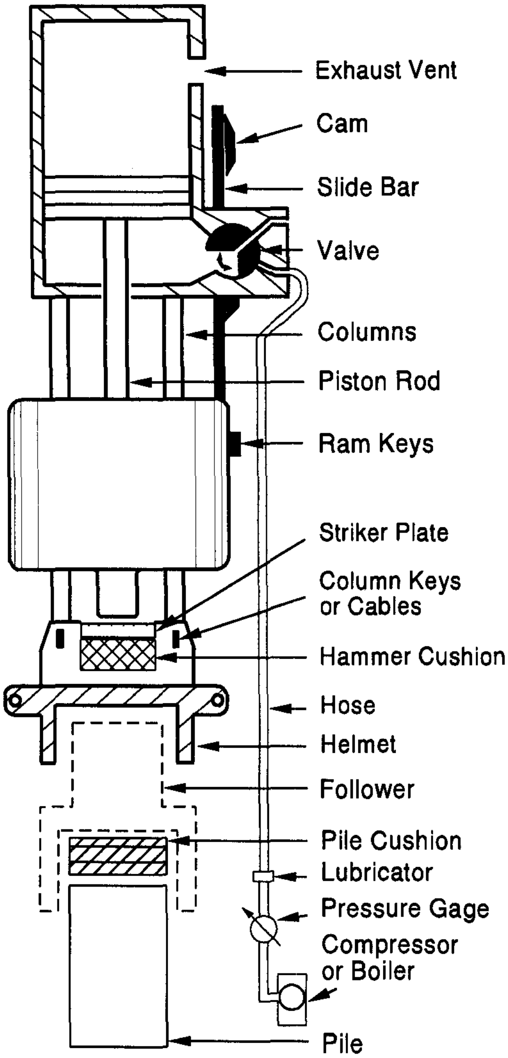
24.6.3 Double Acting or Differential Air/Steam Hammers

a. Determine/confirm the ram weight. Ram weight can be calculated from the ram volume and steel density of 78.5 kN/m^3 if necessary. Check for and record any identifying labels as to hammer make, model and serial number.

b. Check the air or steam supply and confirm it is of adequate capacity to provide the required pressure and flow volume This is extremely important since approximately half the rated energy comes from the pressure on the ram during the downstroke. Check also the number, length, diameter, and condition of the air/steam hoses. Manufacturers provide guidelines for proper compressors and supply hoses. Air should be blown through the hose before attaching it to the hammer. The motive fluid lubricator should occasionally be filled with the appropriate lubricant as specified by the manufacturer. During operation, check that the pressure at the compressor or boiler is equal to the rated pressure plus hose losses. The pressure should not vary significantly during driving. Record the pressure at the beginning of driving.

Project/Pile:____________________
Date:____________________
Conditions:____________________

Hammer Name:____________
Serial No:____________

OBJECT	REQUIREMENTS	OBSERVATIONS
	Slide Bars / Cams Greased? Tight?	Yes / No Remarks__________
	Columns Greased?	Yes / No
	Ram Keys Tight?	Yes / No
	Column Keys or Cables Tight?	Yes / No
	Striker Plate	t =________ D =________
	Hammer Cushion	t =________ D =________ Material__________ How long in use?__________
	Helmet	Type or Weight?__________
	Follower	Yes / No; Type__________
	Pile Cushion	Material__________ t = ________ Size__________ How long in use?__________
	Pile	Material__________ Length________ Size________ Batter__________
	Hose	I.D. Size________ Length________ Leaks?______ Obstructions?______
	Lubricator Filled?	Yes / No
	Pressure at Hammer __________ kPa	Measured__________ kPa at __________ meters from Hammer
	Fluctuating during Driving?	Yes / No; How much?________ kPa
	Check Compressor and Boiler?	Size__________ m^3/min Make__________

MANUFACTURER'S HAMMER DATA

Ram Weight__________

Max. Stroke__________

Rated Energy__________

Blows/min in Hard Driving__________

ATTACHED SAXIMETER PRINTOUT

OBSERVATION WHEN BEARING IS CONFIRMED

Full Ram Stroke Yes/No, ________ %
Blows/min; Blows/m __________
High Pile Rebound; Pile Whipping Yes/No; Yes/No
Pile-Hammer Alignment Front/Back____ Sides____
Crane Size and Make __________
Lead Type __________
Hammer Lead Guides Lubricated Yes/No
Piston Rod Lubricated __________
Exhaust Desciption: Freezing? Condensing? Lubricant Apparent?

Figure 24.6 Inspection Form for Single and Differential Acting Air/Steam Hammers

c. Visually inspect the slide bar and its cams for excessive wear. Measure the stroke being attained and confirm that it meets specification.

d. Check that the columns or ram guides, piston rod, and slide bar are well greased.

e. For most air/steam hammers, the total thickness of hammer cushion and striker plate must match the hammer manufacturer's recommendation and the hammer cushion cavity in the helmet for proper valve timing and hammer operation. This thickness must be maintained, and can be checked before assembly of the helmet into the leads, and thereafter by comparison of cam to valve position and/or gap between ram and hammer base when the ram is at rest on the pile.

f. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

g. The ram and column keys used to fasten together hammer components should all be tight.

h. The hammer hoist line should always be slack with the hammer's weight and be fully carried by the pile. Excessive tension in the hammer hoist line is a safety hazard and will reduce energy to the pile. Leads should always be used.

i. Compare the observed hammer speed in blows per minute near end of driving with the manufacturer's specifications. Blows per minute can be timed with a stopwatch or a saximeter. Slower operating rates may imply a short stroke (from inadequate pressure or volume, restricted or undersized hose, or inadequate lubrication) or improper valve timing (possibly from incorrect cushion thickness or worn parts). Erratic hammer operation, such as skipping blows, can result from improper cushion thickness, poor lubrication, foreign material in a valve, faulty valve/cam system, or loose hammer fasteners or keys.

j. As the driving resistance increases, the ram stroke may also increase, causing it to strike the upper hammer assembly and lifting the hammer (racking) from the pile. If this behavior is detected, the pressure flow should be reduced gradually until racking stops. This will result in a reduction in energy since the pressure also acts during the downstroke, thereby contributing to the rated energy.

Record the final pressure. The flow should not be overly restricted so that the stroke is also reduced, causing a further reduction in energy. For optimum performance, the pressure flow should be kept as full as possible so that the hammer lift-off is imminent.

k. Some manufacturers void their warranty if the hammer is consistently operated above 100 blows per 250 mm of penetration beyond short periods such as required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

l. Record the final pressure and compare with manufacturer's energy rating at this pressure.

m. Common problems and problem indicators for air/steam hammers are summarized in Table 24-1.

An inspection form for enclosed double acting air/steam hammers is provided in Figure 24.7. The primary feature of this form is the three column area in the middle of the form. The left column identifies key objects of the driving system. The middle column contains the manufacturer's requirements for key objects and the right column is used to record the observed condition of those objects. This format allows the inspector to quickly identify potential problems and an immediate correction may be possible. The hammer inspection form is intended to be used periodically during the course of a project as a complement to the pile driving log.

The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

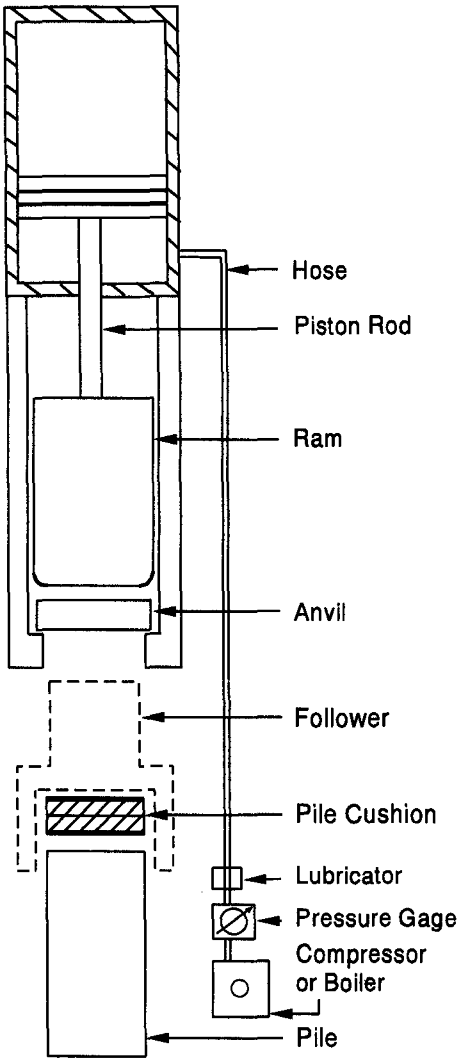
24.6.4 Single Acting Diesel Hammers

a. Determine/confirm that the hammer is the correct make and model. Check for and record any identifying labels as to hammer make, model and serial number.

Project/Pile: ____________________
Date: ____________________
Conditions: ____________________

Hammer Name: ____________
Serial No: ____________

OBJECT



REQUIREMENTS	OBSERVATIONS
Follower	Yes / No; Type ____________
Pile Cushion	Material ____________ t = ________ Size ____________ How long in use? ____________
Pile	Material ____________ Length ________ Size ____________ Batter ____________
Hose Size?	I.D. Size ________ Length ________ Leaks? ______ Obstructions? ______
Lubricator Filled?	Yes / No
Pressure at Hammer __________ kPa	Measured ____________ kPa at __________ meters from Hammer
Fluctuating during Driving?	Yes / No; How much? ____________
Check Compressor and Boiler?	Size ____________ m^3 / min Make ____________

MANUFACTURER'S HAMMER DATA

Ram Weight ____________

Max. Stroke ____________

Rated Energy ____________

Blows/min in Hard Driving ____________

ATTACHED SAXIMETER PRINTOUT

OBSERVATION WHEN BEARING IS CONFIRMED

Full Ram Stroke	Yes/No, __________ %
Blows/min; Blows/m	____________
High Pile Rebound; Pile Whipping	Yes/No; Yes/No
Pile-Hammer Alignment	Front/Back____ Sides____
Crane Size and Make	____________
Lead Type	____________
Hammer Lead Guides Lubricated	Yes/No
Piston Rod Lubricated	____________
Exhaust Desciption:	Freezing? Condensing? Lubricant Apparent?

Figure 24.7 Inspection Form for Enclosed Double Acting Air/Steam Hammers

b. Make sure <u>all</u> exhaust ports are open with all plugs removed.

c. Inspect the recoil dampener for condition and thickness. If excessively worn or improper thickness (consult manufacturer) it should be replaced. If the recoil dampener is too thin, the stroke will be reduced. If it is too thick, or if cylinder does not rest on dampener between blows, the ram could blow out the hammer top and become a safety hazard.

d. Check that lubrication of all grease nipples is regularly made. Most manufacturers recommend the impact block be greased every half hour of operation.

e. As the ram is visible between blows, check the ram for signs of uniform lubrication and ram rotation. Poor lubrication will increase friction and reduce energy to the pile.

f. Determine the hammer stroke, especially at end of driving or beginning of restrike. A "jump stick" attached to the cylinder is a safety hazard and should not be used. The stroke can be determined by a saximeter which measures the time between blows and then calculates the stroke. The hammer stroke can also be calculated from this formula if the number of blows per minute (bpm) is manually recorded.

$$h\ [\text{meters}] = \left[4400/[\text{bpm}^2]\right] - 0.09$$

The calculated stroke may require correction for batter or inclined piles. The inspector should always observe the ram rings and visually estimate the stroke using the manufacturer's chart.

g. As the driving resistance increases, the stroke should also increase. At the end of driving, if the ram fails to achieve the correct stroke (part of the driving criteria from a wave equation analysis), the cause could be lack of fuel. Most hammers have adjustable fuel pumps. Some have distinct fuel settings as shown in Figure 24.8, others are continuously variable as shown in Figure 24.9, and some use a pressure pump as shown in Figure 24.10. Make sure the pump is on the correct fuel setting or pressure necessary to develop the required stroke. The fuel and fuel line should be free of dirt or other contaminants. A clogged or defective fuel injector will also reduce the stroke and should be replaced if needed.

Figure 24.8 Fixed Four Step Fuel Pump on Delmag Hammer

Figure 24.9 Variable Fuel Pump on FEC Hammer

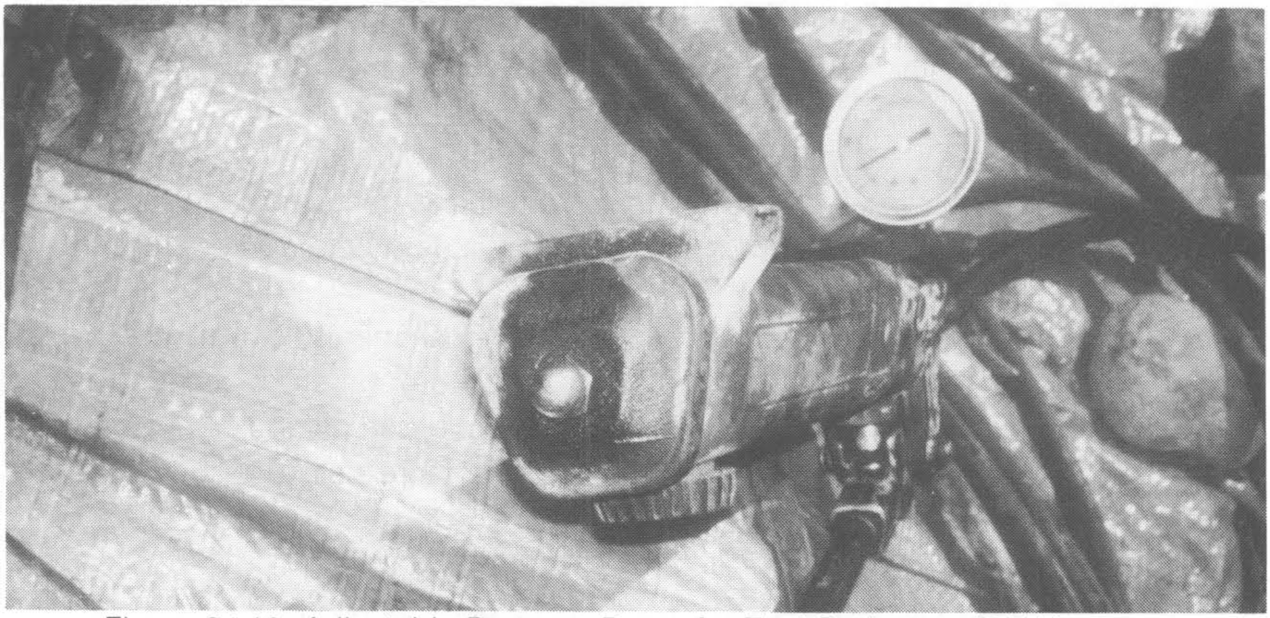


Figure 24.10 Adjustable Pressure Pump for Fuel Setting on ICE Hammer

h. Low strokes could be due to poor compression caused by worn or defective piston or anvil rings. Check compression by raising the ram, and with the fuel turned off, allowing the ram to fall. The ram should bounce several times if the piston and anvil rings are satisfactory.

i. Watch for signs of preignition. When a hammer preignites, the fuel burns before impact, requiring extra energy to compress gas and leaving less energy to transfer to the pile. In long sustained periods of driving, or if the wrong fuel with a low flash point is used, the hammer could overheat and preignite. When preignition occurs, less energy is transferred and the driving resistance rises, giving a false indication of high pile capacity. If piles driven with a cold hammer drive deeper or with less hammer blows, or if the driving resistances decrease after short breaks, preignition could be the cause and should be investigated. Dynamic testing is the preferable method to check for preignition.

j. For some diesel hammers, the total thickness of hammer cushion and striker plate must match the hammer manufacturer's recommendation and the hammer cushion cavity in the helmet for proper fuel injection and hammer operation. This total thickness must be maintained.

k. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

l. The hammer hoist line should always be slack, with the hammer's weight fully carried by the pile. Excessive tension in the hammer hoist line is a safety hazard and will reduce energy to the pile. Leads should always be used.

m. Some manufacturers void their warranty if the hammer is consistently operated above 100 blows per 250 mm of penetration beyond short periods, such as those required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

n. Common problems and problem indicators for single acting diesel hammers are presented in Table 24.2.

TABLE 24-2 COMMON PROBLEMS AND PROBLEM INDICATORS FOR SINGLE ACTING DIESEL HAMMERS (from Williams Earth Sciences, 1995)	
Common Problems	Indicators
Water in fuel.	Hollow sound, white smoke.
Fuel lines clogged.	No smoke or little gray smoke.
Fuel pump malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Fuel injectors malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Oil low.	Blows per minute rate is lower than specified.
Oil pump malfunctioning.	Blows per minute rate is lower than specified.
Water in combustion chamber.	Hollow sound, white smoke.
Piston rings worn.	Low strokes.
Tripping device broken.	Pawl or pin used to lift piston does not engage piston. Pawl engages but does not lift piston.
Over heating.	Paint and oil on cooling fins start to burn/ sound changes.

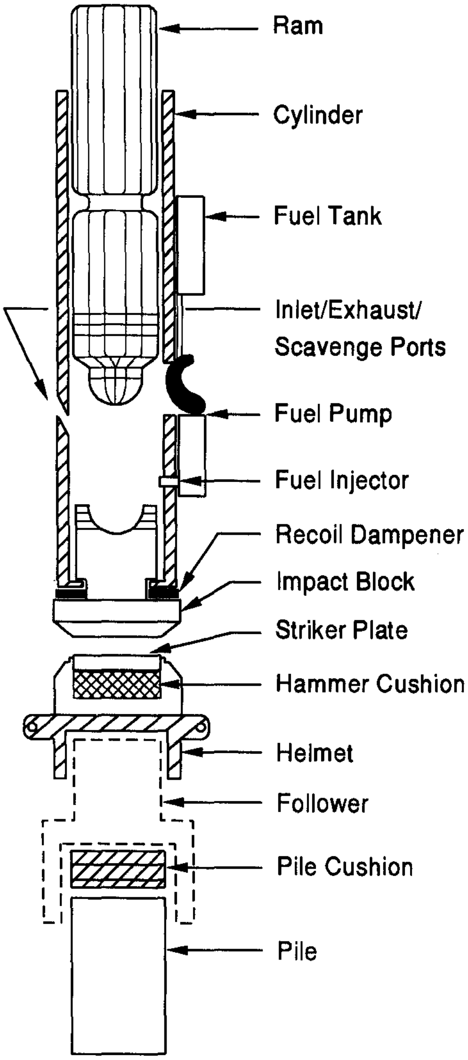
An inspection form for single acting diesel hammers is provided in Figure 24.11. The primary feature of this form is the three column area in the middle of the form. The left column identifies key objects of the driving system, the middle column contains the manufacturer's requirements for that object and the right column is used to record the observed condition of that object. This format allows the inspector to quickly identify potential problems and an immediate correction may be possible. The hammer inspection form is intended to be used periodically during the course of a project as a complement to the pile driving log.

The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

Project/Pile:______________________________
Date:______________________________
Conditions:______________________________

Hammer Name:______________
Serial No:______________

OBJECT



REQUIREMENTS	OBSERVATIONS
Ram Lubricated?	Yes / No
Fuel Tank Filled with Type II Diesel?	Yes / No Type______________
Exhaust Ports Open?	Yes / No
Fuel Pump	Hammer Setting______________
Recoil Dampener Undamaged?	Yes / No
Impact Block Lubricated?	Yes / No
Striker Plate	t =__________ D =__________
Hammer Cushion	t =__________ D =__________ Material______________ How long in use?______________
Helmet	Type or Weight?______________
Follower	Yes / No; Type______________
Pile Cushion	Material______________ t = __________ Size__________ How long in use?______________
Pile	Material______________ Length__________ Size__________ Batter______________

MANUFACTURER'S HAMMER DATA

Ram Weight______________________

Hammer Setting	Rated Energy kJ	Rated Stroke m
min.		
max.		

ATTACHED SAXIMETER PRINTOUT

OBSERVATION WHEN BEARING IS CONFIRMED

Excessive Cylinder Rebound	Yes/No
High Pile Rebound	Yes/No
Pile Whipping	Yes/No
Pile-Hammer Alignment	Front/Back____ Sides____
Crane Size and Make	______________
Lead Type	______________
Hammer Lead Guides Lubricated	Yes/No
Color of Smoke	______________
Steel to Steel Impact Sound	______________

Figure 24.11 Inspection Form for Single Acting Diesel Hammers

24.6.5 Double Acting Diesel Hammers

a. Determine/confirm that the hammer is the correct make and model. Check for and record any identifying labels as to hammer make, model and serial number.

b. Make sure <u>all</u> exhaust ports are open with all plugs removed.

c. Inspect the recoil dampener for condition and thickness. If excessively worn or of improper thickness (consult manufacturer), it should be replaced. If it is too thin, the stroke will be reduced. If it is too thick or if cylinder does not rest on dampener between blows, the ram will cause hammer lift-off.

d. Check that lubrication of all grease nipples is regularly made. Most manufacturers recommend the impact block be greased every half hour of operation.

e. After the hammer is stopped, check the ram for signs of lubrication by looking into the exhaust port or trip slot. Poor lubrication increases friction, thus reducing energy to the pile.

f. Always measure the bounce chamber pressure, especially at end of driving or restrike. This indirectly measures the equivalent stroke or energy. All double acting diesels have a gauge. On most hammers an external gauge is connected by a hose to the bounce chamber. A photograph of a typical external bounce chamber pressure gauge is presented in Figure 24.12. The manufacturer should supply a chart relating the bounce chamber pressure for a specific hose size/length to the rated energy. The inspector should compare measured bounce chamber pressure with the manufacturer's chart to estimate the energy. The bounce chamber pressure measured may require correction for batter or inclined piles.

g. As the driving resistance increases, the stroke and bounce chamber pressure should also increase. At the end of driving, if the ram fails to achieve the correct stroke or bounce chamber pressure (part of the driving criteria from a wave equation analysis), the cause could be lack of fuel. All these hammers have continuously variable fuel pumps. Check that the fuel pump is on the correct fuel setting. The fuel should be free of dirt or other contaminants. A clogged or defective fuel injector reduces the stroke.

Figure 24.12 Typical External Bounce Chamber Pressure Gauge

h. In hard driving, high strokes cause high bounce chamber pressures. If the cylinder weight cannot balance the bounce chamber pressure, the hammer will lift-off of the pile, and the operator must reduce the fuel to prevent this unstable racking behavior. Ideally it is set and maintained so that lift-off is imminent. The bounce chamber pressure gauge reading should correspond to the hammer's maximum bounce chamber pressure for the hose length used when lift-off is imminent. If not, then the bounce chamber pressure gauge is out of calibration and should be replaced, or the bounce chamber pressure tank needs to be drained.

i. Low strokes indicated by a low bounce chamber pressure could be due to poor compression caused by worn or defective piston or anvil rings. Check compression with the fuel turned off by allowing the ram to fall. The ram should bounce several times if the piston and anvil rings are satisfactory.

j. Watch for preignition. When a hammer preignites, the fuel burns before impact requiring extra energy to compress the gas and reducing energy transferred to the pile. When preignition occurs, the pile driving resistance increases giving a false indication of high pile capacity. In long sustained periods of driving or if low flash point fuel is used, the hammer could overheat and preignite. If piles driven with a cold hammer drive deeper or with fewer hammer blows, or if the driving resistance decreases after short breaks, investigate for preignition, preferably with dynamic testing.

k. For some diesel hammers, the total thickness of the hammer cushion and striker plate must match the manufacturer's recommendation for proper fuel injection timing and hammer operation. This total thickness must be maintained.

l. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

m. The hammer hoist line should always be slack, with the hammer's weight fully carried by the pile. Excessive tension in the hammer hoist line is a safety hazard and will reduce energy to the pile. Leads should always be used.

n. Some manufacturers void their warranty if the hammer is consistently operated above 100 blows per 250 mm of penetration beyond short periods such as those required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

o. Common problems and problem indicators for double acting diesel hammers are presented in Table 24.3.

An inspection form for double acting diesel hammers is provided in Figure 24.13. The primary feature of this form is the three column area in the middle of the form. The left column identifies key objects of the driving system, the middle column contains the manufacturer's requirements for that object and the right column is used to record the observed condition of that object. This format allows the inspector to quickly identify potential problems and an immediate correction may be possible. The hammer inspection form is intended to be used periodically during the course of a project as a complement to the pile driving log.

TABLE 24-3 COMMON PROBLEMS AND PROBLEM INDICATORS FOR DOUBLE ACTING DIESEL HAMMERS (from Williams Earth Sciences, 1995)	
Common Problems	Indicators
Water in fuel.	Hollow sound, white smoke.
Fuel lines clogged.	No smoke or little gray smoke.
Fuel pump malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Fuel injectors malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Oil low.	Blows per minute rate is lower than specified.
Oil pump malfunctioning.	Blows per minute rate is lower than specified.
Build-up of oil in bounce chamber.	Not visible from exterior.
Water in combustion chamber.	Hollow sound, white smoke.
Piston rings worn.	Low strokes.
Tripping device broken.	Pawl or pin used to lift piston does not engage piston. Pawl engages but does not lift piston.
Over heating.	Paint and oil on cooling fins start to burn/ sound changes.

The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

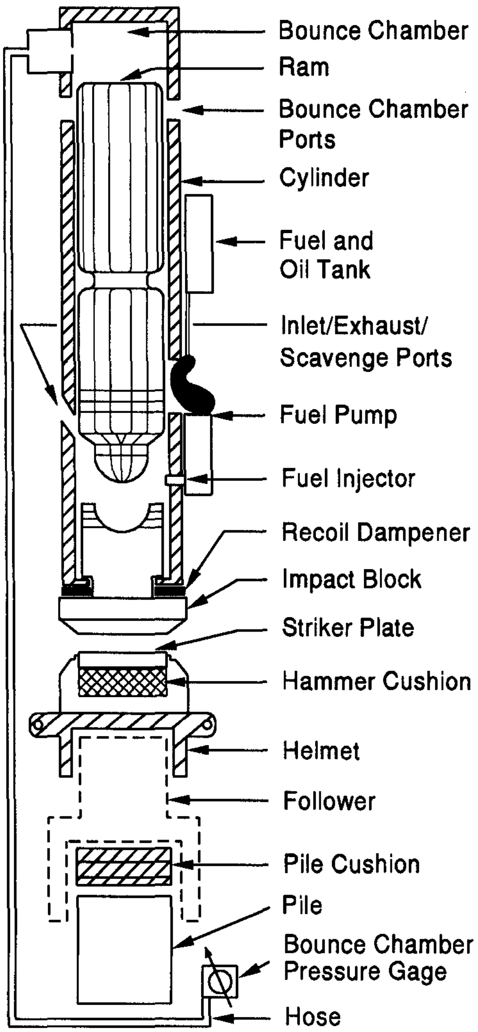
24.6.6 Hydraulic Hammers

a. Determine/confirm the ram weight. If necessary, the ram weight can be calculated from the ram volume and steel density of 78.5 kN/m^3, although some rams may be hollow or filled with lead. There may also be identifying labels as to hammer make, model, and serial number which should be recorded.

Project/Pile:____________________________
Date:____________________________
Conditions:____________________________

Hammer Name:________________
Serial No:________________

OBJECT



REQUIREMENTS	OBSERVATIONS
Ram Lubricated?	Yes / No
Fuel Tank Filled with Type II Diesel?	Yes / No Type__________
Exhaust Ports Open?	Yes / No
Fuel Pump	Hammer Setting__________
Recoil Dampener Undamaged?	Yes / No
Impact Block Lubricated?	Yes / No
Striker Plate	t =______ D =______
Hammer Cushion	t =______ D =______ Material__________ How long in use?__________
Helmet	Type or Weight?__________
Follower	Yes / No; Type__________
Pile Cushion	Material__________ t = ______ Size______ How long in use?__________
Pile	Material__________ Length______ Size______ Batter__________
Bounce Chamber Hose	Length__________

MANUFACTURER'S HAMMER DATA

Ram Weight____________________

Max. Stroke____________________

Bounce Chamber Pressure (kPa)	Rated Energy (kJ)

ATTACHED SAXIMETER PRINTOUT

OBSERVATION WHEN BEARING IS CONFIRMED

Bounce Chamber Pressure	__________
Cylinder Lift-off	Time or Depth______
Excessive Cylinder Rebound	Yes/No
High Pile Rebound	Yes/No
Pile Whipping	Yes/No
Pile-Hammer Alignment	Front/Back____ Sides____
Crane Size and Make	__________
Lead Type	__________
Hammer Lead Guides Lubricated	Yes/No
Color of Smoke	__________
Steel to Steel Impact Sound	__________

Figure 24.13 Inspection Form for Double Acting Diesel Hammers

b. Check the power supply and confirm it has adequate capacity to provide the required pressure and flow volume. Also, check the number, length, diameter, and condition of the hoses (no leaks in hoses or connections). Manufacturers provide guidelines for power supplies and supply hoses. Hoses bent to a radius less than recommended could adversely affect hammer operation or cause hose failure.

c. Hydraulic hammers must be kept clean and free from dirt and water. Check the hydraulic filter for blocked elements. Most units have a built in warning or diagnostic system.

d. Check that the hydraulic power supply is operating at the correct speed and pressure. Check and record the pre-charge pressures or accumulators for double acting hammers. Allow the hammer to warm up before operation, and do not turn off power pack immediately after driving.

e. Most hydraulic hammers have built in sensors to determine the ram velocity just prior to impact. This result may be converted to kinetic energy or equivalent stroke. The inspector should verify that the correct ram weight is entered in the hammer's "computer". **This monitored velocity, stroke, or energy result should be constantly monitored and recorded. Some hammers have, or can be equipped with, a printout device to record that particular hammer's performance information with pile penetration depth and/or pile driving resistance.** This is the most important hammer check that the inspector can and should make for these hammers. A photograph of a hydraulic hammer readout panel is presented in Figure 24.14.

f. For hydraulic hammers with observable rams, measure the stroke being attained and confirm that it meets specification. For hammers with enclosed rams, it is impossible to observe the ram and estimate the stroke.

g. Check that the ram guides and piston rod are well greased.

h. Where applicable, the total thickness of hammer cushion and striker plate must be maintained to match the manufacturer's recommendation for proper valve timing and hammer operation.

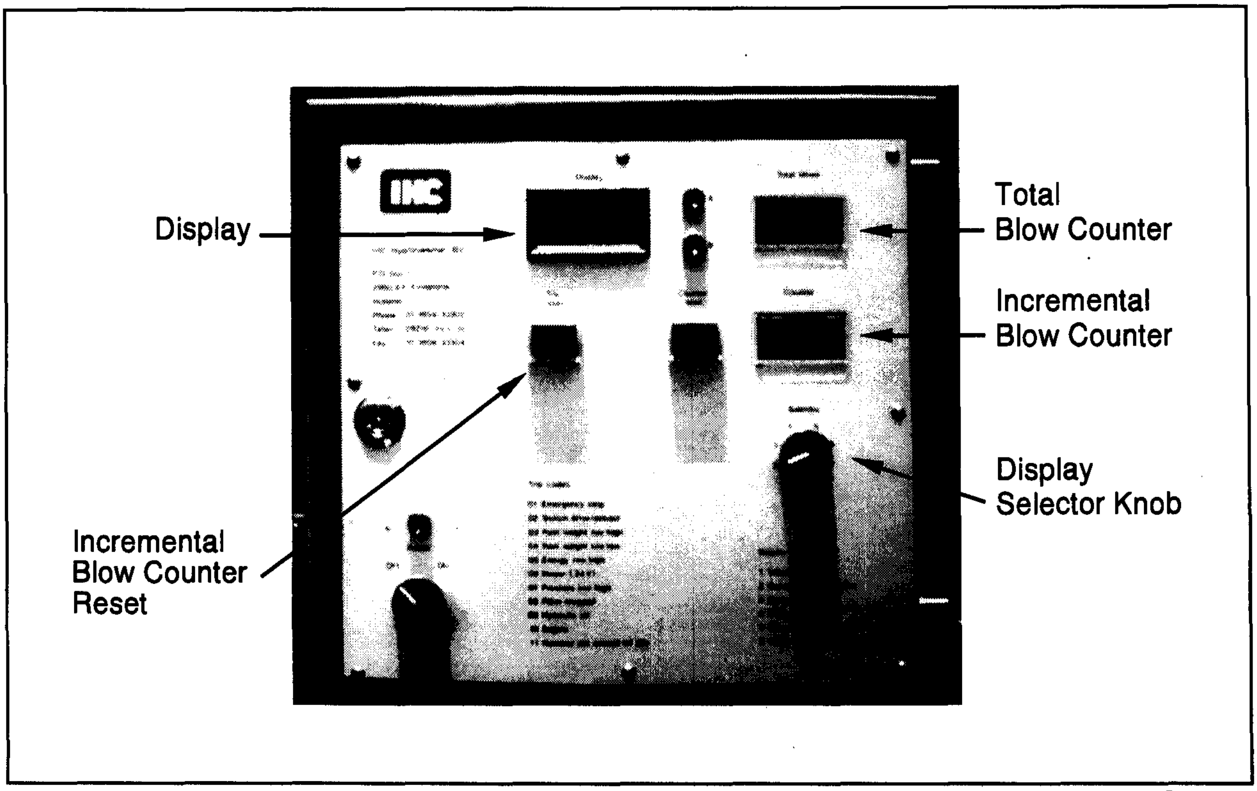


Figure 24.14 IHC Hydraulic Hammer Readout Panel (courtesy of L.B. Foster Co.)

i. Make sure the helmet stays properly seated on the pile and that the hammer and pile maintain alignment during operation.

j. The hammer hoist line should always be slack, with the hammer's weight fully carried by the pile. Excessive tension in the hammer hoist line is a safety hazard and will reduce energy to the pile. Leads should always be used.

k. Compare the observed hammer speed in blows per minute from near end of driving with the manufacturer's specifications. Blows per minute can be timed with a stopwatch or a saximeter. Slower operating rates at full stroke may imply excessive friction, or incorrect hydraulic power supply.

l. As the driving resistance increases, the ram stroke may also increase, causing the ram to strike the upper hammer assembly and lifting the hammer from the pile (racking). If this behavior is detected, the pressure flow should be reduced gradually until racking stops. Many of these hammers have sensors, and if they detect this condition, the hammer will automatically shut down. The flow should not be overly restricted so that the correct stroke is maintained.

m. Some manufacturers void their warranty if the hammer is consistently operated above 100 blows per 250 mm of penetration beyond short periods such as those required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

n. Common problems and problem indicators for hydraulic hammers are summarized in Table 24-4.

TABLE 24-4 COMMON PROBLEMS AND PROBLEM INDICATORS FOR HYDRAULIC HAMMERS (from Williams Earth Sciences, 1995)

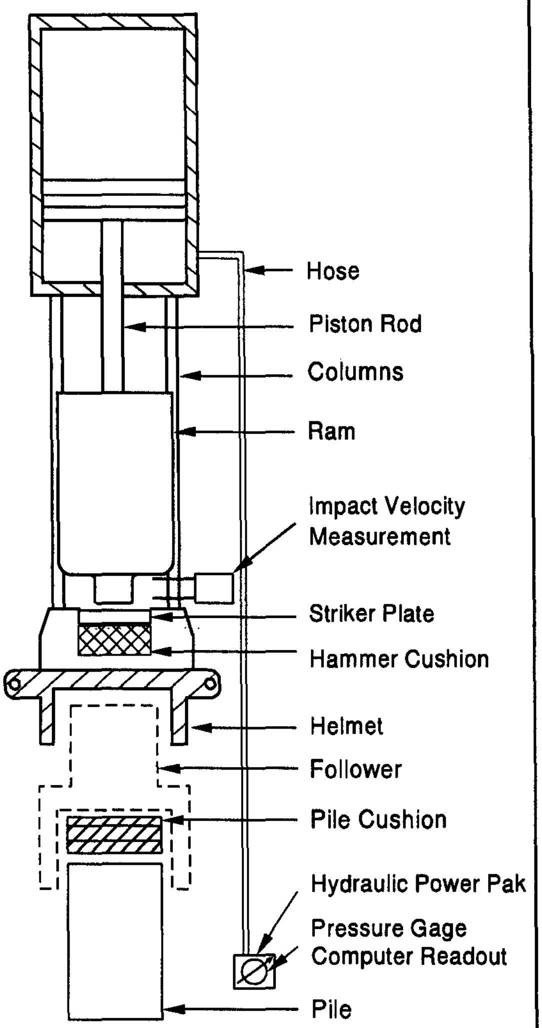
Common Problems	Indicators
Hoses getting caught in leads.	Visually evident.
Fittings leaking.	Hydraulic fluid dripping.
Electrical connections.	Erratic performance.
Sensors.	Erratic performance.

An inspection form for hydraulic hammers is provided in Figure 24.15. The primary feature of this form is the three column area in the middle of the form. The left column identifies key objects of the driving system, the middle column contains the manufacturer's requirements for that object, and the right column is used to record the observed condition of that object. The hammer inspection form is intended to be used periodically during the course of a project as a complement to the pile driving log.

The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

Project/Pile:______________ Hammer Name:______________
Date:______________ Serial No:______________
Conditions:______________

OBJECT



REQUIREMENTS	OBSERVATIONS
Ram Visible?	Yes / No Observed Ram Stroke__________ m
Ram Downward Pressure Provided ?	Yes / No Hydraulic Pressure, Rated______ kPa Hydraulic Pressure, Actual______ kPa
Impact Velocity Measurement ?	Yes / No
If Without Velocity Measurement Then ?	Free Fall?__________ Observed Fall Height__________ m Pressure under ram during fall______ Preadmission Possible?__________
Striker Plate	t =__________ D =__________
Hammer Cushion	t =__________ D =__________ Material__________ How long in use?__________
Helmet	Type or Weight?__________
Follower	Yes / No; Type__________
Pile Cushion	Material__________ t = ________ Size__________ How long in use?__________
Hydraulic Power Pack	Make__________ Model__________
Pressure Gage ?	Yes / No Reading__________
Computer Readout ?	Yes / No Reading__________
Pile	Material__________ Length________ Size__________ Batter__________

MANUFACTURER'S HAMMER DATA

Ram Weight______________
Max. Stroke______________
Min. Stroke______________
Max. Energy______________
Min. Energy______________

ATTACH SAXIMETER PRINTOUT

OBSERVATION WHEN BEARING IS COMPLETED

Hammer Uplifting	Yes/No
Reduced Pressure	Yes/No
Blows/Minute	______________
Blow/meter	______________
High Pile Rebound	Yes/No
Pile Whipping	Yes/No
Pile-Hammer Alignment	Front/Back______Sides____
Crane Size and Make	______________
Lead Type	______________
Lead Guides Lubricated	Yes/No

Figure 24.15 Inspection Form for Hydraulic Hammers

24.6.7 Vibratory Hammers

a. Confirm that the hammer make and model meets specifications. There may also be identifying labels as to hammer make, model and serial number which should be recorded.

b. Check the power supply to confirm adequate capacity to provide the required pressure and flow volume. Check also the number, length, diameter, and condition of the hoses (no leaks in hoses or connections). Manufacturers provide guidelines for proper power supplies and supply hoses. Hoses bent to a smaller radius than recommended could affect hammer operation or cause hose failure.

c. Vibratory hammers must be kept clean, free from dirt and water. Check the hydraulic filter for blocked elements. Most units have a built in warning or diagnostic system.

d. Check and record that the hydraulic power supply is operating at the correct speed and pressure. Allow the hammer to warm up before operation, and do not turn off the power pack immediately after driving.

e. Record, if available, the vibrating frequency.

f. Make sure the hydraulic clamps for attachment to the pile are in good working order and effective.

g. The hammer hoist line should always be slack enough to allow penetration with the hammer's weight primarily carried by the pile. Excessive tension in the hammer hoist line will retard penetration. If used for extraction, the hoist line should be tight at all times. Leads are rarely used.

24.7 INSPECTION OF TEST OR INDICATOR PILES

Most specifications call for preconstruction verification of the foundation design through the testing of some selected piles. The size of the foundation and relative costs of testing often dictate the type and amount, if any, of confirmation testing. The inspector may be responsible for coordinating the test pile program with the contractor, other state personnel, and/or outside testing agencies.

Small foundations with few piles may be designed conservatively with high safety factors and oversized pile length and no further tests are required. All piles are then production piles and the entire pile foundation is usually installed in one or two days.

The piles, hammers, and other observations are recorded by the inspector and information appropriately passed on or filed. Inspection should be thorough as it is the only assurance of a good foundation. If any problems are observed, such as very low blow counts, refusal driving above scour depths, or excessive pile lengths, the problems and all pertinent observations must be reported quickly so that immediate corrective action can be taken.

On most projects, some additional verification is specified. Smaller projects may have only a single static test (Chapter 19) on one pile at a specific depth, or there may be a few dynamic test piles (Chapter 18). The dynamic tests may include either testing during driving to assess hammer performance and driving stresses, or testing during restrike to assess capacity, or both. The static or dynamic tests should be performed by state department of transportation personnel having appropriate knowledge of test procedures, or engineering consultants. Generally, tests are done on some of the first piles driven to verify or adjust the driving criteria which will then be used for subsequent production piles. This further verification provides rational basis for changes to the driving criteria, if necessary, which should be applied to subsequent production pile driving.

On larger projects, multiple test piles distributed across the site are often required to verify or adjust the driving criteria. The goal is to determine a driving criteria which will lead to a safe, but economical, foundation. Such tests could be primarily done at one time at the beginning of the construction. For example, so-called indicator piles are driven in selected locations across the project site to establish order lengths for concrete piles. Such selected piles are generally statically and/or dynamically tested. Alternatively, testing could be performed as the construction progresses with some test(s) establishing the driving criteria for piles in close proximity to the test pile(s), followed by production pile driving, and then repeating the process in stages across the site.

The test piles are often the most critical part of the foundation installation. The procedures and driving criteria established during this phase will be applied to all subsequent production piles. The largest savings are often found at this time. For example, test results may determine that the design pile length results in a greater pile capacity than required and that the piles could be made substantially shorter.

Alternatively, problems with the test piles are usually followed by the same problems with production piles. Since problems are in themselves costly, and if left unresolved may eventually escalate, determination of the best solution as quickly as possible should be accomplished. It is the inspector's responsibility to be observant and communicate significant observations precisely and in a timely manner to the state design and testing teams.

The answers to the following questions should be known before driving test piles. Usually the inspector has the responsibility and the decision making authority regarding these items, although advice from various agency personnel and/or outside consultants may be necessary or desirable.

1. Who determines test pile locations?

2. Who determines the test pile driving criteria?

3. Who stops the driving when the driving criteria is met?

4. Who decides at what depth to stop the indicator/test piles?

5. Who checks cutoff elevations?

6. Who checks for heave?

7. Who determines if static test and/or dynamic test results indicate an acceptable test pile?

8. Who determines if additional tests are required?

9. Who determines if modifications to procedures or equipment are required?

10. Who has authority to allow production pile installation? When is this approval to proceed to production granted?

11. Who produces what documentation?

24.8 INSPECTION OF PRODUCTION PILES

During the production pile driving operations, the inspector's function is to apply the knowledge gained from the test program to each and every production pile. Quality assurance measures for the pile quality and splices; hammer operation and cushion replacement; overall evaluation of pile integrity; procedures for completing the piles (*e.g.* filling pipe piles with concrete); and unusual or unexpected occurrences need to be addressed. Complete documentation for each and every pile must be obtained, and then passed on to the appropriate destination in a timely manner.

The following items should be checked frequently (*e.g.* for each production pile):

1. Does the pile meet specifications of type, size, length, and strength?

2. Is the pile installed in the correct location, within acceptable tolerances, and with the correct orientation?

3. Are splices, if applicable, made to specification?

4. Is pile toe protection required and properly attached?

5. Is the pile acceptably plumb?

6. Is the hammer working correctly?

7. Is the hammer cushion the correct type and thickness?

8. Is the pile cushion the correct type and thickness? Is it being replaced regularly?

9. Did the pile meet the driving criteria as expected?

10. Did the pile have unusual driving conditions and therefore potential problems?

11. Is there any indication of pile heave?

12. Is the pile cutoff at the correct elevation?

13. Is there any visual damage?

14. If appropriate, has the pipe pile been visually inspected prior to concrete filling? Has it been filled with the specified strength concrete? Were concrete samples taken?

15. Are piles which are to be filled with concrete, such as open ended pipes and prestressed concrete piles with center voids, being cleaned properly after driving is completed?

16. If there is any question about pile integrity, has the issue been resolved? Is the pile acceptable, or does it need remediation or replacement?

17. Is the documentation for this pile complete, including driving log? Has it been submitted on a timely basis to the appropriate authority?

Many of the above questions are self explanatory and need no further explanation. Every previous section of this chapter has material which will relate to inspection of production piles and offer the detailed answers to other questions raised above. Although the inspector has now had the experience of test pile installation, a few additional details and concerns are perhaps appropriate.

Counting the number of hammer blows per minute and comparing it to the manufacturer's specification will provide a good indication of whether or not the hammer is working properly. The stroke of the hammer for most single and double acting air/steam hammers can be observed. Check the stroke of a single acting diesel hammer with a saximeter or by computation from the blows per minute. Check the bounce chamber pressure for double acting diesel hammers. Most hydraulic hammers have built-in energy monitors, and this information should be recorded for each pile. The hammer inspection form presented earlier in this chapter should be completed for the hammer type being used.

A hammer cushion of manufactured material usually lasts for many hours of pile driving, (as much as 200 hours for some manufactured materials) so it is usually sufficient to check before the pile driving begins and periodically thereafter. Pile cushions (usually made of plywood) need frequent changing because of excessive compression or charring and have a typical life of about 1000 to 2000 hammer blows. Pile cushions should preferably be replaced as soon as they compress to one half of the original thickness, or if they begin to burn. No changes to the pile cushion thickness should be permitted near final driving. The required driving resistance for pile acceptance should only be allowed after the first 100 blows after cushion replacement.

Inspection of splices is important for pile integrity. Poorly made splices are a potential source of problems and possible pile damage during driving. In some cases damage may be detected from the blow count records. Dynamic pile testing can be useful in questionable cases.

Pile driving stresses should be kept within specified limits. If dynamic monitoring equipment was used during test pile driving, the developed driving criteria should keep driving stresses within specified limits. If periodic dynamic tests are made, a check that the driving stresses are within the specified limits can be provided. Adjustments of the ram stroke for all hammer types may be necessary to avoid pile damage. For concrete piles, cushion thicknesses or driving procedures may need adjustment to control tension and compression stresses. If dynamic testing is not used, a wave equation analysis is essential to evaluate the anticipated driving stresses.

Driving of piles at high driving resistances, above 120 blows per 250 mm, should be avoided by matching the driving system with the pile type, length and subsurface conditions. This should have been accomplished in the design phase by performing wave equation analysis. However, conditions can change across the project due to site variability.

All piles should be checked for damage after driving is completed. The driving records for all pile types can be compared with adjacent piles for unusual records or vastly different penetrations. Piles suspected of damage (including timber, H, and solid concrete piles) could be tested to confirm integrity and/or determine extent and location of damage using the pile driving analyzer, or for concrete piles, low strain integrity testing methods. These methods are discussed in Chapter 18. Alternatively, the pile could be replaced or repaired, if possible.

Check for water leakage for closed end pipe piles before placing concrete. The concrete mix should have a high slump and small aggregate. A pipe pile can be easily checked for damage and sweep by lowering a light source inside the pile.

The driving sequence of piles in a pier or bent can be important. The driving sequence can affect the way piles drive as well as the influence the new construction has on adjacent structures. This is especially true for displacement piles. For non-displacement piles the driving sequence is generally not as critical.

The driving sequence of displacement pile groups should be from the center of the group outward or from one side to the other side. The preferred driving sequence of the displacement pile group shown in Figure 24.16 would be (a) by the pile number shown, (sequence 1), (b) by driving each row starting in the center and working outward (sequence 2), or (c) by driving each row starting on one side of the group and working to the other side (sequence 3).

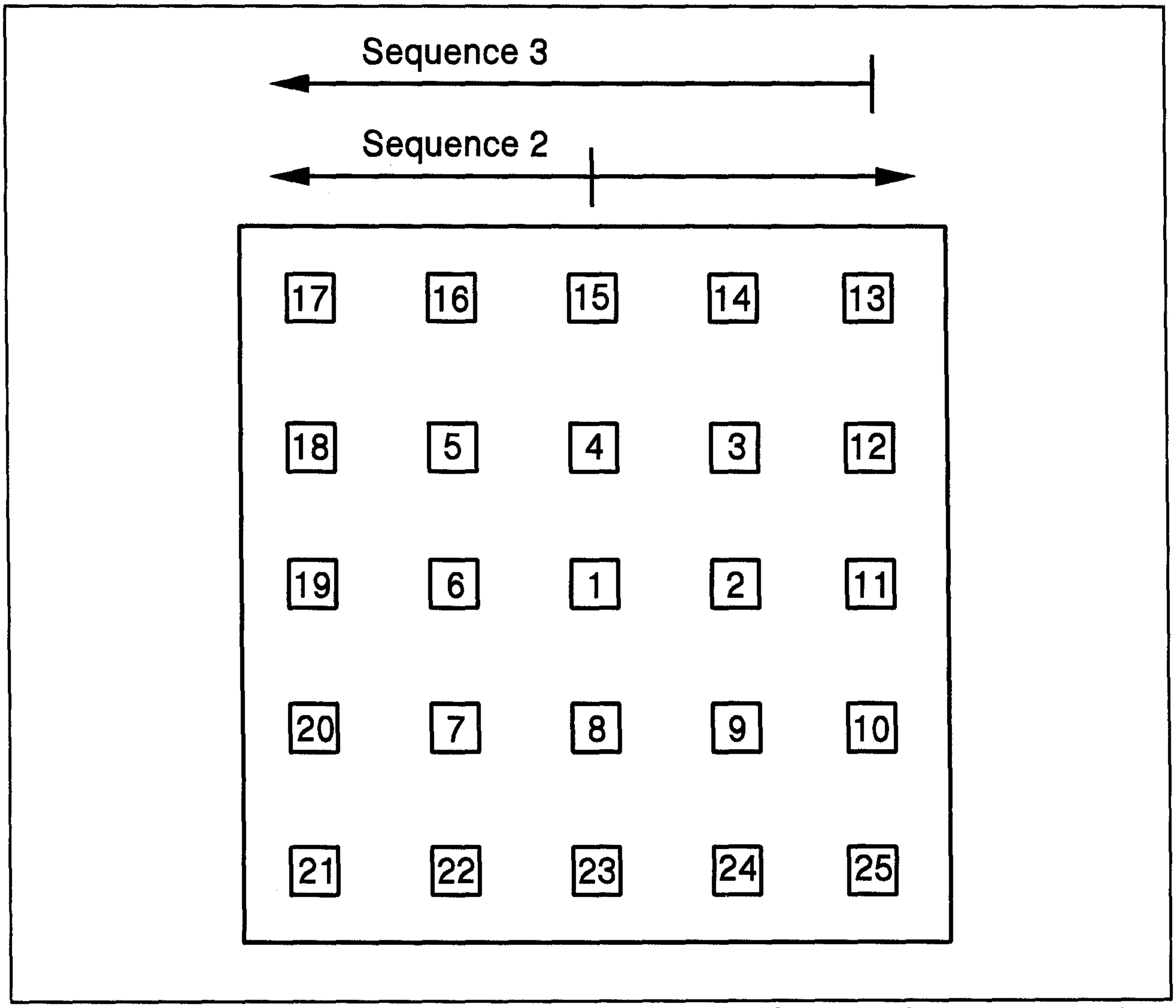


Figure 24.16 Driving Sequence of Displacement Pile Groups (after Passe, 1994)

Pile groups should not be driven from the outside to the center (the reverse of sequences 1 or 2). If groups are driven in that order, displaced soils becomes trapped and compacted in the center of the pile group. This can cause problems with driving the piles in the center of the group.

When driving close to an existing structure, it is generally preferable to drive the piles nearest the existing structure first and work away. For example, if a structure was located on the right side of the pile group shown in Figure 24.16, the piles should be driven by sequence 3. This reduces the amount of soil displaced toward the existing structure. The displacement of soil toward an existing structure has caused problems before. It can be especially critical next to a bascule bridge where, very small movements can prevent the locking mechanism from locking.

On some projects, vibration measurements may be required to ascertain if pile driving induced vibrations are within acceptable and/or specified maximum levels. Woods (1997) noted that vibration damage is relatively uncommon at a distance of one pile length away from driving. However, damage from vibration induced settlement of loose, clean sands can be a problem up to 400 m away from driving. To document existing conditions of nearby structures, a preconstruction survey of structures within 120 m of pile driving activities is often performed prior to the start of construction. The preconstruction survey generally consists of photographing or videotaping existing damage, as well as affixing crack gages to existing cracks in some cases. Woods also noted that damage to freshly placed concrete from pile driving vibrations may not be a risk but further research on the setting and curing of concrete may be warranted.

A cold hammer should not be used when restriking piles after a setup period. Twenty hammer blows are usually sufficient to warm up most hammers. Also be sure to record the restrike driving resistance for each 25 mm during the first 250 mm of restrike.

A summary of common pile installation problems and possible solutions is presented in Table 24-5.

TABLE 24-5 COMMON PILE INSTALLATION PROBLEMS & POSSIBLE SOLUTIONS	
Problem	Possible Solutions
Piles encountering refusal driving resistance (blow count) above minimum pile penetration requirements.	Have wave equation analysis performed and check that pile has sufficient driveability and that the driving system is matched to the pile. If the pile and driving system are suitably matched, check driving system operation for compliance with manufacturer's guidelines. If no obvious problems are found, dynamic measurements should be made to determine if the problem is driving system or soil behavior related. Driving system problems could include preignition, preadmission, low hammer efficiency, or soft cushion. Soil problems could include greater soil strength than anticipated, temporarily increased soil resistance with later relaxation (requires restrike to check), large soil quakes, or high soil damping.
Piles driving significantly deeper than estimated pile penetration depths.	Soil resistance at the time of driving probably is lower than anticipated or driving system performance is better than anticipated. Have wave equation analysis performed to assess ultimate pile capacity based on the blow count at the time of driving. Perform restrike tests after an appropriate waiting period to evaluate soil strength changes with time. If the ultimate capacity based on restrike blow count is still low, check drive system performance and restrike capacity with dynamic measurements. If drive system performance is as assumed and restrike capacity low, the soil conditions are weaker than anticipated. Foundation piles will probably need to be driven deeper than originally estimated or additional piles will be required to support the load. Contact the structural engineer/designer for recommended change.

TABLE 24-5 COMMON PILE INSTALLATION PROBLEMS & POSSIBLE SOLUTIONS (CONTINUED)	
Problem	Possible Solutions
Abrupt change or decrease in driving resistance (blow count) for bearing piles.	If borings do not indicate weathered profile above bedrock/bearing layer then pile toe damage is likely. Have wave equation analysis performed and evaluate pile toe stress. If calculated toe stress is high and blow counts are low, a reduced hammer energy (stroke) and higher blow count could be used to achieve capacity with a lower toe stress. If calculated toe stress is high at high blow counts, a different hammer or pile section may be required. For piles that allow internal inspection, reflect light to the pile toe and tape the length inside the pile for indications of toe damage. For piles that cannot be internally inspected, dynamic measurements could be made to evaluate problem or pile extraction could be considered for confirmation of a damage problem.
Driving resistance (blow count) significantly lower than expected during driving.	Review soil borings. If soil borings do not indicate soft layers, pile may be damaged below grade. Have wave equation analysis performed and investigate both tensile stresses along pile and compressive stresses at toe. If calculated stresses are within allowable limits, investigate possibility of obstructions / uneven toe contact on hard layer or other reasons for pile toe damage. If pile was spliced, re-evaluate splice detail and field splicing procedures for possible splice failure.
Vertical (heave) or lateral movement of previously installed piles when driving new piles.	Pile movements likely due to soil displacement from adjacent pile driving. Contact geotechnical engineer for recommended action. Possible solutions include redriving of installed piles, change in sequence of pile installation, or predrilling of pile locations to reduce ground movements. Lateral pile movements could also result from adjacent slope failure in applicable conditions.

TABLE 24-5 COMMON PILE INSTALLATION PROBLEMS & POSSIBLE SOLUTIONS (CONTINUED)	
Problem	Possible Solutions
Piles driving out of alignment tolerance.	Piles may be moving out of alignment tolerance due to hammer-pile alignment control or due to soil conditions. If due to poor hammer-pile alignment control, a pile gate, template or fixed lead system may improve the ability to maintain alignment tolerance. Soil conditions such as near surface obstructions (see subsequent section) or steeply sloping bedrock having minimal overburden material (pile point detail is important) may prevent tolerances from being met even with good alignment control. In these cases, survey the as-built condition and contact the structural engineer for recommended action.
Piles driving out of location tolerance.	Piles may be moving out of location tolerance due to hammer-pile alignment control or due to soil conditions. If due to poor hammer-pile alignment control, a pile gate, template or fixed lead system may improve the ability to maintain location tolerance. Soil conditions such as near surface obstructions (see subsequent section) or steeply sloping bedrock having minimal overburden material (pile point detail is important) may prevent tolerances from being met even with good alignment control. In these cases, survey the as-built condition and contact the structural engineer for recommended action.
Piles encountering shallow obstructions.	If obstructions are within 3 m of working grade, obstruction excavation and removal is probably feasible. If obstructions are at deeper depth, are below the water table, or the soil is contaminated, excavation may not be feasible. Spudding or predrilling of pile locations may provide a solution with method selection based on the type of obstructions and soil conditions.

TABLE 24-5 COMMON PILE INSTALLATION PROBLEMS & POSSIBLE SOLUTIONS (CONTINUED)	
Problem	Possible Solutions
Piles encountering obstructions at depth.	If deep obstructions are encountered that prevent reaching the desired pile penetration depth, contact the structural engineer/designer for remedial design. Ultimate capacity of piles hitting obstructions should be reduced based upon pile damage potential and soil matrix support characteristics. Additional foundation piles may be necessary.
Concrete piles develop complete horizontal cracks in easy driving.	Have wave equation analysis performed and check tension stresses along pile (extrema tables) for the observed blow counts. If the calculated tension stresses are high, add cushioning or reduce stroke. If calculated tension stresses are low, check hammer performance and/or perform dynamic measurements.
Concrete piles develop complete horizontal cracks in hard driving.	Have wave equation analysis performed and check tension stresses along pile (extrema table). If the calculated tension stresses are high, consider a hammer with a heavier ram. If the calculated tension stresses are low, perform dynamic measurements and evaluate soil quakes which are probably higher than anticipated.
Concrete piles develop partial horizontal cracks in easy driving.	Check hammer-pile alignment since bending may be causing the problem. If the alignment appears to be normal, tension and bending combined may be too high. The possible solution is as above with complete cracks.
Concrete pile spalling or slabbing near pile head.	Have wave equation analysis performed. Determine the pile head stress at the observed blow count and compare predicted stress with allowable material stress. If the calculated stress is high, increase the pile cushioning. If the calculated stress is low, investigate pile quality, hammer performance, and hammer-pile alignment.

TABLE 24-5 COMMON PILE INSTALLATION PROBLEMS & POSSIBLE SOLUTIONS (CONTINUED)	
Problem	Possible Solutions
Steel pile head deforms.	Check helmet size/shape, steel yield strength, and evenness of the pile head. If all seem acceptable, have wave equation analysis performed and determine the pile head stress. If the calculated stress is high and blow counts are low, use reduced hammer energy (stroke) and higher blow count to achieve capacity. If the calculated stress is high at high blow counts, a different hammer or pile type may be required. Ultimate capacity determination should not be made using blow counts obtained when driving with a deformed pile head.
Timber pile head mushrooms	Check helmet size/shape, the evenness of the pile head, and banding of the timber pile head. If all seem acceptable, have wave equation analysis performed and determine the pile head stress. If the calculated stress is high and blow counts are low, use reduced hammer energy (stroke) and higher blow count to achieve capacity. Ultimate capacity determination should not be made using blow counts obtained when driving with a mushroomed pile head.

24.9 DRIVING RECORDS AND REPORTS

Pile driving records vary with the organization performing the inspection service. A typical pile driving record is presented in Figure 24.17. The following is a list of items that appear on most pile driving records:

1. Project identification number.

2. Project name and location.

3. Structure identification number.

PILE DRIVING LOG

STATE PROJECT NO.: ____________________ DATE: ____________

JOB LOCATION: ____________________

PILE TYPE: ________ LENGTH: ________ BENT/PIER NO.: ________ PILE NO.: ________

HAMMER: ________ ENERGY/BLOW: ________ OPERATING RATE: ________ HELMET WEIGHT: ________

REF. ELEV.: ________ PILE TOE ELEV.: ________ PILE CUTOFF ELEV.: ________

PILE CUSHION THICKNESS AND MATERIAL: ____________________

WEATHER: ________ TEMP.: ________ START TIME: ________ STOP TIME: ________

METERS	BLOWS	STROKE / PRESSURE	REMARKS	METERS	BLOWS	STROKE / PRESSURE	REMARKS
0 - 0.25				8.00 - 8.25			
0.25 - 0.50				8.25 - 8.50			
0 50 - 0.75				8.50 - 8.75			
0.75 - 1.00				8.75 - 9.00			
1.00 - 1.25				9.00 - 9.25			
1.25 - 1.50				9.25 - 9.50			
1.50 - 1.75				9.50 - 9.75			
1.75 - 2.00				9.75 - 10.00			
2.00 - 2.25				10.00 - 10.25			
2.25 - 2.50				10.25 - 10.50			
2.50 - 2.75				10.50 - 10.75			
2.75 - 3.00				10.75 - 11.00			
3.00 - 3.25				11.00 - 11.25			
3.25 - 3.50				11.25 - 11.50			
3.50 - 3.75				11.50 - 11.75			
3.75 - 4.00				11.75 - 12.00			
4.00 - 4.25				12.00 - 12.25			
4.25 - 4.50				12.25 - 12.50			
4.50 - 4.75				12.50 - 12.75			
4.75 - 5.00				12.75 - 13.00			
5.00 - 5.25				13.00 - 13.25			
5.25 - 5.50				13.25 - 13.50			
5.50 - 5.75				13.50 - 13.75			
5.75 - 6.00				13.75 - 14.00			
6.00 - 6.25				14.00 - 14.25			
6.25 - 6.50				14.25 - 14.50			
6.50 - 6.75				14.50 - 14.75			
6.75 - 7.00				14.75 - 15.00			
7.00 - 7.25				15.00 - 15.25			
7.25 - 7.50				15.25 - 15.50			
7.50 - 7.75				15.50 - 15.75			
7.75 - 8.00				15.75 - 16.00			

PILE INFORMATION: ________ MANUFACTURED BY: ________

WORK ORDER NO.: ________ DATE CAST: ________

MANUFACTURER'S PILE NO.: ________ PILE HEAD CONDITIONS: ________

PILE TOE ATTACHMENTS: ________ SIGNATURE: ________

Figure 24.17 Pile Driving Log

4. Date and time of driving (start, stop, and interruptions).
5. Name of the contractor.
6. Hammer make, model, ram weight, energy rating. The actual stroke and operating speed should also be recorded whenever it is changed.
7. Hammer cushion description, size and thickness, and helmet weight.
8. Pile cushion description, size and thickness, depth where changed.
9. Pile location, type, size and length.
10. Pile number or designation matching pile layout plans.
11. Pile ground surface, cut off, and final penetration elevations and embedded length.
12. Driving resistance data in blows per 0.25 meter with the final 0.25 meter normally recorded in blows per 25 mm.
13. Graphical presentation of driving data (optional).
14. Cut-off length, length in ground and order length.
15. Comments or unusual observations, including reasons for all interruptions.
16. Signature and title of the inspector.

The importance of maintaining detailed pile driving records can not be overemphasized. The driving records form a basis for payment and for making engineering decisions regarding the adequacy of the foundation to support the design loads. Great importance is given to driving records in litigations involving claims. Sloppy, inaccurate, or incomplete records encourage claims and result in higher cost foundations. The better the pile driving is documented, the lower the cost of the foundation will probably be and the more likely it will be completed on schedule.

In addition to the driving records, the inspector should be required to prepare a daily inspection report. The daily inspection report should include information on equipment working at the site, description of construction work accomplished, and the progress of work. Figure 24.18 shows an example of a daily inspection report.

DAILY INSPECTION REPORT

Project No.: ______________________

Date: ______________________

Project: ____

Weather Conditions: ____

Contractor: ____

Contractor's Personnel Present: ____

____

____

Equipment Working: ____

____

Description of Work Accomplished: ____

____

____

____

Special Persons Visiting Job: ____

____

Test Performed: ____

____

Special Comments: ____

____

____

Figure 24.18 Daily Inspection Report

REFERENCES

Associated Pile and Fitting Corporation. Design and Installation of Driven Pile Foundations.

Canadian Geotechnical Society (1978). Canadian Foundation Engineering Manual. Part 3, Deep Foundations.

Deep Foundations Institute (1981). Glossary of Foundation Terms.

Deep Foundations Institute (1995). A Pile Inspector's Guide to Hammers, Second Edition Deep Foundations Institute.

Fuller, F.M. (1983). Engineering of Pile Installations. McGraw-Hill Book Company.

Passe, Paul D. (1994). Pile Driving Inspector's Manual. State of Florida Department of Transportation.

Rausche, F., Likins, G.E., Goble, G.G., Hussien, M. (1986). The Performance of Pile Driving Systems; Inspector's Manual.

Williams Earth Sciences (1995). Inspector's Qualification Program for Pile Driving Inspection Manual. State of Florida Department of Transportation.

Woods, R.D. (1997). Dynamic Effects of Pile Installations on Adjacent Structures. NCHRP Synthesis 253, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.

STUDENT EXERCISE #15 - HAMMER INSPECTION

You are inspecting the pile driving operations on two bridge projects. On the first project, Bridge #1, the contractor is using a single acting diesel hammer. The driving criteria with this hammer has been established as follows:

Minimum Toe Elevation:	EL 96.5 m
Minimum Driving Resistance:	80 blows / 250 mm at a 3.0 m stroke.

The driving record for the first pile driven is attached. The hammer operating speed was timed at 40 blows per minute at final driving. Has this pile met the driving criteria ?

STEP 1. Calculate the stroke hammer stroke basedon the recorded hammer operating speed using the formula on page 24-22.

STEP 2. Determine the pile toe elevation.

STEP 3. Based on hammer stroke, driving resistance and pile toe elevation, determine if the pile has met the driving criteria.

On the second project, Bridge #2, the contractor is using a double acting diesel hammer. The bounce chamber - equivalent energy correlation for the hammer as provided by the contractor in the equipment submittal is attached. The driving criteria on the second project has been established as follows:

Minimum Toe Elevation:	EL 80
Minimum Driving Resistance:	60 blows / 250 mm at a bounce chamber pressure of 180 kPa. (Based on 15.2 m of hose.)

The driving record for the first production pile driven on this project is attached. The hose between the bounce chamber pressure is 24.4 m long. Has this pile met the driving criteria?

STEP 1. Determine equivalent hammer energy based on the bounce chamber pressure on the driving log.

STEP 2. Compare observed equivalent hammer energy with required energy.

STEP 3. Based on observed hammer energy, driving resistance and pile toe elevation, determine if the pile has met the driving criteria.

PILE DRIVING LOG

STATE PROJECT NO.: Bridge #1 DATE: 5-29-98

JOB LOCATION: Bogalusa

PILE TYPE: 457 mm PCC LENGTH: 15 m BENT/PIER NO.: 1 PILE NO.: 1

HAMMER: D-30-32 ENERGY/BLOW: 99.9 kJ OPERATING RATE: 36-52 BPM HELMET WEIGHT: 14.5 kN

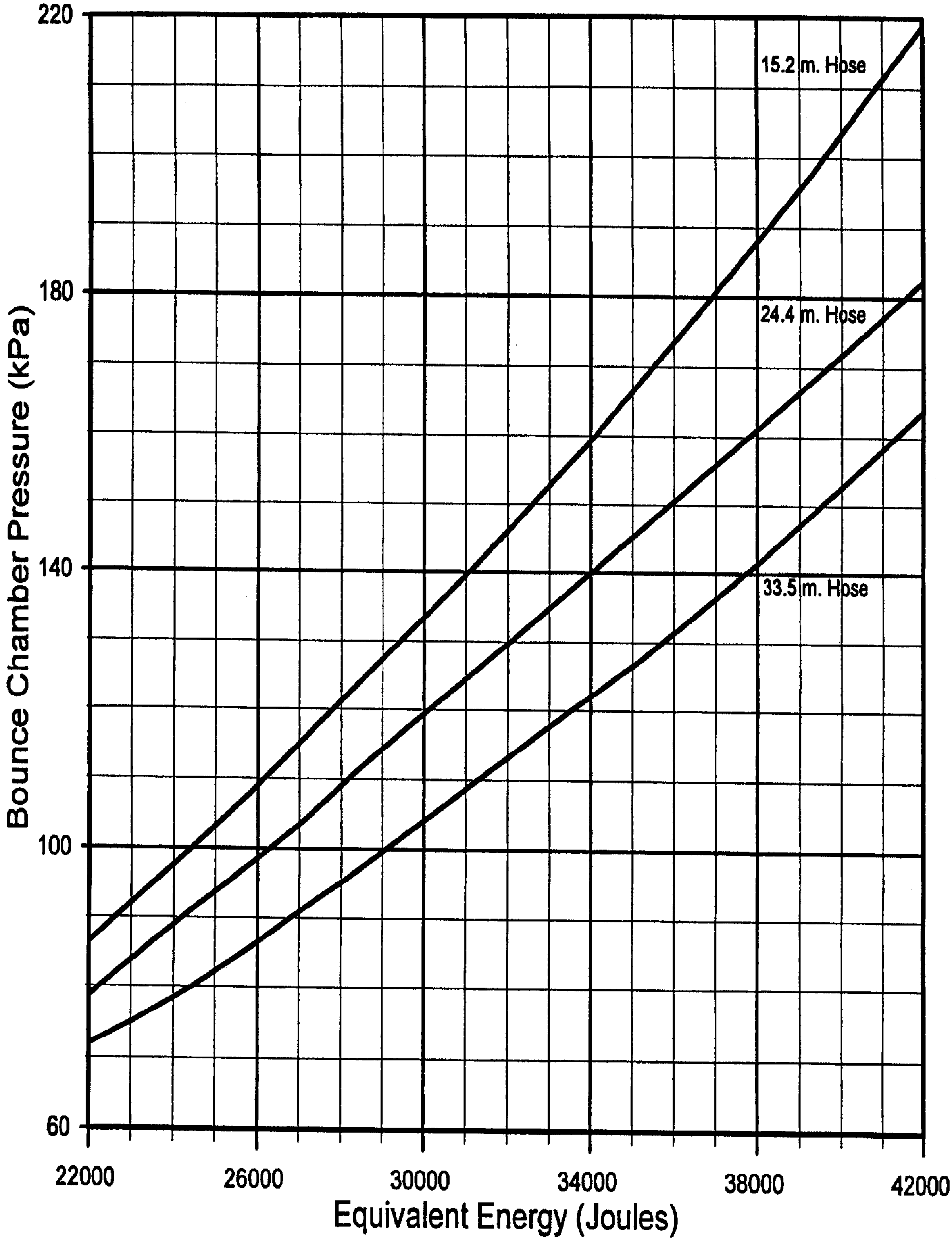
REF. ELEV.: 109.5 m PILE TOE ELEV.: PILE CUTOFF ELEV.: 108.3 m

PILE CUSHION THICKNESS AND MATERIAL: 190 mm of plywood

WEATHER: sunny TEMP.: 80° START TIME: 8:23 am STOP TIME: 8:58 am

METERS	BLOWS	STROKE / PRESSURE	REMARKS	METERS	BLOWS	STROKE / PRESSURE	REMARKS
0 - 0.25	W.O.P			8.00 - 8.25	25		
0.25 - 0.50	W.O.P			8.25 - 8.50	21	51 BPM	
0 50 - 0.75	W.O.P			8.50 - 8.75	23		
0.75 - 1.00	W.O.P			8.75 - 9.00	26		
1.00 - 1.25	W.O.P			9.00 - 9.25	22	51 BPM	
1.25 - 1.50	W.O.P			9.25 - 9.50	21		
1.50 - 1.75	W.O.H			9.50 - 9.75	23		
1.75 - 2.00	W.O.H			9.75 - 10.00	24	51 BPM	
2.00 - 2.25	W.O.H			10.00 - 10.25	22		
2.25 - 2.50	5		Fuel #2	10.25 - 10.50	26		
2.50 - 2.75	6	52 BPM		10.50 - 10.75	30	44 BPM	
2.75 - 3.00	8			10.75 - 11.00	34		
3.00 - 3.25	10			11.00 - 11.25	40		
3.25 - 3.50	12			11.25 - 11.50	51	43 BPM	
3.50 - 3.75	17	50 BPM		11.50 - 11.75	38	42 BPM	Fuel #4
3.75 - 4.00	22			11.75 - 12.00	41		
4.00 - 4.25	30	49 BPM		12.00 - 12.25	42	42 BPM	
4.25 - 4.50	21	47 BPM	Fuel #3	12.25 - 12.50	53		
4.50 - 4.75	24			12.50 - 12.75	58	41 BPM	
4.75 - 5.00	27			12.75 - 13.00	65		
5.00 - 5.25	29			13.00 - 13.25	77	40 BPM	
5.25 - 5.50	31	45 BPM		13.25 - 13.50	80	40 BPM	
5.50 - 5.75	32			13.50 - 13.75			
5.75 - 6.00	32			13.75 - 14.00			
6.00 - 6.25	35	45 BPM		14.00 - 14.25			
6.25 - 6.50	31			14.25 - 14.50			
6.50 - 6.75	25			14.50 - 14.75			
6.75 - 7.00	21	47 BPM		14.75 - 15.00			
7.00 - 7.25	18			15.00 - 15.25			
7.25 - 7.50	20			15.25 - 15.50			
7.50 - 7.75	19	51 BPM		15.50 - 15.75			
7.75 - 8.00	22			15.75 - 16.00			

Bounce Chamber Pressure vs. Equivalent Energy



PILE DRIVING LOG

STATE PROJECT NO.: Bridge #2 DATE: 5-29-98

JOB LOCATION: Hoboken

PILE TYPE: 324 mm CEP LENGTH: 15.5 m BENT/PIER NO.: 4 PILE NO.: 1

HAMMER: LB 520 ENERGY/BLOW: 35.7 kJ OPERATING RATE: 80-84 BPM HELMET WEIGHT: 8.9 kN

REF. ELEV.: 91.25 PILE TOE ELEV.: PILE CUTOFF ELEV.: 94.1 m

PILE CUSHION THICKNESS AND MATERIAL: none

WEATHER: cloudy TEMP.: 75° START TIME: 10:52 STOP TIME: 11:09

METERS	BLOWS	STROKE / PRESSURE	REMARKS	METERS	BLOWS	STROKE / PRESSURE	REMARKS
0 - 0.25	W.O.H.		24.4 m hose	8.00 - 8.25	38		
0.25 - 0.50	W.O.H.			8.25 - 8.50	37	BCP 160	
0 50 - 0.75	W.O.H.			8.50 - 8.75	39		
0.75 - 1.00	W.O.H.			8.75 - 9.00	41		
1.00 - 1.25	3			9.00 - 9.25	40		
1.25 - 1.50	5			9.25 - 9.50	39	BCP 160	
1.50 - 1.75	6			9.50 - 9.75	42		
1.75 - 2.00	5			9.75 - 10.00	41		
2.00 - 2.25	6			10.00 - 10.25	44	BCP 160	
2.25 - 2.50	4	BCP 110		10.25 - 10.50	50		
2.50 - 2.75	5			10.50 - 10.75	51		
2.75 - 3.00	6			10.75 - 11.00	53	BCP 165	
3.00 - 3.25	8	BCP 115		11.00 - 11.25	51		min pen
3.25 - 3.50	10			11.25 - 11.50	54		
3.50 - 3.75	12			11.50 - 11.75	55	BCP 170	
3.75 - 4.00	20	BCP 125		11.75 - 12.00	57		
4.00 - 4.25	22			12.00 - 12.25	58	BCP 170	
4.25 - 4.50	21			12.25 - 12.50	60		
4.50 - 4.75	20			12.50 - 12.75	65	BCP 175	
4.75 - 5.00	23	BCP 135		12.75 - 13.00			
5.00 - 5.25	21			13.00 - 13.25			
5.25 - 5.50	25			13.25 - 13.50			
5.50 - 5.75	28	BCP 150		13.50 - 13.75			
5.75 - 6.00	30			13.75 - 14.00			
6.00 - 6.25	33			14.00 - 14.25			
6.25 - 6.50	32	BCP 155		14.25 - 14.50			
6.50 - 6.75	33			14.50 - 14.75			
6.75 - 7.00	35			14.75 - 15.00			
7.00 - 7.25	33	BCP 155		15.00 - 15.25			
7.25 - 7.50	37			15.25 - 15.50			
7.50 - 7.75	36			15.50 - 15.75			
7.75 - 8.00	33	BCP 155		15.75 - 16.00			

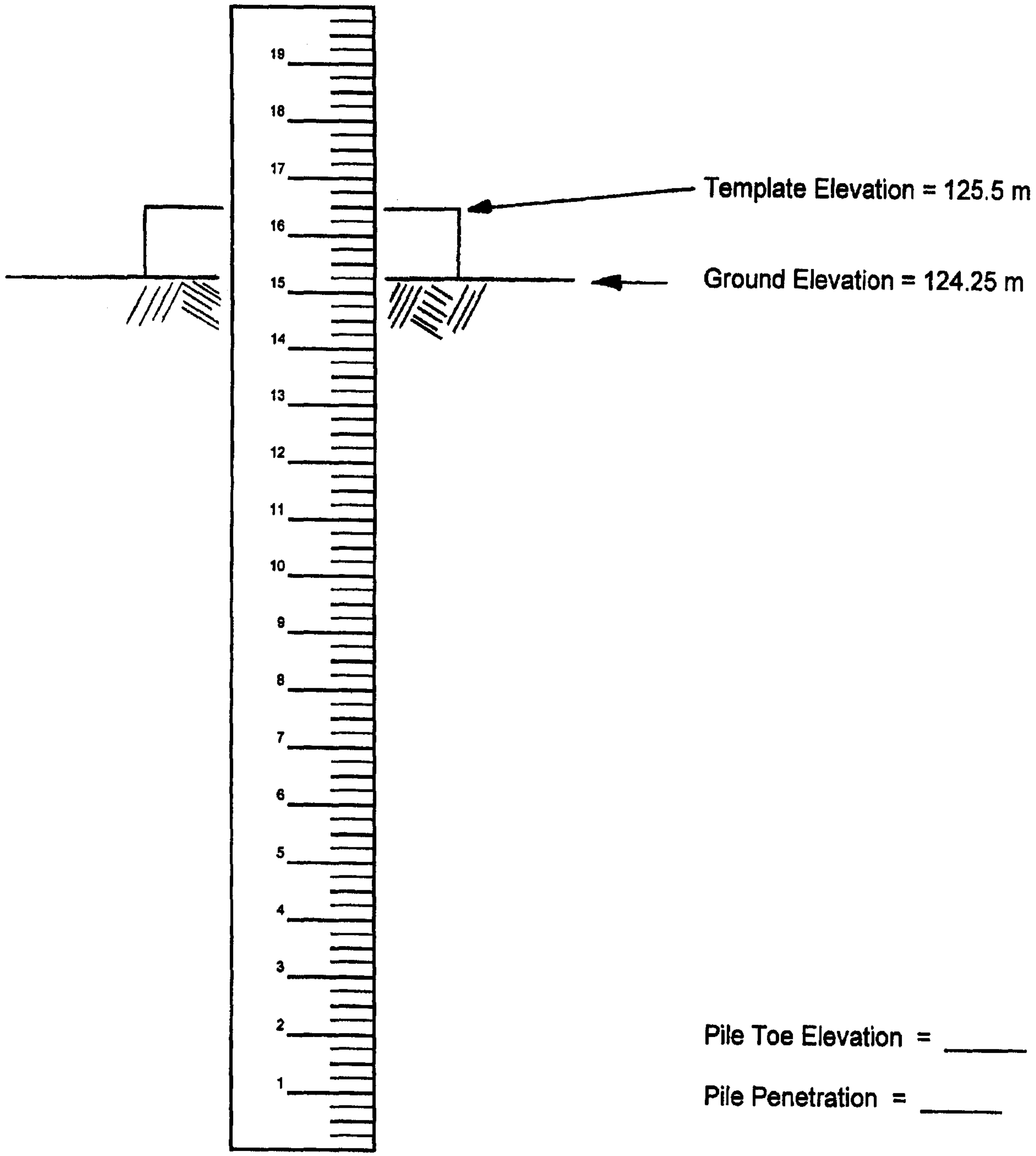
STUDENT EXERCISE #16 - DETERMINING PILE TOE ELEVATIONS

Pile driving criteria often include obtaining a specified driving resistance in conjunction with a pile penetration requirement or pile toe elevation. For many land based driving situations determination of the pile toe elevation is a relatively straightforward task. For batter pile driving and pile installations over water, determination of the pile toe elevation can be more problematic.

The following pages contain pile installation illustrations where the reference elevation is given and the pile penetration shown. For each example, calculate the final pile toe elevation and pile penetration depth.

STUDENT EXERCISE 16a - DETERMINING PILE TOE ELEVATIONS

Land Pile Installation

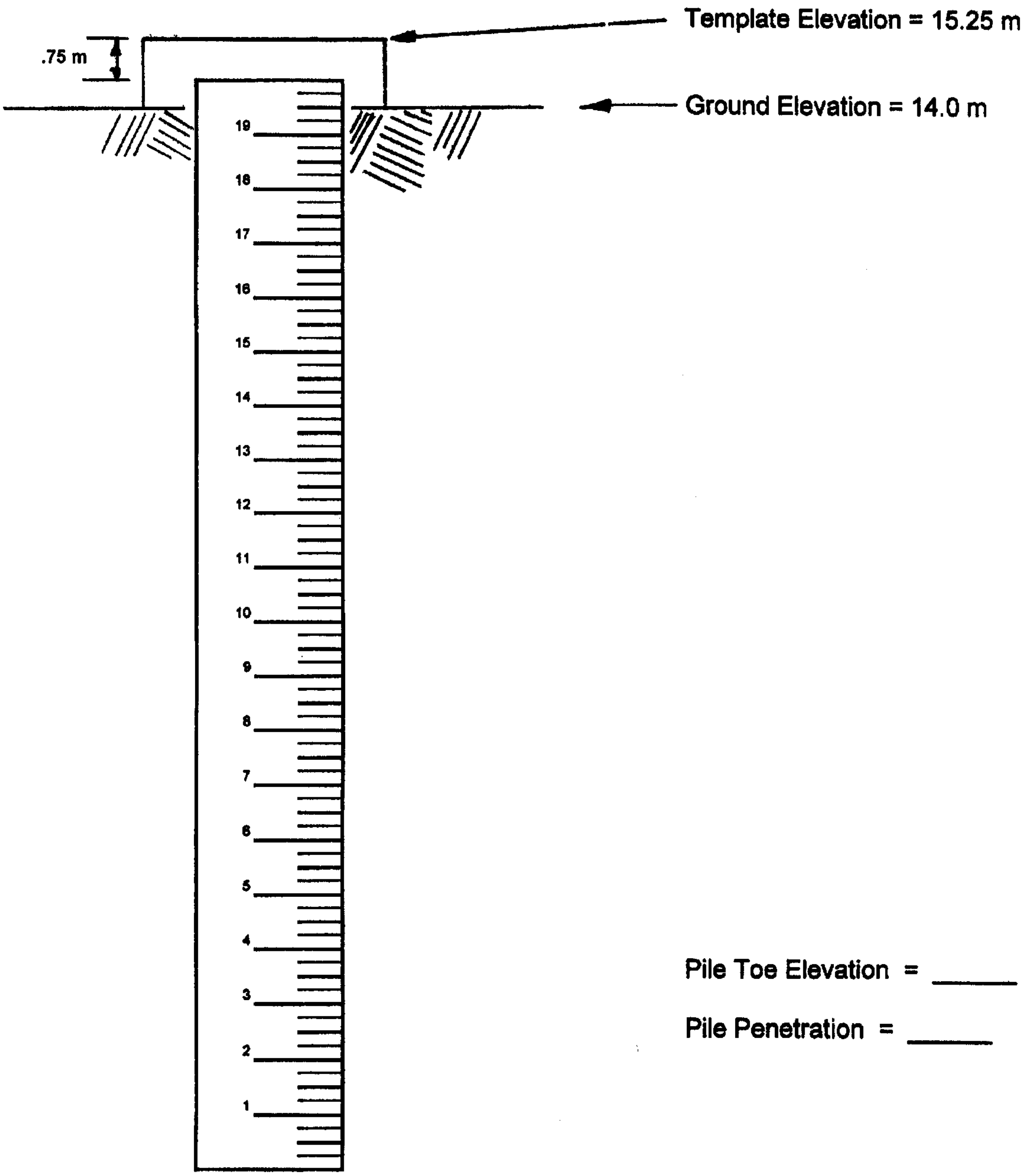


Pile Toe Elevation = ______

Pile Penetration = ______

STUDENT EXERCISE #16b - DETERMINING PILE TOE ELEVATIONS

Land Pile Installation

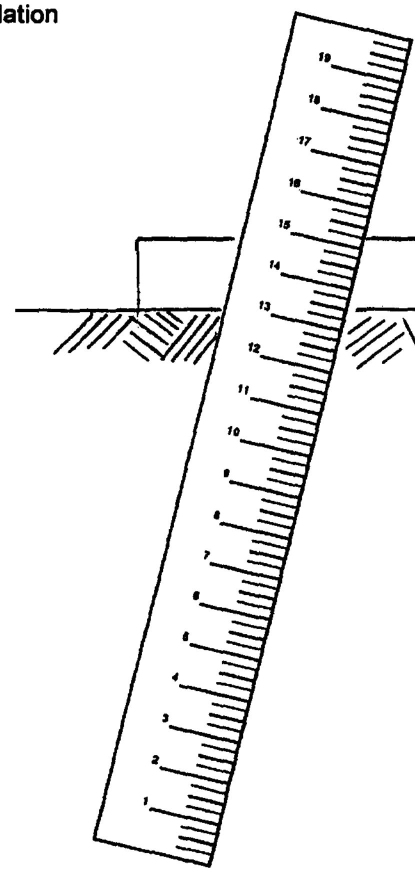


Pile Toe Elevation = ______

Pile Penetration = ______

STUDENT EXERCISE #16c - DETERMINING PILE TOE ELEVATIONS

Batter Pile Installation



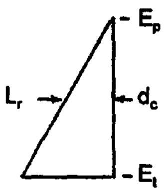
Batter Angle (1H:4V)

Pile Toe Elevation = ______

Pile Penetration = ______

Calculating Pile Toe Elevation of Batter Piles

Batter Angle	Correction Factor, (B_c)
1H : 12V	.997
1.5H : 12V	.992
2H : 12V (1H : 6V)	.986
3H : 12V (1H : 4V)	.971
4H : 12V (1H : 3V)	.949
5H : 12V	.923



Batter Angle

Definitions

L_r = Pile Length Below Reference Point (m)
E_p = Reference Point Elevation (m)
d_c = Corrected Pile Depth (m)
E_t = Pile Toe Elevation

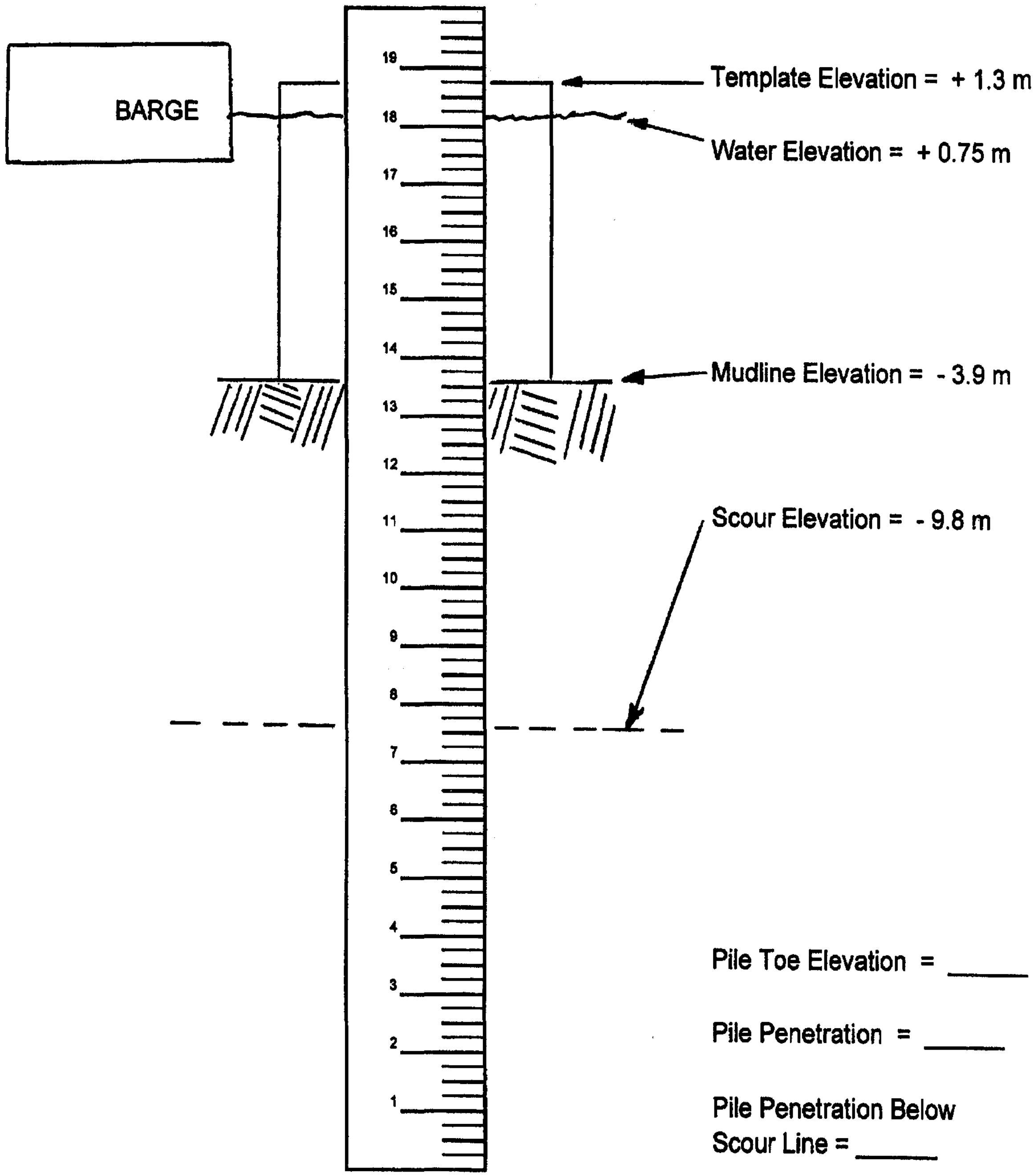
Formulas

$d_c = (L_r)(B_c)$

$E_t = E_p - d_c$

STUDENT EXERCISE #16d - DETERMINING PILE TOE ELEVATIONS

Pile Installation over Water



Pile Toe Elevation = ______

Pile Penetration = ______

Pile Penetration Below
Scour Line = ______

APPENDIX A

List of FHWA Pile Foundation Design and Construction References

Briaud, J-L. (1989). The Pressuremeter Test for Highway Applications. Report No. FHWA IP-89-008, U.S. Department of Transportation, Federal Highway Administration, Office of Implementation, McLean, 156.

Briaud, J-L. and Miran, J. (1991). The Cone Penetrometer Test. Report No. FHWA-SA-91-043, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 161.

Briaud, J-L. and Miran, J. (1992). The Flat Dilatometer Test. Report No. FHWA-SA-91-44, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 102.

Briaud, J-L., Tucker, L., Lytton, R.L. and Coyle, H.M. (1985). Behavior of Piles and Pile Groups in Cohesionless Soils. Report No. FHWA/RD-83/038, U.S. Department of Transportation, Federal Highway Administration, Office of Research - Materials Division, Washington, D.C., 233.

Cheney, R.S. and Chassie, R.G. (1993). Soils and Foundations Workshop Manual. Second Edition, Report No. HI-88-009, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering, Washington, D.C., 395.

Goble, G.G., and Rausche, F. (1986). Wave Equation Analysis of Pile Driving - WEAP86 Program. U.S. Department of Transportation, Federal Highway Administration, Implementation Division, McLean, Volumes I-IV.

Kyfor, Z.G., Schnore, A.S., Carlo, T.A. and Bailey, P.F. (1992). Static Testing of Deep Foundations. Report No. FHWA-SA-91-042, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 174.

Lam, I.P. and Martin, G.R. (1986). Seismic Design of Highway Bridge Foundations. Volume II - Design Procedures and Guidelines, Report No. FHWA/RD-86/102, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering and Highway Operations, McLean, 181.

Mathias, D. and Cribbs, M. (1998). DRIVEN 1.0: A Microsoft Windows™ Based Program for Determining Ultimate Vertical Static Pile Capacity. Report No. FHWA-SA-98-074, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington D.C. 112.

Osterberg, J.O. (1995). The Osterberg Cell for Load Testing Drilled Shafts and Driven Piles. Report No. FHWA-SA-94-035, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 92.

Rausche, F., Likins, G.E., Goble, G.G. and Miner, R. (1985). The Performance of Pile Driving Systems. Main Report, U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development, Washington, D.C., Volumes I-IV.

Reese, L.C. (1984). Handbook on Design of Piles and Drilled Shafts Under Lateral Load. Report No. FHWA-IP-84 11, U.S. Department of Transportation, Federal Highway Administration, Office of Implementation, McLean, 386.

Urzua, A. (1992). SPILE A Microcomputer Program for Determining Ultimate Vertical Static Pile Capacity. Users Manual, Report No. FHWA-SA-92-044, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering and Office of Technology Applications, Washington, D.C., 58.

Wang, S-T, and Reese, L.C. (1993). COM624P - Laterally Loaded Pile Analysis Program for the Microcomputer, Version 2.0. Report No. FHWA-SA-91-048, U.S. Department of Transportation, Federal Highway Administration, Office of Technology Applications, Washington, D.C., 504.

APPENDIX B

List of ASTM Pile Design and Testing Specifications

DESIGN

Standard Specification for Welded and Seamless Steel Pipe Piles.
ASTM Designation: A 252

Standard Specification for Round Timber Piles.
ASTM Designation: D 25

Standard Method for Establishing Design Stresses for Round Timber Piles.
ASTM Designation: D 2899

Standard Methods for Establishing Clear Wood Strength Values.
ASTM Designation: D 2555

TESTING

Standard Method for Testing Piles under Axial Compressive Load.
ASTM Designation: D 1143

Standard Method for Testing Individual Piles under Static Axial Tensile Load.
ASTM Designation: D 3689

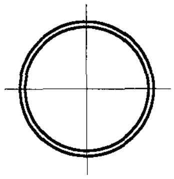
Standard Method for Testing Piles under Lateral Load.
ASTM Designation: D 3966

Standard Test Method for High Strain Dynamic Testing of Piles.
ASTM Designation: D 4945

Standard Test Method for Low Strain Dynamic Testing of Piles.
ASTM Designation: D 5882

APPENDIX C

Information and Data on Various Pile Types



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP203	3.58	2,245	173	11.197	110.12	70.61	0.64	30,193	0.0301	266
	4.17	2,607	200	12.903	127.00	70.36	0.64	29,806	0.0298	422
	4.37	2,729	210	13.486	132.74	70.36	0.64	29,677	0.0296	487
	4.55	2,839	218	13.985	137.82	70.36	0.64	29,613	0.0296	548
	4.78	2,974	229	14.651	144.21	70.10	0.64	29,484	0.0293	621
	5.56	3,452	266	16.857	165.51	69.85	0.64	28,968	0.0291	874
PP219	2.77	1,884	145	10.989	100.45	76.45	0.69	35,806	0.0359	97
	3.18	2,155	166	12.570	114.55	76.45	0.69	35,548	0.0356	147
	3.58	2,426	187	14.069	128.47	76.20	0.69	35,290	0.0354	212
	3.96	2,678	206	15.484	141.42	75.95	0.69	35,032	0.0351	288
	4.17	2,813	216	16.233	148.30	75.95	0.69	34,903	0.0349	335
	4.37	2,949	227	16.982	155.02	75.95	0.69	34,774	0.0349	388
	4.55	3,065	236	17.648	160.92	75.95	0.69	34,645	0.0346	438
	4.78	3,213	247	18.481	168.79	75.69	0.69	34,452	0.0344	508
	5.16	3,465	266	19.813	180.26	75.69	0.69	34,258	0.0341	623
	5.56	3,729	287	21.269	195.01	75.44	0.69	33,935	0.0339	744
	6.35	4,245	326	24.017	219.59	75.18	0.69	33,419	0.0334	979
	7.04	4,684	360	26.389	240.89	74.93	0.69	33,032	0.0331	1,180
	7.92	5,258	404	29.344	267.11	74.68	0.69	32,452	0.0324	1,500
	8.18	5,420	417	30.177	275.30	74.68	0.69	32,258	0.0324	1,600
	8.74	5,775	444	31.967	291.69	74.42	0.69	31,935	0.0319	1,820
	9.53	6,271	482	34.506	314.63	74.17	0.69	31,419	0.0314	2,120
	10.31	6,775	520	36.920	337.57	73.91	0.69	30,903	0.0309	2,420
	11.13	7,291	559	39.417	358.88	73.66	0.69	30,452	0.0304	2,740
	12.70	8,259	633	44.121	401.48	73.15	0.69	29,484	0.0293	3,340

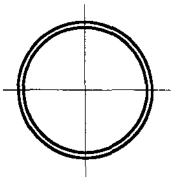
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP254	2.77	2,187	168	17.232	135.68	88.90	0.80	48,516	0.0484	62
	3.05	2,400	185	18.939	148.96	88.65	0.80	48,258	0.0482	83
	3.40	2,678	206	21.020	165.51	88.65	0.80	48,000	0.0479	116
	3.58	2,820	217	22.102	173.70	88.65	0.80	47,871	0.0479	135
	3.81	2,994	230	23.434	185.17	88.39	0.80	47,677	0.0477	163
	4.17	3,271	251	25.515	201.56	88.39	0.80	47,419	0.0474	214
	4.37	3,426	263	26.680	209.75	88.39	0.80	47,226	0.0472	247
	4.55	3,562	274	27.721	217.95	88.14	0.80	47,097	0.0472	279
	4.78	3,742	287	29.053	229.42	88.14	0.80	46,903	0.0469	324
	5.16	4,033	310	31.217	245.81	87.88	0.80	46,645	0.0467	409
	5.56	4,342	334	33.507	263.83	87.88	0.80	46,322	0.0464	515
	5.84	4,555	350	35.088	276.94	87.88	0.80	46,129	0.0462	588
	6.35	4,942	380	37.919	298.24	87.63	0.80	45,742	0.0457	719
PP273	2.77	2,349	181	21.478	157.32	95.50	0.86	56,193	0.0562	50
	3.05	2,587	199	23.559	172.06	95.50	0.86	56,000	0.0559	67
	3.18	2,690	207	24.516	180.26	95.50	0.86	55,871	0.0559	76
	3.40	2,884	222	26.223	191.73	95.25	0.86	55,677	0.0557	93
	3.58	3,032	233	27.513	201.56	95.25	0.86	55,548	0.0554	109
	3.81	3,226	248	29.219	214.67	95.25	0.86	55,355	0.0554	131
	3.96	3,349	258	30.343	222.86	95.25	0.86	55,226	0.0552	148
	4.17	3,516	271	31.800	232.70	95.00	0.86	55,032	0.0549	172
	4.37	3,691	284	33.299	244.17	95.00	0.86	54,839	0.0549	199
	4.55	3,832	295	34.589	254.00	95.00	0.86	54,710	0.0547	224
	4.78	4,026	310	36.212	265.47	94.74	0.86	54,516	0.0544	260
	5.16	4,342	334	38.959	285.13	94.74	0.86	54,193	0.0542	328
	5.56	4,679	359	41.623	306.44	94.49	0.86	53,871	0.0539	414

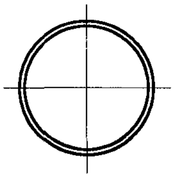
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties I	S	r	Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP273 (cont'd)	5.84	4,904	377	43.704	321.19	94.49	0.86	53,677	0.0537	480
	6.35	5,323	409	47.450	347.41	94.23	0.86	53,226	0.0532	605
	7.09	5,923	455	52.445	383.46	93.98	0.86	52,645	0.0527	781
	7.80	6,517	500	57.024	419.51	93.73	0.86	52,064	0.0522	951
	8.74	7,226	558	63.267	465.39	93.47	0.86	51,290	0.0514	1,180
	9.27	7,678	591	67.013	489.97	93.22	0.86	50,903	0.0509	1,320
	11.13	9,162	704	78.668	576.82	92.71	0.86	49,419	0.0494	1,890
	12.70	10,389	799	88.241	645.65	92.20	0.86	48,193	0.0482	2,380
PP305	3.40	3,226	248	36.587	240.89	106.68	0.96	69,677	0.0697	67
	3.58	3,387	261	38.460	252.36	106.43	0.96	69,677	0.0695	78
	3.81	3,600	277	40.791	267.11	106.43	0.96	69,677	0.0695	94
	4.17	3,936	303	44.537	291.69	106.43	0.96	69,032	0.0690	123
	4.37	4,123	317	46.618	304.80	106.17	0.96	69,032	0.0687	142
	4.55	4,291	330	48.283	317.91	106.17	0.96	68,387	0.0687	161
	4.78	4,503	346	50.780	332.66	106.17	0.96	68,387	0.0685	186
	5.16	4,852	373	54.526	357.24	105.92	0.96	68,387	0.0682	235
	5.56	5,233	402	58.689	383.46	105.92	0.96	67,742	0.0677	296
	5.84	5,484	422	61.186	403.12	105.66	0.96	67,742	0.0675	344
	6.35	5,955	458	66.181	435.90	105.66	0.96	67,097	0.0670	443
	7.14	6,646	513	74.089	485.06	105.16	0.96	66,451	0.0662	616
	7.92	7,420	568	81.581	534.22	104.90	0.96	65,806	0.0655	784
PP324	2.77	2,794	215	36.004	222.86	113.54	1.02	79,355	0.0795	30
	3.18	3,200	246	41.124	254.00	113.28	1.02	79,355	0.0793	45
	3.40	3,426	264	44.121	272.03	113.28	1.02	78,710	0.0790	56
	3.58	3,607	277	46.202	285.13	113.28	1.02	78,710	0.0788	65
	3.81	3,832	295	49.115	303.16	113.29	1.02	78,710	0.0785	78
	3.96	3,981	306	50.780	314.63	113.03	1.02	78,710	0.0785	88
	4.17	4,181	322	53.278	329.38	113.03	1.02	78,064	0.0783	103

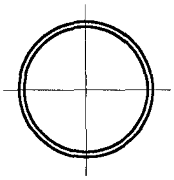
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP324 (cont'd)	4.37	4,387	337	55.775	345.77	113.03	1.02	78,064	0.0780	118
	4.55	4,562	351	58.272	358.88	113.03	1.02	78,064	0.0778	134
	4.78	4,787	368	60.770	376.90	112.78	1.02	77,419	0.0775	155
	5.16	5,162	397	65.765	404.76	112.78	1.02	77,419	0.0773	196
	5.56	5,562	428	70.343	435.90	112.52	1.02	76,774	0.0768	246
	5.84	5,839	449	73.673	455.56	112.52	1.02	76,774	0.0765	286
	6.35	6,336	487	79.916	493.25	112.27	1.02	76,129	0.0760	368
	7.14	7,097	546	89.074	550.61	112.01	1.02	75,484	0.0753	526
	7.92	7,871	605	98.231	606.32	111.76	1.02	74,193	0.0745	684
	8.38	8,323	639	103.225	639.10	111.51	1.02	74,193	0.0740	776
	8.74	8,646	665	107.388	663.68	111.51	1.02	73,548	0.0737	848
	9.53	9,420	723	116.129	717.75	111.25	1.02	72,903	0.0730	1,010
	10.31	10,131	781	124.869	771.83	111.00	1.02	72,258	0.0722	1,170
	11.13	10,905	840	133.610	825.91	110.74	1.02	71,613	0.0715	1,350
	12.70	12,389	955	150.676	929.15	109.98	1.02	69,677	0.0700	1,760
PP356	3.40	3,768	290	58.272	327.74	124.47	1.12	95,484	0.0956	42
	3.58	3,962	305	61.186	345.77	124.47	1.12	95,484	0.0953	49
	3.81	4,213	324	65.348	367.07	124.46	1.12	94,839	0.0951	59
	3.96	4,374	337	67.846	380.18	124.21	1.12	94,839	0.0948	66
	4.17	4,600	354	71.176	399.84	124.21	1.12	94,839	0.0948	77
	4.37	4,820	371	74.505	417.87	124.21	1.12	94,193	0.0946	89
	4.55	5,013	386	77.419	434.26	124.21	1.12	94,193	0.0943	101
	4.78	5,265	405	81.165	455.56	123.95	1.12	94,193	0.0941	117
	5.16	5,678	436	86.992	489.97	123.95	1.12	93,548	0.0936	147
	5.33	5,871	451	89.906	506.36	123.95	1.12	93,548	0.0936	163
	5.56	6,116	470	93.652	527.66	123.70	1.12	92,903	0.0933	815
	5.84	6,420	494	98.231	552.24	123.70	1.12	92,903	0.0928	215
	6.35	6,968	536	106.139	598.13	123.44	1.12	92,258	0.0923	277

Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

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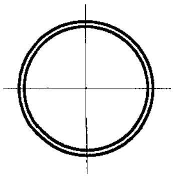

PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP356 (cont'd)	7.14	7,807	601	118.626	666.95	123.19	1.12	91,613	0.0916	395
	7.92	8,646	666	130.697	735.78	122.94	1.12	90,968	0.0906	542
	8.74	9,549	732	143.184	806.24	122.68	1.12	89,677	0.0898	691
	9.53	10,389	796	155.254	873.43	122.43	1.12	89,032	0.0890	835
	11.13	12,065	926	178.563	1,006.17	121.92	1.12	87,097	0.0873	1,130
	11.91	12,839	989	190.218	1,070.08	121.67	1.12	86,451	0.0865	1,280
	12.70	13,678	1,052	201.456	1,132.35	121.41	1.12	85,806	0.0855	1,460
PP406	3.40	4,310	331	87.409	430.98	142.49	1.28	125,161	1.2542	28
	3.58	4,529	348	91.987	452.28	142.49	1.28	125,161	0.1252	33
	3.81	4,820	371	97.814	480.14	142.24	1.28	125,161	0.1249	39
	3.96	5,007	385	101.560	499.81	142.24	1.28	124,516	0.1247	44
	4.17	5,265	405	106.555	524.39	142.24	1.28	124,516	0.1244	52
	4.37	5,516	424	111.550	548.97	142.24	1.28	124,516	0.1242	60
	4.55	5,742	441	115.712	570.27	141.99	1.28	123,871	0.1239	67
	4.78	6,026	463	121.540	598.13	141.99	1.28	123,871	0.1237	78
	5.16	6,517	500	130.697	644.01	141.99	1.28	123,226	0.1232	98
	5.56	7,033	539	140.686	693.17	141.73	1.28	122,580	0.1227	124
	5.84	7,355	565	147.346	725.95	141.73	1.28	122,580	0.1224	144
	6.35	8,000	614	159.833	786.58	141.48	1.28	121,935	0.1217	185
	7.14	8,968	688	178.563	878.35	141.22	1.28	120,645	0.1207	264
	7.92	9,936	763	196.877	970.11	140.97	1.28	120,000	0.1199	362
	8.74	10,905	839	216.024	1,061.88	140.72	1.28	118,709	0.1189	487
	9.53	11,873	913	233.922	1,152.01	140.46	1.28	118,064	0.1179	617
	11.13	13,807	1,062	270.134	1,328.99	139.70	1.28	116,129	0.1159	874
	11.91	14,775	1,135	287.616	1,414.20	139.45	1.28	114,838	0.1149	1,000
	12.70	15,679	1,208	304.681	1,499.42	139.19	1.28	114,193	0.1141	1,130

Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

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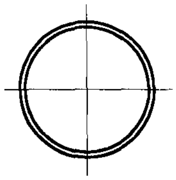

PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP457	3.58	5,104	392	131.113	573.55	160.27	1.44	159,355	0.1590	23
	4.37	6,213	478	159.417	696.45	160.02	1.44	158,064	0.1580	42
	4.78	6,775	522	173.569	760.36	160.02	1.44	157,419	0.1573	55
	5.16	7,291	563	186.888	817.71	159.77	1.44	156,774	0.1568	69
	5.56	7,871	607	201.456	879.99	159.77	1.44	156,129	0.1563	87
	5.84	8,259	637	211.029	922.59	159.51	1.44	156,129	0.1558	101
	6.35	8,968	692	228.511	999.61	159.51	1.44	155,484	0.1553	129
	7.14	10,065	776	255.566	1,117.60	159.26	1.44	154,193	0.1540	184
	7.92	11,163	860	282.205	1,235.58	158.75	1.44	152,903	0.1530	253
	8.74	12,323	947	309.676	1,353.57	158.50	1.44	151,613	0.1518	341
	9.53	13,420	1,030	335.899	1,468.28	158.24	1.44	150,967	0.1508	443
	10.31	14,452	1,113	361.705	1,581.35	157.99	1.44	149,677	0.1498	559
	11.13	15,615	1,199	387.928	1,704.25	157.73	1.44	148,387	0.1485	675
	11.91	16,646	1,281	413.318	1,802.58	157.48	1.44	147,742	0.1475	788
	12.70	17,743	1,364	437.043	1,917.29	157.23	1.44	146,451	0.1465	900
PP508	3.58	5,678	436	180.644	711.20	178.31	1.60	196,774	0.1969	17
	4.37	6,904	531	219.354	863.60	178.05	1.60	195,483	0.1957	30
	4.78	7,549	581	238.917	940.62	177.80	1.60	194,838	0.1952	40
	5.16	8,130	626	257.647	1,014.36	177.80	1.60	194,838	0.1947	50
	5.56	8,775	675	277.210	1,091.38	177.55	1.60	194,193	0.1939	63
	6.35	10,002	769	314.671	1,238.86	177.29	1.60	192,903	0.1926	94
	7.14	11,226	864	352.132	1,386.35	177.04	1.60	191,613	0.1914	134
	7.92	12,452	957	389.176	1,532.19	176.78	1.60	190,322	0.1901	184
	8.74	13,678	1,054	428.718	1,687.87	176.53	1.60	189,032	0.1889	247

Pile design data converted to SI units from US units published in 1985 version of this manual.

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Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

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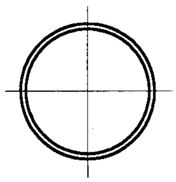


PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP508 (cont'd)	9.53	14,904	1,147	462.017	1,818.96	176.28	1.60	187,742	0.1879	321
	10.31	16,130	1,240	499.478	1,966.45	176.02	1.60	186,451	0.1866	409
	11.13	17,357	1,335	536.939	2,113.93	175.77	1.60	185,161	0.1854	515
	11.91	18,583	1,428	570.237	2,245.03	175.51	1.60	183,871	0.1841	618
	12.70	19,743	1,520	607.698	2,392.51	175.26	1.60	183,225	0.1829	719
PP559	4.37	7,613	585	292.611	1,047.13	196.09	1.76	237,419	0.2375	23
	4.78	8,323	639	318.833	1,142.18	195.83	1.76	236,774	0.2370	30
	5.56	9,678	743	370.030	1,324.07	195.58	1.76	235,483	0.2355	47
	6.35	11,034	847	420.394	1,504.33	195.33	1.76	234,193	0.2343	70
	7.14	12,389	951	470.342	1,687.87	195.07	1.76	232,903	0.2328	100
	7.92	13,744	1,055	520.289	1,868.13	194.82	1.76	231,612	0.2315	138
	8.74	15,099	1,161	570.237	2,048.38	194.56	1.76	230,322	0.2303	185
	9.53	16,454	1,264	620.185	2,212.25	194.31	1.76	229,032	0.2288	241
	10.31	17,743	1,366	670.133	2,392.51	194.06	1.76	227,741	0.2275	306
	11.13	19,162	1,472	715.918	2,572.77	193.55	1.76	225,806	0.2260	386
	11.91	20,454	1,574	765.866	2,736.64	193.29	1.76	224,516	0.2248	475
	12.70	21,809	1,675	811.651	2,900.51	193.04	1.76	223,225	0.2235	571
PP610	4.37	8,323	639	380.436	1,248.69	213.87	1.91	283,870	0.2834	18
	4.78	9,097	698	414.983	1,361.77	213.87	1.91	282,580	0.2834	23
	5.56	10,582	812	482.828	1,579.71	213.61	1.91	281,290	0.2809	36
	6.35	12,065	925	549.425	1,802.58	213.36	1.91	279,999	0.2809	54
	7.14	13,486	1,039	611.860	2,015.61	213.11	1.91	278,064	0.2784	77
	7.92	14,970	1,152	678.457	2,228.64	212.85	1.91	276,774	0.2759	106
	8.74	16,517	1,268	745.054	2,441.67	212.34	1.91	275,483	0.2759	142
	9.53	17,937	1,381	807.489	2,654.70	212.09	1.91	274,193	0.2734	185

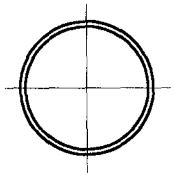
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP610 (cont'd)	10.31	19,421	1,493	869.924	2,867.74	211.84	1.91	272,258	0.2734	235
	11.13	20,904	1,608	936.521	3,080.77	211.58	1.91	270,967	0.2709	296
	11.91	22,388	1,720	998.955	3,277.41	211.33	1.91	269,677	0.2684	364
	12.70	23,809	1,831	1,061.390	3,474.06	211.07	1.91	267,741	0.2684	443
PP660	6.35	13,033	1,003	699.269	2,113.93	231.14	2.08	329,677	0.3286	43
	7.14	14,646	1,126	782.515	2,359.74	230.89	2.08	327,741	0.3286	61
	7.92	16,259	1,249	865.761	2,621.93	230.63	2.08	326,451	0.3261	83
	8.74	17,872	1,376	949.008	2,884.12	230.38	2.08	324,515	0.3236	112
	9.53	19,485	1,498	1,032.254	3,129.93	230.12	2.08	323,225	0.3236	145
	10.31	21,034	1,620	1,111.338	3,375.74	229.87	2.08	321,290	0.3211	184
	11.13	22,711	1,745	1,194.584	3,621.54	229.62	2.08	319,999	0.3211	232
	11.91	24,260	1,866	1,277.830	3,867.35	229.36	2.08	318,064	0.3186	286
	12.70	25,873	1,987	1,356.914	4,113.15	229.11	2.08	316,774	0.3161	347
	14.27	28,969	2,228	1,510.920	4,588.38	228.60	2.08	313,548	0.3135	495
	15.88	32,132	2,472	1,669.088	5,063.60	227.84	2.08	310,322	0.3110	656
	17.48	35,292	2,714	1,823.094	5,522.44	227.33	2.08	307,096	0.3060	814
	19.05	38,389	2,951	1,977.099	5,981.28	226.82	2.08	303,870	0.3035	970
PP711	6.35	14,065	1,081	874.086	2,458.06	249.17	2.23	383,225	0.3838	34
	7.14	15,807	1,214	978.144	2,753.03	248.92	2.23	381,290	0.3813	48
	7.92	17,486	1,346	1,082.202	3,047.99	248.67	2.23	379,999	0.3788	66
	8.74	19,291	1,483	1,190.422	3,342.96	248.41	2.23	378,064	0.3788	89
	9.53	20,969	1,615	1,294.480	3,637.93	248.16	2.23	376,128	0.3763	116
	10.31	22,711	1,746	1,394.375	3,916.51	247.90	2.23	374,838	0.3737	147
	11.13	24,453	1,881	1,498.433	4,211.48	247.65	2.23	372,902	0.3737	185
	11.91	26,195	2,012	1,598.329	4,506.44	247.40	2.23	370,967	0.3712	228

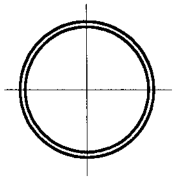
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP711 (cont'd)	12.70	27,874	2,143	1,698.224	4,785.02	246.89	2.23	369,677	0.3687	277
	14.27	31,229	2,403	1,898.015	5,342.18	246.38	2.23	365,806	0.3587	395
	15.88	34,713	2,667	2,097.806	5,899.34	245.87	2.23	362,580	0.3612	544
	17.48	38,068	2,929	2,293.435	6,440.12	245.36	2.23	359,354	0.3587	691
	19.05	41,423	3,185	2,480.739	6,980.89	244.86	2.23	356,128	0.3562	835
PP762	6.35	15,099	1,159	1,078.039	2,818.58	266.70	2.39	440,644	0.4415	28
	7.14	16,904	1,302	1,207.071	3,162.70	266.70	2.39	439,354	0.4390	39
	7.92	18,775	1,444	1,336.103	3,506.83	266.70	2.39	437,418	0.4365	54
	8.74	20,646	1,590	1,465.135	3,850.96	266.70	2.39	435,483	0.4365	72
	9.53	22,517	1,731	1,594.166	4,178.70	266.70	2.39	433,548	0.4340	94
	10.31	24,324	1,873	1,719.036	4,522.83	266.70	2.39	431,612	0.4314	120
	11.13	26,261	2,018	1,848.068	4,850.57	266.70	2.39	429,677	0.4289	150
	11.91	28,066	2,159	1,972.937	5,178.31	264.16	2.39	427,741	0.4289	185
	12.70	29,874	2,299	2,097.806	5,506.05	264.16	2.39	426,451	0.4264	225
	14.27	33,550	2,578	2,343.383	6,145.15	264.16	2.39	422,580	0.4214	321
	15.88	37,228	2,861	2,588.959	6,800.63	264.16	2.39	418,709	0.4189	443
	17.48	40,907	3,143	2,834.536	7,439.73	264.16	2.39	415,483	0.4164	584
	19.05	44,454	3,419	3,071.788	8,062.44	261.62	2.39	411,612	0.4114	719
PP813	6.35	16,065	1,237	1,306.967	3,211.86	284.48	2.55	502,580	0.5017	23
	7.14	18,067	1,389	1,465.135	3,605.15	284.488	2.55	500,644	0.5017	32
	7.92	20,067	1,541	1,623.303	3,998.44	284.48	2.55	498,709	0.4992	44
	8.74	22,067	1,697	1,785.633	4,391.73	284.48	2.55	496,773	0.4967	60
	9.53	24,067	1,848	1,939.638	4,768.64	284.48	2.55	494,838	0.4942	77
	10.31	26,002	1,999	2,093.644	5,145.54	284.48	2.55	492,902	0.4916	98
	11.13	28,003	2,155	2,251.812	5,538.83	284.48	2.55	490,967	0.4916	124
	11.91	30,003	2,305	2,401.655	5,915.73	281.94	2.55	489,031	0.4891	152

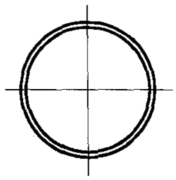
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP813 (cont'd)	12.70	31,937	2,455	2,555.661	6,292.63	281.94	2.55	487,096	0.4866	185
	14.27	35,810	2,754	2,855.348	7,030.05	281.94	2.55	483,225	0.4841	264
	15.88	39,744	3,056	3,155.034	7,767.47	281.94	2.55	479,354	0.4791	364
	17.48	43,680	3,358	3,454.721	8,504.89	281.94	2.55	475,483	0.4741	487
	19.05	47,488	3,653	3,741.921	9,209.53	281.94	2.55	471,612	0.4716	617
PP864	6.35	17,099	1,315	1,569.192	3,637.93	302.26	2.71	568,386	0.5694	19
	7.14	19,228	1,477	1,760.659	4,080.38	302.26	2.71	566,450	0.5669	27
	7.92	21,293	1,638	1,947.963	4,522.83	302.26	2.71	564,515	0.5644	37
	8.74	23,485	1,804	2,143.592	4,965.28	302.26	2.71	562,580	0.5619	50
	9.53	25,551	1,965	2,330.896	5,391.34	302.26	2.71	559,999	0.5594	64
	10.31	27,615	2,126	2,518.200	5,833.79	302.26	2.71	558,063	0.5569	82
	11.13	29,808	2,291	2,705.504	6,276.25	302.26	2.71	556,128	0.5569	103
	11.91	31,873	2,451	2,888.646	6,702.31	302.26	2.71	554,192	0.5544	127
	12.70	33,938	2,611	3,071.788	7,111.99	299.72	2.71	551,612	0.5518	154
	14.27	38,068	2,929	3,433.909	7,964.11	299.72	2.71	547,741	0.5468	219
	15.88	42,262	3,251	3,800.193	8,799.85	299.72	2.71	543,225	0.5443	303
	17.48	46,454	3,572	4,158.152	9,635.59	299.72	2.71	539,354	0.5393	405
	19.05	50,519	3,887	4,495.299	10,438.56	299.72	2.71	535,483	0.5343	527
	22.23	58,779	4,517	5,202.893	12,044.49	297.18	2.71	527,096	0.5268	767
	25.40	67,102	5,143	5,868.863	13,617.65	297.18	2.71	518,709	0.5192	1,010
PP914	6.35	18,130	1,393	1,868.879	4,080.38	320.04	2.87	638,708	0.6396	16
	7.14	20,325	1,564	2,093.644	4,571.99	320.04	2.87	636,128	0.6371	23
	7.92	22,582	1,735	2,318.409	5,063.60	320.04	2.87	634,192	0.6346	31
	8.74	24,840	1,912	2,547.336	5,571.60	320.04	2.87	631,612	0.6321	42

Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

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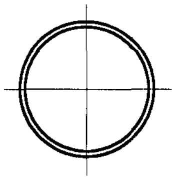

PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP914 (cont'd)	9.53	27,098	2,082	2,772.101	6,063.21	320.04	2.87	629,676	0.6296	54
	10.31	29,292	2,252	2,992.704	6,538.44	320.04	2.87	627,096	0.6271	69
	11.13	31,550	2,428	3,221.631	7,046.44	320.04	2.87	625,160	0.6246	87
	11.91	33,808	2,597	3,438.072	7,521.66	320.04	2.87	623,225	0.6221	107
	12.70	36,002	2,766	3,658.674	7,996.89	320.04	2.87	620,644	0.6221	129
	14.27	40,390	3,104	4,087.393	8,947.34	317.50	2.87	616,128	0.6171	184
	15.88	44,841	3,446	4,536.923	9,897.79	317.50	2.87	611,612	0.6120	254
	17.48	49,230	3,786	4,953.154	10,831.85	317.50	2.87	607,741	0.6070	341
	19.05	53,616	4,120	5,369.385	11,749.52	317.50	2.87	603,225	0.6020	443
	22.23	62,326	4,790	6,201.848	13,568.49	314.96	2.87	594,192	0.5945	674
	25.40	70,972	5,455	7,034.311	15,338.29	314.96	2.87	585,805	0.5870	900
	31.75	87,747	6,770	8,574.367	18,845.12	312.42	2.87	568,386	0.5694	1,380
PP965	6.35	19,099	1,471	2,197.702	4,555.60	337.82	3.03	709,676	0.7124	14
	7.14	21,485	1,652	2,464.090	5,112.76	337.82	3.03	709,676	0.7099	19
	7.92	23,809	1,833	2,730.478	5,653.54	337.82	3.03	709,676	0.7074	26
	8.74	26,261	2,019	3,001.029	6,227.08	337.82	3.03	703,224	0.7049	35
	9.53	28,582	2,199	3,263.254	6,767.86	337.82	3.03	703,224	0.7023	46
	10.31	30,971	2,379	3,525.480	7,308.63	337.82	3.03	703,224	0.6998	59
	11.13	33,358	2,564	3,796.031	7,865.79	337.82	3.03	696,773	0.6973	74
	11.91	35,680	2,743	4,054.094	8,406.56	337.82	3.03	696,773	0.6973	90
	12.70	38,002	2,922	4,328.807	8,930.95	337.82	3.03	696,773	0.6923	110
	14.27	42,649	3,279	4,828.285	9,996.11	335.28	3.03	690,321	0.6898	156
	15.88	47,359	3,641	5,327.762	11,061.27	335.28	3.03	683,870	0.6848	216
	17.48	52,003	4,001	5,827.240	12,110.04	335.28	3.03	677,418	0.6798	289

Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

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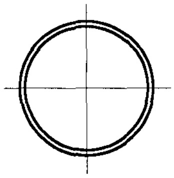


PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP965 (cont'd)	19.05	56,649	4,354	6,326.718	13,142.43	335.28	3.03	677,418	0.6748	376
	22.23	65,810	5,063	7,325.673	15,174.42	332.74	3.03	664,515	0.6647	590
	25.40	74,843	5,767	8,283.005	17,206.42	332.74	3.03	658,063	0.6572	805
	31.75	92,909	7,160	10,156.047	20,975.44	330.20	3.03	638,708	0.6396	1,230
	38.10	110,974	8,533	11,945.842	24,744.47	327.66	3.03	620,644	0.6221	1,780
PP1016	7.92	25,098	1,930	3,188.333	6,276.25	355.60	3.20	787,095	0.7851	23
	8.74	27,679	2,126	3,508.831	6,898.95	355.60	3.20	780,644	0.7826	30
	9.53	30,131	2,316	3,812.680	7,505.28	355.60	3.20	780,644	0.7801	39
	10.31	32,583	2,505	4,120.691	8,111.60	355.60	3.20	780,644	0.7776	50
	11.13	35,099	2,701	4,453.676	8,734.31	355.60	3.20	774,192	0.7751	63
	11.91	37,551	2,890	4,745.038	9,324.24	355.60	3.20	774,192	0.7726	77
	12.70	40,002	3,078	5,036.400	9,914.17	355.60	3.20	767,740	0.7701	94
	14.27	44,906	3,454	5,619.124	11,094.04	353.06	3.20	767,740	0.7651	134
	15.88	49,874	3,836	6,243.471	12,273.91	353.06	3.20	761,289	0.7600	185
	17.48	54,842	4,215	6,826.195	13,453.78	353.06	3.20	754,837	0.7550	247
	19.05	59,681	4,588	7,408.919	14,600.87	353.06	3.20	748,386	0.7500	321
	22.23	69,682	5,336	8,574.367	16,878.68	350.52	3.20	741,934	0.7425	514
	25.40	79,360	6,078	9,698.192	19,172.86	350.52	3.20	729,031	0.7324	719
	31.75	98,070	7,549	11,904.219	23,433.50	347.98	3.20	709,676	0.7124	1,130
	38.10	116,781	9,001	14,026.999	27,530.27	345.44	3.20	696,773	0.6923	1,620
	44.45	135,492	10,433	16,024.910	31,627.03	342.90	3.20	677,418	0.6748	2,140
PP1067	7.92	26,389	2,027	3,696.135	6,931.73	373.38	3.35	864,514	0.8679	20
	8.74	29,034	2,233	4,066.581	7,619.98	373.38	3.35	864,514	0.8654	26
	9.53	31,615	2,433	4,412.053	8,291.85	373.38	3.35	864,514	0.8629	34
	10.31	34,260	2,632	4,786.661	8,947.34	373.38	3.35	864,514	0.8604	43

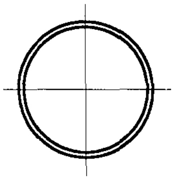
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP1067 (cont'd)	11.13	36,905	2,837	5,161.270	9,635.59	373.38	3.35	858,063	0.8579	54
	11.91	39,486	3,036	5,494.255	10,291.08	373.38	3.35	851,611	0.8554	67
	12.70	42,067	3,234	5,827.240	10,946.56	373.38	3.35	851,611	0.8528	81
	14.27	47,229	3,630	6,534.833	12,257.52	373.38	3.35	845,160	0.8478	116
	15.88	52,390	4,030	7,242.427	13,568.49	370.84	3.35	838,708	0.8403	159
	17.48	57,616	4,430	7,950.020	14,863.07	370.84	3.35	838,708	0.8353	213
	19.05	62,713	4,822	8,615.991	16,141.26	370.84	3.35	832,256	0.8303	277
	22.23	72,908	5,608	9,947.931	18,681.25	368.30	3.35	819,353	0.8202	443
	25.40	83,231	6,390	11,279.872	21,139.31	368.30	3.35	812,902	0.8102	641
	31.75	103,232	7,939	13,818.883	25,891.56	365.76	3.35	793,547	0.7901	1,030
	38.10	123,233	9,468	16,316.272	30,643.81	363.22	3.35	767,740	0.7701	1,460
	44.45	142,589	10,978	18,688.791	35,068.32	360.68	3.35	748,386	0.7500	1,970
	50.80	161,945	12,468	20,978.064	39,328.95	360.68	3.35	729,031	0.7324	2,470
PP1118	8.74	30,453	2,341	4,661.792	8,373.79	391.16	3.51	948,385	0.9507	23
	9.53	33,163	2,550	5,078.023	9,111.21	391.16	3.51	948,385	0.9482	30
	10.31	35,873	2,759	5,494.255	9,832.24	391.16	3.51	941,934	0.9457	38
	11.13	38,647	2,974	5,910.486	10,586.04	391.16	3.51	941,934	0.9432	47
	11.91	41,357	3,182	6,326.718	11,323.46	391.16	3.51	941,934	0.9406	58
	12.70	44,067	3,390	6,742.949	12,044.49	391.16	3.51	935,482	0.9381	70
	15.88	54,971	4,225	8,324.629	14,928.62	388.62	3.51	929,030	0.9256	138
	19.05	65,810	5,056	9,906.308	17,698.03	388.62	3.51	916,127	0.9156	241
	22.23	76,779	5,881	11,487.987	20,483.83	388.62	3.51	903,224	0.9055	384
	25.40	87,102	6,702	12,986.420	23,269.63	386.08	3.51	896,772	0.8930	571

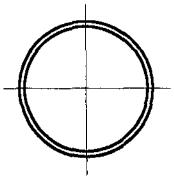
Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.



PIPE PILES

Approximate Pile Dimensions and Design Properties

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Meter	Section Properties			Area of Exterior Surface	Inside Cross Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
mm	mm	mm^2	N	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	m^2/m	mm^2	m^3/m	*
PP1118 (cont'd)	31.75	108,394	8,328	15,983.287	28,513.49	383.54	3.51	870,966	0.8729	941
	38.10	129,040	9,936	18,855.284	33,757.35	381.00	3.51	851,611	0.8528	1,300
	44.45	149,686	11,524	21,602.411	38,673.47	381.00	3.51	832,256	0.8303	1,810
	50.80	170,333	13,092	24,266.292	43,425.72	378.46	3.51	812,902	0.8102	2,290
	57.15	190,334	14,641	26,846.927	48,014.10	375.92	3.51	793,547	0.7901	2,770
PP1219	8.74	33,228	2,555	6,076.979	9,979.72	426.72	3.84	1,135,482	1.1338	18
	9.53	36,196	2,784	6,618.080	10,864.62	426.72	3.84	1,129,030	1.1313	23
	10.31	39,164	3,012	7,159.181	11,733.14	426.72	3.84	1,129,030	1.1288	29
	11.13	42,196	3,247	7,700.281	12,634.43	426.72	3.84	1,122,578	1.1263	36
	11.91	45,164	3,474	8,241.382	13,502.94	426.72	3.84	1,122,578	1.1212	45
	12.70	48,132	3,702	8,740.860	14,371.46	426.72	3.84	1,116,127	1.1187	54
	15.88	60,004	4,615	10,863.640	17,861.90	426.72	3.84	1,109,675	1.1087	106
	19.05	71,617	5,523	12,944.797	21,139.31	424.18	3.84	1,096,772	1.0962	185
	22.23	83,876	6,427	14,984.331	24,580.60	424.18	3.84	1,083,869	1.0836	295
	25.40	95,490	7,325	16,982.242	27,858.01	421.64	3.84	1,070,966	1.0711	443
	31.75	118,717	9,108	20,894.818	34,248.96	419.10	3.84	1,051,611	1.0485	787
	38.10	141,299	10,871	24,682.524	40,476.05	416.56	3.84	1,025,804	1.0259	1,130
	44.45	163,881	12,614	28,345.360	46,539.26	416.56	3.84	1,006,450	1.0034	1,530
	50.80	186,463	14,339	31,883.327	52,274.73	414.02	3.84	980,643	0.9808	1,970
	57.15	208,400	16,043	35,296.425	57,846.34	411.48	3.84	961,288	0.9582	2,410
	63.50	230,336	17,729	38,626.276	63,254.07	408.94	3.84	935,482	0.9381	2,850

Pile design data converted to SI units from US units published in 1985 version of this manual.

Note: Designer must confirm section properties and local availability of selected pile section.

Material Specifications - ASTM A252

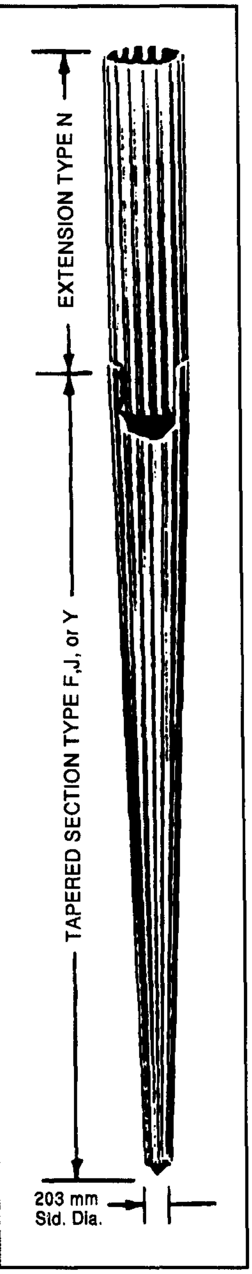
Example of suggested method of designation: PP219 x 2.77

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ratio and is for general guidance only. The higher the number, the greater is the resistance to collapse.

MONOTUBE PILES

Standard Monotube Weights and Volumes

TYPE	SIZE POINT DIAMETER x BUTT DIAMETER x LENGTH	Weight (N) per m 9 GA.	7 GA.	5 GA.	3 GA.	EST. CONC. VOL. m^3
F Taper 3.6 mm per Meter	216 mm x 305 mm x 7.62 m	248	292	350	409	0.329
	203 mm x 305 mm x 9.14 m	233	292	336	394	0.420
	216 mm x 356 mm x 12.19 m	277	321	379	452	0.726
	203 mm x 406 mm x 18.29 m	292	350	409	482	1.284
	203 mm x 457 mm x 22.86 m	-	379	452	511	1.979
J Taper 6.4 mm per Meter	203 mm x 305 mm x 5.18 m	248	292	336	394	0.244
	203 mm x 356 mm x 7.62 m	263	321	379	438	0.443
	203 mm x 406 mm x 10.06 m	292	350	409	467	0.726
	203 mm x 457 mm x 12.19 m	-	379	438	511	1.047
Y Taper 10.2 mm per Meter	203 mm x 305 mm x 3.05 m	248	292	350	409	0.138
	203 mm x 356 mm x 4.57 m	277	321	379	438	0.260
	203 mm x 406 mm x 6.10 m	292	350	409	482	0.428
	203 mm x 457 mm x 7.62 m	-	379	452	511	0.657



Extensions (Overall Length 0.305 m Greater than indicated)

TYPE	DIAMETER + LENGTH	9 GA.	7 GA.	5 GA.	3 GA.	m^3 /m
N 12	305 mm x 305 mm x 6.10 / 12.19 m	292	350	409	482	0.065
N 14	356 mm x 356 mm x 6.10 m / 12.19 m	350	423	496	598	0.088
N 16	406 mm x 406 mm x 6.10 m / 12.19 m	409	482	569	671	0.113
N 18	457 mm x 457 mm x 6.10 m / 12.19 m	-	555	642	759	0.145

Note: Designer must confirm section properties of selected pile section.

Pile design data converted to SI units from US units published in Monotube Pile Corporation Catalog 592.

MONOTUBE PILES
Physical Properties

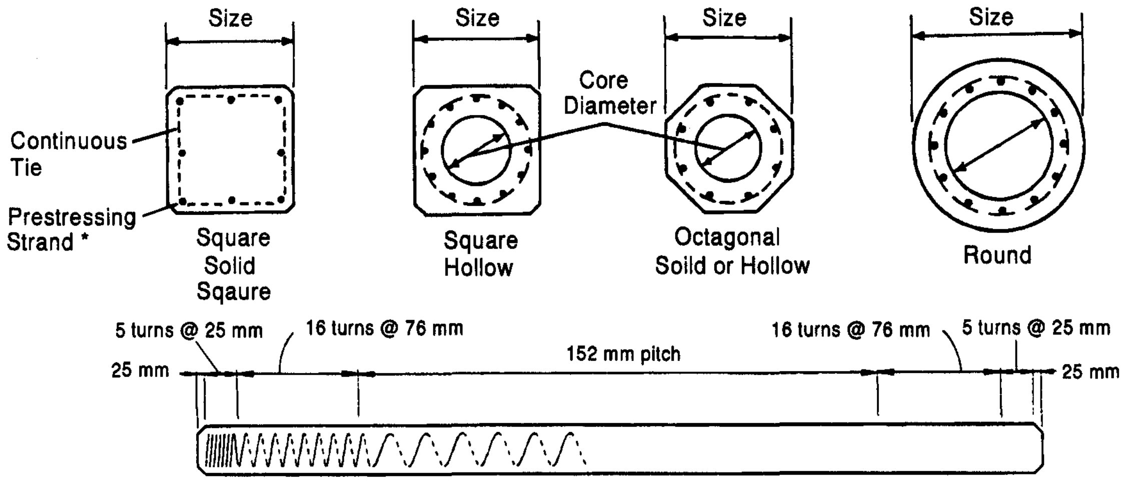
	POINTS		BUTTS OF PILE SECTIONS							
	203 mm	216 mm	305 mm				356 mm			
STEEL THICKNESS	A mm^2	A mm^2	A mm^2	I mm^4 x 10^6	S mm^3 x 10^3	r mm	A mm^2	I mm^4 x 10^6	S mm^3 x 10^3	r mm
9 GAUGE 3.797 mm	2,342	2,535	3,748	42.456	267.109	106	4,355	66.181	360.515	123
7 GAUGE 4.554 mm	2,839	3,077	4,497	50.780	319.548	106	5,252	80.749	437.535	124
5 GAUGE 5.314 mm	3,348	3,619	5,277	60.354	376.902	107	6,129	94.485	507.999	124
3 GAUGE 6.073 mm	3,787	4,245	5,781	61.602	396.567	103	6,839	99.479	550.605	121
CONCRETE AREA mm^2	27,290	30,518	65,161				87,742			

	POINTS		BUTTS OF PILE SECTIONS							
	203 mm	216 mm	406 mm				457 mm			
STEEL THICKNESS	A mm^2	A mm^2	A mm^2	I mm^4 x 10^6	S mm^3 x 10^3	r mm	A mm^2	I mm^4 x 10^6	S mm^3 x 10^3	r mm
9 GAUGE 3.797 mm	2,342	2,535	4,929	96.566	463.754	140	-	-	-	-
7 GAUGE 4.554 mm	2,839	3,077	5,923	115.712	555.521	140	6,710	168.157	712.837	158
5 GAUGE 5.314 mm	3,348	3,619	6,968	136.940	555.521	140	7,871	198.959	839.018	159
3 GAUGE 6.073 mm	3,787	4,245	7,742	144.849	706.282	137	8,774	209.781	907.843	155
CONCRETE AREA mm^2	27,290	30,518	113,548				144,516			

Note: Designer must confirm section properties of selected pile section.

Pile design data converted to SI units from US units published in Monotube Pile Corporation Catalog 592.

PRECAST/PRESTRESSED CONCRETE PILES



Typical Elevation

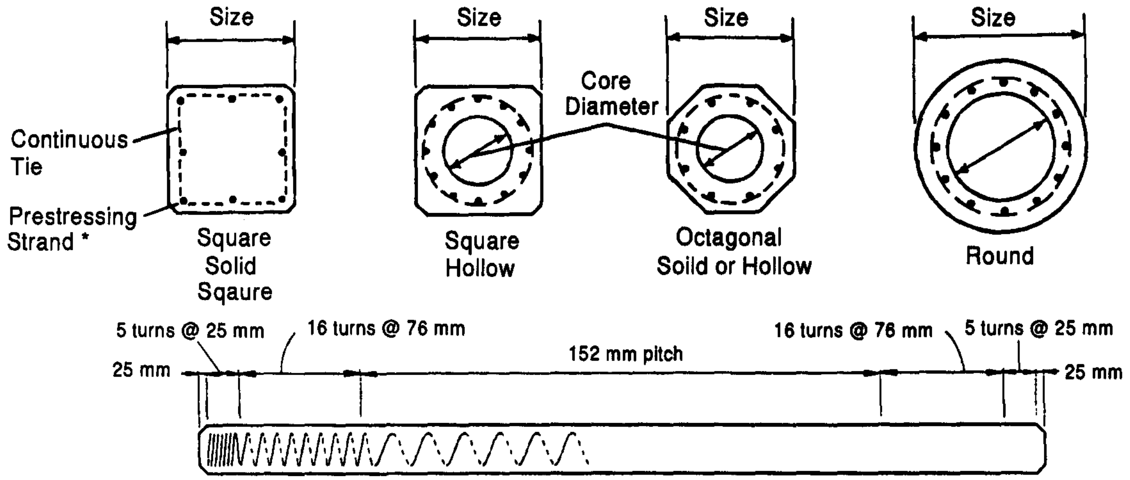
* Strand pattern may be circular or square

Size mm	Core Diameter mm	Area mm^2	Weight N/m	Moment of Inertia $mm^4 \times 10^6$	Section Modulus $mm^3 \times 10^3$	Radius of Gyration mm	Perimeter m
		Section Properties					
Square Piles							
254	Solid	64,516	1,518	346.721	2,736.640	73.4	1.015
305	Solid	92,903	2,189	719.248	4,719.474	87.9	1.219
356	Solid	126,451	2,977	1,332.357	7,488.888	102.6	1.423
406	Solid	165,161	3,896	2,273.040	11,192.365	117.3	1.625
457	Solid	209,032	4,932	3,641.193	15,928.226	132.1	1.829
508	Solid	258,064	6,085	5,549.614	21,843.956	146.6	2.033
508	279 mm	196,774	4,641	5,250.759	20,680.475	163.3	2.033
610	Solid	371,612	8,756	11,507.966	37,755.795	176.0	2.438
610	305 mm	298,709	7,034	11,084.243	36,362.895	192.5	2.438
610	356 mm	272,258	6,406	10,722.954	35,183.026	198.4	2.438
610	381 mm	257,419	6,056	10,473.631	34,363.673	201.7	2.438
762	457 mm	416,773	9,807	25,950.781	68,121.025	249.4	3.048
914	457 mm	672,257	15,834	56,114.240	122,739.109	289.1	3.658

Note: Designer must confirm section properties for a selected pile. Form dimensions may vary with producers, with corresponding variations in section properties.

Data converted to SI units from US unit properties in PCI (1993), Precast/Prestressed Concrete Institute Journal, Volume 38, No. 2, March-April, 1993.

PRECAST/PRESTRESSED CONCRETE PILES



Typical Elevation

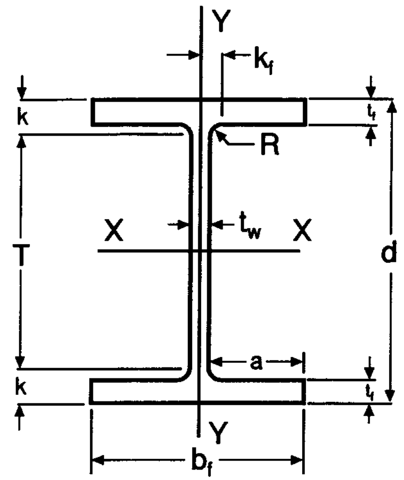
* Strand pattern may be circular or square

Size mm	Core Diameter mm	Section Properties					
		Area mm^2	Weight N/m	Moment of Inertia $mm^4 \times 10^6$	Section Modulus $mm^3 \times 10^3$	Radius of Gyration mm	Perimeter m
Octagonal Piles							
254	Solid	53,548	1,240	231.008	1,818.964	65.8	0.841
305	Solid	76,774	1,824	472.006	3,097.155	78.5	1.009
356	Solid	104,516	2,466	876.167	4,932.506	91.4	1.180
406	Solid	136,774	3,210	1,495.103	7,357.792	104.6	1.347
457	Solid	172,903	4,086	2,374.600	10,471.334	117.1	1.515
508	Solid	213,548	5,035	3,650.350	14,371.455	130.8	1.682
508	279 mm	152,258	3,575	3,350.663	13,191.587	148.3	1.682
559	Solid	258,709	6,129	5,343.163	19,123.704	143.8	1.853
559	330 mm	172,903	4,086	4,761.688	17,042.547	165.9	1.853
610	Solid	307,741	7,224	7,567.087	24,826.402	156.7	2.021
610	381 mm	193,548	4,597	6,533.168	21,434.280	183.6	2.021
Round Piles							
914	660 mm	314,193	7,399	24,976.799	54,634.471	281.9	2.874
1,067	813 mm	374,838	8,829	42,153.005	79,034.810	335.3	3.353
1,219	965 mm	435,483	10,259	65,856.969	108,023.526	388.9	3.831
1,372	1118 mm	496,773	11,704	97,137.176	141,633.394	442.2	4.310
1,676	1372 mm	729,676	17,191	213,954.191	255,261.296	541.5	5.267

Note: Designer must confirm section properties for a selected pile. Form dimensions may vary with producers, with corresponding variations in section properties.

Data converted to SI units from US unit properties in PCI (1993), Precast/Prestressed Concrete Institute Journal, Volume 38, No. 2, March-April, 1993.

H-PILES



Section Designation	Area A	Depth d	Web Thickness t_w	Flange		Distance				Fillet Radius R	Elastic Properies					
				Width b_f	Thickness t_f						X-X			Y-Y		
						T	k	k_f	a		I	S	r	I	S	r
mm x kg/m	mm^2	mm	mm	mm	mm	mm	mm	mm	mm	mm	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm	$mm^4 \times 10^6$	$mm^3 \times 10^3$	mm
HP360 x 174	22,200	361	20.4	378	20	277	42	30.2	179	20	511	2,830	152	183	968	91
HP360 x 152	19,400	356	17.9	376	18	277	40	29.0	179	20	442	2,480	151	158	840	90
HP360 x 132	16,900	351	15.6	373	16	277	37	27.8	179	20	378	2,150	150	135	724	89
HP360 x 108	13,800	346	12.8	370	13	277	34	26.4	179	20	306	1,770	148	108	584	88
HP310 x 125	15,800	312	17.4	312	17	244	34	23.7	147	15	270	1,730	131	88	565	75
HP310 x 110	14,000	308	15.4	310	15	244	32	22.7	147	15	236	1,530	130	77	497	74
HP310 x 93	11,800	303	13.1	308	13	244	30	21.6	148	15	196	1,290	129	64	414	74
HP310 x 79	9,970	299	11.0	306	11	244	28	20.5	148	15	162	1,080	127	53	343	73
HP250 x 85	10,800	254	14.4	260	14	196	29	20.2	123	13	123	969	107	42	325	63
HP250 x 62	7,980	246	10.5	256	11	96	25	18.3	123	13	88	711	105	30	234	61
HP200 x 53	6,810	204	11.3	207	11	158	23	15.7	98	10	50	487	86	17	161	50

Note: Designer must confirm section properties for a selected pile.

Data obtained from FHWA Geotechnical Metrication Guidelines (1995) FHWA-SA-95-035.

BITUMEN COATING FOR CONCRETE PILES

(This is a generic specification that should be modified to meet the specific needs of a given project.)

Description. This work shall consist of furnishing and applying bituminous coating and primer to prestressed concrete pile surfaces as required in the plans and as specified herein.

Materials

A. Bituminous Coating. Bituminous coating shall be an asphalt type bitumen conforming to ASTM D946, with a minimum penetration grade 50 at the time of pile driving. Bituminous coating shall be applied uniformly over an asphalt primer. Grade 40-50 or lower grades shall not be used.

B. Primer. Primer shall conform to the requirements of ASTM D41.

Construction Requirements. All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather, nor when the temperature is below 18 degrees Celsius.

The primer shall be applied to the surfaces and allowed to completely dry before the bituminous coating is applied. Primer shall be applied uniformly at the quantity of one liter per 2.43 square meters.

Bitumen shall be applied uniformly at a temperature of not less than 149 degrees Celsius, nor more than 177 degrees Celsius, and shall be applied either by mopping, brushing, or spraying at the project site. All holes or depressions in the concrete surface shall be completely filled with bitumen. The bituminous coating shall be applied to a minimum dry thickness of 3.2 mm, but in no case shall the quantity of application be less than 3.29 liters per square meters.

Bitumen coated piles shall be stored before driving and protected from sunlight and heat. Pile coatings shall not be exposed to damage during storage, hauling or handling. The Contractor shall take appropriate measures to preserve and maintain the bitumen coating. At the time of pile driving, the bitumen coating shall have a minimum dry thickness of 3.2 mm and a minimum penetration value of 50. If necessary, the Contractor shall recoat the piles, at his expense, to comply with these requirements.

Method of Measurement. Bitumen coating will be measured by the square meter of coating in place on concrete pile surfaces. No separate payment will be made for primer.

Basis of Payment. The accepted quantities of bitumen coating will be paid for at the contract unit price per square meter, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the Engineer.

Payment will be made under:

Pay Item	Pay Unit
Bitumen Coating	Square Meters.

BITUMEN COATING FOR STEEL PILES

(This is a generic specification that should be modified to meet the specific needs of a given project.)

Description. This work shall consist of furnishing and applying bituminous coating and primer to steel pile surfaces as required in the plans and as specified herein.

Materials

A. Bituminous Coating. Canal Liner Bitumen (ASTM D-2521) shall be used for the bitumen coating and shall have a softening point of 88 to 93 degrees Celsius, a penetration of 56 to 61 at 25 degrees Celsius, and a ductility at 25 degrees Celsius, in excess of 35.0 mm.

B. Primer. Primer shall conform to the requirements of AASHTO M116.

Construction Requirements. All surfaces to be coated with bitumen shall be dry and thoroughly cleaned of dust and loose materials. No primer or bitumen shall be applied in wet weather, nor when the temperature is below 18 degrees Celsius.

Application of the prime coat shall be with a brush or other approved means and in a manner to thoroughly coat the surface of the piling with a continuous film of primer. The purpose of the primer is to provide a suitable bond of the bitumen coating to the pile. The primer shall set thoroughly before the bitumen coating is applied.

The bitumen should be heated to 149 degrees Celsius, and applied at a temperature between 93 and 149 degrees Celsius, by one or more mop coats, or other approved means, to apply an average coating depth of 9.5 mm. Whitewashing of the coating may be required, as deemed necessary by the engineer, to prevent running and sagging of the asphalt coating prior to driving, during hot weather.

Bitumen coated piles shall be stored immediately after the coating is applied for protection from sunlight and heat. Pile coatings shall not be exposed to damage or contamination during storage, hauling, or handling. Once the bitumen coating has been applied, the contractor will not be allowed to drag the piles on the ground or to use cable wraps around the pile during handling. Pad eyes, or other suitable devices, shall be attached to the pile to be used for lifting and handling. If necessary, the contractor shall recoat the piles, at his expense to comply with these requirements.

A nominal length of pile shall be left uncoated where field splices will be required. After completing the field splice, the splice area shall be brush or map coated with at least one coat of bitumen.

Method of Measurement. Bitumen coating will be measured by the linear meter of coating in place on the pile surfaces. No separate payment will be made for primer or coating of the splice areas.

Basis of Payment. The accepted quantities of bitumen coating will be paid for at the contract unit price per linear meter, which price shall be full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in applying the bituminous coating and primer, as shown in the plans, and as specified in these specifications, and as directed by the Engineer.

Payment will be made under:

Pay Item	Pay-Unit
Bitumen Coating	Meter.

APPENDIX D

Pile Hammer Information

Note: GRLWEAP hammer ID numbers correspond to those contained in Version 1.996-2 of the GRLWEAP program.

TABLE D-1 DIESEL HAMMER LISTING
(sorted by Maximum Energy)

GRLWEAP ID	Hammer Mfgr	Hammer Name E	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type T
81	LINKBELT	LB 180	10.98	7.70	1.43	CED
120	ICE	180	11.03	7.70	1.43	CED
1	DELMAG	D 5	11.16	4.89	2.28	OED
36	DELMAG	D 6-32	14.24	5.88	2.42	OED
82	LINKBELT	LB 312	20.37	17.18	1.19	CED
146	MKT	DE 10	20.75	7.57	2.74	OED
147	MKT	DE 20	21.70	8.90	2.44	OED
2	DELMAG	D 8-22	23.87	7.83	3.05	OED
402	BERMINGH	B200	24.41	8.90	2.74	OED
83	LINKBELT	LB 440	24.69	17.80	1.39	CED
122	ICE	440	25.17	17.80	1.41	CED
141	MKT 20	DE333020	27.13	8.90	3.05	OED
151	MKT	DA 35B	28.48	12.46	2.29	CED
148	MKT	DE 30	30.38	12.46	2.44	OED
41	FEC	FEC 1200	30.51	12.24	2.49	OED
127	ICE	30-S	30.52	13.35	2.29	OED
401	BERMINGH	B23	31.18	12.46	2.50	CED
414	BERMINGH	B23 5	31.18	12.46	2.50	CED
121	ICE	422	31.36	17.80	1.76	CED
3	DELMAG	D 12	32.00	12.24	2.62	OED
149	MKT	DA35B SA	32.28	12.46	2.59	OED
150	MKT	DE 30B	32.28	12.46	2.59	OED
61	MITSUB.	M 14	34.24	13.22	2.59	OED
350	HERA	1250	34.38	12.50	2.75	OED
101	KOBE	K 13	34.49	12.77	2.70	OED
84	LINKBELT	LB 520	35.69	22.56	1.58	CED
42	FEC	FEC 1500	36.75	14.68	2.50	OED
201	VULCANI	VUL V12	36.77	12.26	3.00	OED
142	MKT 30	DE333020	37.98	12.46	3.05	OED
62	MITSUB.	MH 15	38.16	14.73	2.59	OED
4	DELMAG	D 15	38.40	14.68	2.62	OED
403	BERMINGH	B225	39.67	13.35	2.97	OED
123	ICE	520	41.19	22.56	1.83	CED
351	HERA	1500	41.25	15.00	2.75	OED
152	MKT	DA 45	41.67	17.80	2.34	CED
37	DELMAG	D 12-32	42.50	12.55	3.39	OED
153	MKT	DE 40	43.40	17.80	2.44	OED
143	MKT 33	DE333020	44.76	14.68	3.05	OED
415	BERMINGH	B250 5	48.02	13.35	3.60	OED
161	MKT	DA 55B	51.81	22.25	2.33	CED
202	VULCAN	VUL V18	52.97	17.66	3.00	OED

TABLE D-1 DIESEL HAMMER LISTING
(sorted by Maximum Energy)

GRLWEAP ID	Hammer Mfgr	Hammer Name E	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type T
5	DELMAG	D 16-32	53.23	15.66	3.40	OED
128	ICE	40-S	54.25	17.80	3.05	OED
144	MKT 40	DE333020	54.25	17.80	3.05	OED
160	MKT	DA55B SA	54.25	22.25	2.44	OED
404	BERMINGH	B300	54.68	16.69	3.28	OED
410	BERMINGH	B300 M	54.68	16.69	3.28	OED
6	DELMAG	D 22	55.08	21.85	2.52	OED
124	ICE	640	55.10	26.70	2.06	CED
129	ICE	42-S	56.97	18.19	3.13	OED
38	DELMAG	D 19-32	57.51	17.80	3.23	OED
159	MKT	DE 50B	57.65	22.25	2.59	OED
63	MITSUB.	M 23	58.34	22.52	2.59	OED
412	BERMINGH	B400 4.8	58.59	21.36	2.74	OED
413	BERMINGH	B400 5.0	61.04	22.25	2.74	OED
103	KOBE	K22-Est	61.51	21.58	2.85	OED
64	MITSUB.	MH 25	63.53	24.52	2.59	OED
416	BERMINGH	B350 5	64.02	17.80	3.60	OED
7	DELMAG	D 22-02	65.78	21.58	3.05	OED
8	DELMAG	D 22-13	65.78	21.58	3.05	OED
43	FEC	FEC 2500	67.81	24.47	2.77	OED
163	MKT 50	DE70/50B	67.82	22.25	3.05	OED
352	HERA	2500	68.75	25.00	2.75	OED
9	DELMAG	D 22-23	69.53	21.58	3.22	OED
104	KOBE	K 25	69.88	24.52	2.85	OED
125	ICE	660	70.03	33.69	2.08	CED
85	LINKBELT	LB 660	70.03	33.69	2.08	CED
405	BERMINGH	B400	72.90	22.25	3.28	OED
411	BERMINGH	B400 M	72.90	22.25	3.28	OED
44	FEC	FEC 2800	75.95	27.41	2.77	OED
353	HERA	2800	77.00	28.00	2.75	OED
203	VULCAN	VUL V25	78.51	24.53	3.20	OED
417	BERMINGH	B400 5	80.03	22.25	3.60	OED
162	MKT	DE 70B	80.70	31.15	2.59	OED
11	DELMAG	D 30	80.84	29.37	2.75	OED
10	DELMAG	D 25-32	83.40	24.52	3.40	OED
65	MITSUB.	M 33	83.70	32.31	2.59	OED
45	FEC	FEC 3000	85.49	29.37	2.91	OED
66	MITSUB.	MH 35	89.00	34.35	2.59	OED
12	DELMAG	D 30-02	89.52	29.37	3.05	OED
13	DELMAG	D 30-13	89.52	29.37	3.05	OED
131	ICE	70-S	94.95	31.15	3.05	OED

TABLE D-1 DIESEL HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name E	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type T
164	MKT 70	DE70/50B	94.95	31.15	3.05	OED
354	HERA	3500	96.25	35.00	2.75	OED
107	KOBE	K 35	97.90	34.35	2.85	OED
126	ICE	1070	98.47	44.50	2.21	CED
130	ICE	60-S	98.93	31.15	3.18	OED
46	FEC	FEC 3400	99.02	33.29	2.97	OED
14	DELMAG	D 30-23	99.90	29.37	3.40	OED
15	DELMAG	D 30-32	99.90	29.37	3.40	OED
418	BERMINGH	B450 5	105.63	29.37	3.60	OED
67	MITSUB.	M 43	109.06	42.10	2.59	OED
16	DELMAG	D 36	113.69	35.29	3.22	OED
17	DELMAG	D 36-02	113.69	35.29	3.22	OED
18	DELMAG	D 36-13	113.69	35.29	3.22	OED
68	MITSUB.	MH 45	115.87	44.72	2.59	OED
421	BERMINGH	B550 C	119.36	48.95	2.44	OED
19	DELMAG	D 36-23	120.04	35.29	3.40	OED
20	DELMAG	D 36-32	120.04	35.29	3.40	OED
133	ICE	90-S	122.07	40.05	3.05	OED
21	DELMAG	D 44	122.67	42.27	2.90	OED
419	BERMINGH	B500 5	124.84	34.71	3.60	OED
110	KOBE	K 45	125.81	44.14	2.85	OED
24	DELMAG	D 46-13	130.93	45.12	2.90	OED
132	ICE	80-S	134.77	35.60	3.79	OED
136	ICE	200-S	135.64	89.00	1.52	OED
355	HERA	5000	137.50	50.00	2.75	OED
420	BERMINGH	B550 5	144.05	40.05	3.60	OED
22	DELMAG	D 46	145.37	45.12	3.22	OED
23	DELMAG	D 46-02	145.37	45.12	3.22	OED
25	DELMAG	D 46-23	145.37	45.12	3.22	OED
165	MKT 110	DE110150	149.20	48.95	3.05	OED
26	DELMAG	D 46-32	153.49	45.12	3.40	OED
356	HERA	5700	156.75	57.00	2.75	OED
134	ICE	100-S	162.76	44.50	3.66	OED
27	DELMAG	D 55	168.91	52.78	3.20	OED
357	HERA	6200	170.50	62.00	2.75	OED
112	KOBE	KB 60	176.58	58.87	3.00	OED
70	MITSUB.	MH 72B	183.31	70.75	2.59	OED
135	ICE	120-S	202.15	53.40	3.79	OED
71	MITSUB.	MH 80B	202.91	78.32	2.59	OED
166	MKT 150	DE110150	203.45	66.75	3.05	OED
358	HERA	7500	206.25	75.00	2.75	OED

TABLE D-1 DIESEL HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name E	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type T
28	DELMAG	D 62-02	206.77	60.79	3.40	OED
29	DELMAG	D 62-12	206.77	60.79	3.40	OED
30	DELMAG	D 62-22	206.77	60.79	3.40	OED
113	KOBE	KB 80	235.43	78.50	3.00	OED
359	HERA	8800	242.00	88.00	2.75	OED
31	DELMAG	D 80-12	252.61	78.41	3.22	OED
32	DELMAG	D 80-23	266.71	78.41	3.40	OED
33	DELMAG	D100-13	333.47	98.03	3.40	OED

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING
(sorted by Maximum Energy)

GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
301	MKT	No. 5	1.36	.89	1.52	ECH
302	MKT	No. 6	3.39	1.78	1.90	ECH
303	MKT	No. 7	5.63	3.56	1.58	ECH
205	VULCAN	VUL 02	9.85	13.35	.74	ECH
220	VULCAN	VUL 30C	9.85	13.35	.74	ECH
521	DAWSON	HPH 1200	11.73	10.20	1.15	ECH
304	MKT	9B3	11.87	7.12	1.67	ECH
305	MKT	10B3	17.78	13.35	1.33	ECH
306	MKT	C5-Air	19.26	22.25	.87	ECH
171	CONMACO	C 50	20.35	22.25	.91	ECH
204	VULCAN	VUL 01	20.35	22.25	.91	ECH
251	RAYMOND	R 1	20.35	22.25	.91	ECH
221	VULCAN	VUL 50C	20.48	22.25	.92	ECH
307	MKT	C5-Steam	21.97	22.25	.99	ECH
308	MKT	S-5	22.04	22.25	.99	ECH
522	DAWSON	HPH 2400	23.49	18.64	1.26	ECH
541	BANUT	3 Tonnes	23.53	29.41	.80	ECH
309	MKT	11B3	25.97	22.25	1.17	ECH
222	VULCAN	VUL 65C	26.01	28.92	.90	ECH
172	CONMACO	C 65	26.45	28.92	.91	ECH
206	VULCAN	VUL 06	26.45	28.92	.91	ECH
252	RAYMOND	R 1S	26.45	28.92	.91	ECH
253	RAYMOND	R 65C	26.45	28.92	.91	ECH
254	RAYMOND	R 65CH	26.45	28.92	.91	ECH
223	VULCAN	VUL 65CA	26.54	28.92	.92	ECH
311	MKT	C826 Air	28.75	35.60	.81	ECH
341	IHC Hydh	SC 30	30.02	16.20	1.85	ECH
542	BANUT	4 Tonnes	31.39	39.25	.80	ECH
255	RAYMOND	R 0	33.06	33.38	.99	ECH
310	MKT	C826 Stm	33.10	35.60	.93	ECH
224	VULCAN	VUL 80C	33.20	35.60	.93	ECH
256	RAYMOND	R 80C	33.20	35.60	.93	ECH
257	RAYMOND	R 80CH	33.20	35.60	.93	ECH
449	MENCK	MHF3-3	33.59	31.39	1.07	ECH
515	UDDCOMB	H3H	33.75	29.37	1.15	ECH
173	CONMACO	C 550	33.91	22.25	1.52	ECH
235	VULCAN	VUL 505	33.91	22.25	1.52	ECH
320	IHC Hydh	S 35	35.01	32.35	1.08	ECH
225	VULCAN	VUL 85C	35.25	37.91	.93	ECH
175	CONMACO	C 80	35.27	35.60	.99	ECH
207	VULCAN	VUL 08	35.27	35.60	.99	ECH

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
312	MKT	S-8	35.27	35.60	.99	ECH
381	BSP	HH 3	35.29	29.42	1.20	ECH
481	JUNTTAN	HHK 3	36.01	29.46	1.22	ECH
543	BANUT	5 Tonnes	39.22	49.04	.80	ECH
342	IHC Hydh	SC 40	39.98	24.52	1.63	ECH
321	IHC Hydh	S 40	41.18	24.52	1.68	ECH
313	MKT	MS-350	41.78	34.35	1.22	ECH
450	MENCK	MHF3-4	41.99	39.24	1.07	ECH
174	CONMACO	C 565	44.08	28.92	1.52	ECH
176	CONMACO	C 100	44.08	44.50	.99	ECH
208	VULCAN	VUL 010	44.08	44.50	.99	ECH
236	VULCAN	VUL 506	44.08	28.92	1.52	ECH
258	RAYMOND	R 2/0	44.08	44.50	.99	ECH
314	MKT	S 10	44.08	44.50	.99	ECH
506	HPSI	650	44.08	28.92	1.52	ECH
372	FAIRCHLD	F-32	44.15	48.28	.91	ECH
226	VULCAN	VUL 100C	44.62	44.50	1.00	ECH
516	UDDCOMB	H4H	45.00	39.16	1.15	ECH
544	BANUT	6 Tonnes	47.09	58.87	.80	ECH
482	JUNTTAN	HHK 4	47.97	39.25	1.22	ECH
227	VULCAN	VUL 140C	48.80	62.30	.78	ECH
177	CONMACO	C 115	50.69	51.17	.99	ECH
315	MKT	S 14	50.89	62.30	.82	ECH
551	ICE	110-SH	51.16	51.17	1.00	ECH
552	ICE	115-SH	51.47	51.17	1.01	ECH
441	MENCK	MHF5-5	52.48	49.05	1.07	ECH
451	MENCK	MHF3-5	52.48	49.05	1.07	ECH
209	VULCAN	VUL 012	52.90	53.40	.99	ECH
178	CONMACO	C 80E5	54.25	35.60	1.52	ECH
237	VULCAN	VUL 508	54.25	35.60	1.52	ECH
545	BANUT	7 Tonnes	54.92	68.66	.80	ECH
259	RAYMOND	R 3/0	55.10	55.62	.99	ECH
517	UDDCOMB	H5H	56.25	48.95	1.15	ECH
182	CONMACO	C 140	56.97	62.30	.91	ECH
210	VULCAN	VUL 014	56.97	62.30	.91	ECH
382	BSP	HH 5	58.83	49.04	1.20	ECH
316	MKT	MS 500	59.68	48.95	1.22	ECH
501	HPSI	110	59.68	48.95	1.22	ECH
489	JUNTTAN	HHK 5A	59.79	49.04	1.22	ECH
483	JUNTTAN	HHK 5	59.99	49.08	1.22	ECH
343	IHC Hydh	SC 60	60.00	34.35	1.75	ECH

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
322	IHC Hydh	S 60	60.04	58.86	1.02	ECH
371	FAIRCHLD	F-45	61.04	66.75	.91	ECH
282	MENCK	MRBS 500	61.13	49.04	1.25	ECH
442	MENCK	MHF5-6	62.98	58.86	1.07	ECH
452	MENCK	MHF3-6	62.98	58.86	1.07	ECH
183	CONMACO	C 160	66.12	72.31	.91	ECH
211	VULCAN	VUL 016	66.12	72.31	.91	ECH
260	RAYMOND	R 150C	66.12	66.75	.99	ECH
261	RAYMOND	R 4/0	66.12	66.75	.99	ECH
271	MENCK	MH 68	66.70	34.35	1.94	ECH
518	UDDCOMB	H6H	67.50	58.74	1.15	ECH
179	CONMACO	C 100E5	67.82	44.50	1.52	ECH
507	HPSI	1000	67.82	44.50	1.52	ECH
238	VULCAN	VUL 510	67.82	44.50	1.52	ECH
228	VULCAN	VUL 200C	68.09	89.00	.77	ECH
323	IHC Hydh	S 70	70.05	34.35	2.04	ECH
191	CONMACO	C 160 **	70.23	76.81	.91	ECH
484	JUNTTAN	HHK 6	71.96	58.87	1.22	ECH
443	MENCK	MHF5-7	73.48	68.67	1.07	ECH
453	MENCK	MHF3-7	73.48	68.67	1.07	ECH
262	RAYMOND	R 5/0	77.14	77.88	.99	ECH
180	CONMACO	C 115E5	77.99	51.17	1.52	ECH
344	IHC Hydh	SC 80	79.89	50.02	1.60	ECH
184	CONMACO	C 200	81.38	89.00	.91	ECH
212	VULCAN	VUL 020	81.38	89.00	.91	ECH
231	VULCAN	VUL 320	81.38	89.00	.91	ECH
239	VULCAN	VUL 512	81.38	53.40	1.52	ECH
317	MKT	S 20	81.38	89.00	.91	ECH
502	HPSI	150	81.38	66.75	1.22	ECH
383	BSP	HH 7	82.44	68.65	1.20	ECH
503	HPSI	154	83.55	68.53	1.22	ECH
490	JUNTTAN	HHK 7A	83.71	68.66	1.22	ECH
444	MENCK	MHF5-8	83.97	78.48	1.07	ECH
485	JUNTTAN	HHK 7	83.98	68.71	1.22	ECH
181	CONMACO	C 125E5	84.77	55.62	1.52	ECH
553	ICE	160-SH	86.81	71.20	1.22	ECH
324	IHC Hydh	S 90	90.01	44.14	2.04	ECH
283	MENCK	MRBS 750	91.92	73.56	1.25	ECH
519	UDDCOMB	H8H	94.06	78.32	1.20	ECH
272	MENCK	MH 96	94.17	49.04	1.92	ECH
384	BSP	HH 8	94.27	78.50	1.20	ECH

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
445	MENCK	MHF5-9	94.47	88.29	1.07	ECH
263	RAYMOND	R 30X	101.73	133.50	.76	ECH
446	MENCK	MHF5-10	104.97	98.10	1.07	ECH
345	IHC Hydh	SC 110	105.01	67.68	1.55	ECH
385	BSP	HH 9	106.03	88.29	1.20	ECH
491	JUNTTAN	HHK 9A	107.64	88.29	1.22	ECH
504	HPSI	200	108.51	89.00	1.22	ECH
264	RAYMOND	R 8/0	110.2	111.25	.99	ECH
508	HPSI	1605	112.58	73.87	1.52	ECH
447	MENCK	MHF5-11	115.46	107.91	1.07	ECH
520	UDDCOMB	H10H	117.84	98.12	1.20	ECH
486	JUNTTAN	HHK 10	119.93	98.12	1.22	ECH
185	CONMACO	C 300	122.07	133.50	.91	ECH
213	VULCAN	VUL 030	122.07	133.50	.91	ECH
232	VULCAN	VUL 330	122.07	133.50	.91	ECH
270	9K DROP	9K DROP	122.07	40.05	3.05	ECH
505	HPSI	225	122.07	100.12	1.22	ECH
448	MENCK	MHF5-12	125.96	117.72	1.07	ECH
284	MENCK	MRBS 800	126.53	84.37	1.50	ECH
285	MENCK	MRBS 850	126.53	84.37	1.50	ECH
509	HPSI	2005	128.99	84.64	1.52	ECH
386	BSP	HH 11	129.59	107.91	1.20	ECH
186	CONMACO	C 5200	135.64	89.00	1.52	ECH
240	VULCAN	VUL 520	135.64	89.00	1.52	ECH
265	RAYMOND	R 40X	135.64	178.00	.76	ECH
346	IHC Hydh	SC 150	140.12	107.91	1.30	ECH
273	MENCK	MH 145	142.15	73.56	1.93	ECH
487	JUNTTAN	HHK 12	143.92	117.75	1.22	ECH
229	VULCAN	VUL 400C	154.08	178.00	.87	ECH
454	MENCK	MHF10-15	157.39	147.12	1.07	ECH
214	VULCAN	VUL 040	162.76	178.00	.91	ECH
233	VULCAN	VUL 340	162.76	178.00	.91	ECH
387	BSP	HH 14	164.92	137.33	1.20	ECH
286	MENCK	MRBS1100	167.42	107.91	1.55	ECH
488	JUNTTAN	HHK 14	167.90	137.37	1.22	ECH
287	MENCK	MRBS1502	183.90	147.16	1.25	ECH
388	BSP	HH 16	188.35	156.96	1.20	ECH
274	MENCK	MH 195	191.41	98.12	1.95	ECH
325	IHC Hydh	S 200	199.63	97.90	2.04	ECH
461	MENCK	MHUT 200	199.90	117.75	1.70	ECH
187	CONMACO	C 5300	203.45	133.50	1.52	ECH

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
241	VULCAN	VUL 530	203.45	133.50	1.52	ECH
266	RAYMOND	R 60X	203.45	267.00	.76	ECH
347	IHC Hydh	SC 200	204.81	134.39	1.52	ECH
455	MENCK	MHF10-20	209.81	196.11	1.07	ECH
510	HPSI	3005	209.32	137.35	1.52	ECH
275	MENCK	MHU 220	215.76	111.83	1.93	ECH
389	BSP	HH 20	235.44	196.20	1.20	ECH
390	BSP	HH 20S	235.44	196.20	1.20	ECH
511	HPSI	3505	239.16	156.93	1.52	ECH
348	IHC Hydh	SC 250	240.04	174.62	1.37	ECH
230	VULCAN	VUL 600C	243.01	267.00	.91	ECH
215	VULCAN	VUL 060	244.14	267.00	.91	ECH
234	VULCAN	VUL 360	244.14	267.00	.91	ECH
326	IHC Hydh	S 250	250.44	122.82	2.04	ECH
288	MENCK	MRBS1800	257.46	171.68	1.50	ECH
242	VULCAN	VUL 540	271.27	182.01	1.49	ECH
327	IHC Hydh	S 280	280.11	132.61	2.11	ECH
188	CONMACO	C 5450	305.18	200.25	1.52	ECH
290	MENCK	MRBS2502	306.47	245.24	1.25	ECH
291	MENCK	MRBS2504	306.47	245.24	1.25	ECH
391	BSP	HA 30	353.16	294.30	1.20	ECH
289	MENCK	MRBS2500	355.52	284.49	1.25	ECH
276	MENCK	MHU 400	392.74	225.66	1.74	ECH
328	IHC Hydh	S 400	399.58	197.13	2.03	ECH
462	MENCK	MHUT 400	400.29	234.51	1.71	ECH
243	VULCAN	VUL 560	406.91	278.13	1.46	ECH
245	VULCAN	VUL 3100	406.91	445.00	.91	ECH
292	MENCK	MRBS3000	441.30	294.28	1.50	ECH
392	BSP	HA 40	470.88	392.40	1.20	ECH
189	CONMACO	C 5700	474.73	311.50	1.52	ECH
329	IHC Hydh	S 500	499.54	246.08	2.03	ECH
463	MENCK	MHUT 500	499.89	264.95	1.89	ECH
277	MENCK	MHU 600	588.17	343.36	1.71	ECH
294	MENCK	MRBS4600	676.74	451.27	1.50	ECH
246	VULCAN	VUL 5100	678.18	445.00	1.52	ECH
190	CONMACO	C 6850	691.74	378.25	1.83	ECH
293	MENCK	MRBS3900	696.28	386.53	1.80	ECH
464	MENCK	MHUT700U	700.06	413.09	1.69	ECH
295	MENCK	MRBS5000	735.60	490.52	1.50	ECH
330	IHC Hydh	S 800	800.05	363.00	2.20	ECH
465	MENCK	MHUT700A	839.83	413.09	2.03	ECH

TABLE D-2 EXTERNAL COMBUSTION HAMMER LISTING (sorted by Maximum Energy)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
297	MENCK	MRBS7000	856.41	685.30	1.25	ECH
466	MENCK	MHUT1000	999.52	588.73	1.70	ECH
331	IHC Hydh	S 1000	999.99	451.26	2.22	ECH
278	MENCK	MHU 1000	1000.58	565.02	1.77	ECH
247	VULCAN	VUL 5150	1017.27	667.50	1.52	ECH
296	MENCK	MRBS6000	1029.79	588.60	1.75	ECH
298	MENCK	MRBS8000	1176.97	784.85	1.50	ECH
299	MENCK	MRBS8800	1294.69	863.34	1.50	ECH
332	IHC Hydh	S 1600	1597.52	694.20	2.30	ECH
279	MENCK	MHU 1700	1666.80	922.17	1.81	ECH
280	MENCK	MHU 2100	2099.09	1138.31	1.84	ECH
300	MENCK	MBS12500	2145.53	1226.33	1.75	ECH
333	IHC Hydh	S 2300	2298.99	1008.37	2.28	ECH
248	VULCAN	VUL 6300	2441.45	1335.00	1.83	ECH
281	MENCK	MHU 3000	2945.54	1618.73	1.82	ECH
334	IHC Hydh	S 3000	2997.72	1477.40	2.03	ECH

TABLE D-3 COMPLETE HAMMER LISTING
(sorted by GRLWEAP ID Numbers)

GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
1	DELMAG	D 5	11.16	4.89	2.28	OED
2	DELMAG	D 8-22	23.87	7.83	3.05	OED
3	DELMAG	D 12	32.00	12.24	2.62	OED
4	DELMAG	D 15	38.40	14.68	2.62	OED
5	DELMAG	D 16-32	53.23	15.66	3.40	OED
6	DELMAG	D 22	55.08	21.85	2.52	OED
7	DELMAG	D 22-02	65.78	21.58	3.05	OED
8	DELMAG	D 22-13	65.78	21.58	3.05	OED
9	DELMAG	D 22-23	69.53	21.58	3.22	OED
10	DELMAG	D 25-32	83.40	24.52	3.40	OED
11	DELMAG	D 30	80.84	29.37	2.75	OED
12	DELMAG	D 30-02	89.52	29.37	3.05	OED
13	DELMAG	D 30-13	89.52	29.37	3.05	OED
14	DELMAG	D 30-23	99.90	29.37	3.40	OED
15	DELMAG	D 30-32	99.90	29.37	3.40	OED
16	DELMAG	D 36	113.69	35.29	3.22	OED
17	DELMAG	D 36-02	113.69	35.29	3.22	OED
18	DELMAG	D 36-13	113.69	35.29	3.22	OED
19	DELMAG	D 36-23	120.04	35.29	3.40	OED
20	DELMAG	D 36-32	120.04	35.29	3.40	OED
21	DELMAG	D 44	122.67	42.27	2.90	OED
22	DELMAG	D 46	145.37	45.12	3.22	OED
23	DELMAG	D 46-02	145.37	45.12	3.22	OED
24	DELMAG	D 46-13	130.93	45.12	2.90	OED
25	DELMAG	D 46-23	145.37	45.12	3.22	OED
26	DELMAG	D 46-32	153.49	45.12	3.40	OED
27	DELMAG	D 55	168.91	52.78	3.20	OED
28	DELMAG	D 62-02	206.77	60.79	3.40	OED
29	DELMAG	D 62-12	206.77	60.79	3.40	OED
30	DELMAG	D 62-22	206.77	60.79	3.40	OED
31	DELMAG	D 80-12	252.61	78.41	3.22	OED
32	DELMAG	D 80-23	266.71	78.41	3.40	OED
33	DELMAG	D100-13	333.47	98.03	3.40	OED
36	DELMAG	D 6-32	14.24	5.88	2.42	OED
37	DELMAG	D 12-32	42.50	12.55	3.39	OED
38	DELMAG	D 19-32	57.51	17.80	3.23	OED
41	FEC	FEC 1200	30.51	12.24	2.49	OED
42	FEC	FEC 1500	36.75	14.68	2.50	OED
43	FEC	FEC 2500	67.81	24.47	2.77	OED
44	FEC	FEC 2800	75.95	27.41	2.77	OED
45	FEC	FEC 3000	85.49	29.37	2.91	OED

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
46	FEC	FEC 3400	99.02	33.29	2.97	OED
61	MITSUB.	M 14	34.24	13.22	2.59	OED
62	MITSUB.	MH 15	38.16	14.73	2.59	OED
63	MITSUB.	M 23	58.34	22.52	2.59	OED
64	MITSUB.	MH 25	63.53	24.52	2.59	OED
65	MITSUB.	M 33	83.70	32.31	2.59	OED
66	MITSUB.	MH 35	89.00	34.35	2.59	OED
67	MITSUB.	M 43	109.06	42.10	2.59	OED
68	MITSUB.	MH 45	115.87	44.72	2.59	OED
70	MITSUB.	MH 72B	183.31	70.75	2.59	OED
71	MITSUB.	MH 80B	202.91	78.32	2.59	OED
81	LINKBELT	LB 180	10.98	7.70	1.43	CED
82	LINKBELT	LB 312	20.37	17.18	1.19	CED
83	LINKBELT	LB 440	24.69	17.80	1.39	CED
84	LINKBELT	LB 520	35.69	22.56	1.58	CED
85	LINKBELT	LB 660	70.03	33.69	2.08	CED
101	KOBE	K 13	34.49	12.77	2.70	OED
103	KOBE	K22-Est	61.51	21.58	2.85	OED
104	KOBE	K 25	69.88	24.52	2.85	OED
107	KOBE	K 35	97.90	34.35	2.85	OED
110	KOBE	K 45	125.81	44.14	2.85	OED
112	KOBE	KB 60	176.58	58.87	3.00	OED
113	KOBE	KB 80	235.43	78.50	3.00	OED
120	ICE	180	11.03	7.70	1.43	CED
121	ICE	422	31.36	17.80	1.76	CED
122	ICE	440	25.17	17.80	1.41	CED
123	ICE	520	41.19	22.56	1.83	CED
124	ICE	640	55.10	26.70	2.06	CED
125	ICE	660	70.03	33.69	2.08	CED
126	ICE	1070	98.47	44.50	2.21	CED
127	ICE	30-S	30.52	13.35	2.29	OED
128	ICE	40-S	54.25	17.80	3.05	OED
129	ICE	42-S	56.97	18.19	3.13	OED
130	ICE	60-S	98.93	31.15	3.18	OED
131	ICE	70-S	94.95	31.15	3.05	OED
132	ICE	80-S	134.77	35.60	3.79	OED
133	ICE	90-S	122.07	40.05	3.05	OED
134	ICE	100-S	162.76	44.50	3.66	OED
135	ICE	120-S	202.15	53.40	3.79	OED
136	ICE	200-S	135.64	89.00	1.52	OED
141	MKT 20	DE333020	27.13	8.90	3.05	OED

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
142	MKT 30	DE333020	37.98	12.46	3.05	OED
143	MKT 33	DE333020	44.76	14.68	3.05	OED
144	MKT 40	DE333020	54.25	17.80	3.05	OED
146	MKT	DE10	20.75	7.57	2.74	OED
147	MKT	DE 20	21.70	8.90	2.44	OED
148	MKT	DE 30	30.38	12.46	2.44	OED
149	MKT	DA35B SA	32.28	12.46	2.59	OED
150	MKT	DE 30B	32.28	12.46	2.59	OED
151	MKT	DA 35B	28.48	12.46	2.29	CED
152	MKT	DA 45	41.67	17.80	2.34	CED
153	MKT	DE 40	43.40	17.80	2.44	OED
159	MKT	DE 50B	57.65	22.25	2.59	OED
160	MKT	DA55B SA	54.25	22.25	2.44	OED
161	MKT	DA 55B	51.81	22.25	2.33	CED
162	MKT	DE 70B	80.70	31.15	2.59	OED
163	MKT 50	DE70/50B	67.82	22.25	3.05	OED
164	MKT 70	DE70/50B	94.95	31.15	3.05	OED
165	MKT110	DE110150	149.20	48.95	3.05	OED
166	MKT150	DE110150	203.45	66.75	3.05	OED
171	CONMACO	C 50	20.35	22.25	.91	ECH
172	CONMACO	C 65	26.45	28.92	.91	ECH
173	CONMACO	C 550	33.91	22.25	1.52	ECH
174	CONMACO	C 565	44.08	28.92	1.52	ECH
175	CONMACO	C 80	35.27	35.60	.99	ECH
176	CONMACO	C 100	44.08	44.50	.99	ECH
177	CONMACO	C 115	50.69	51.17	.99	ECH
178	CONMACO	C 80E5	54.25	35.60	1.52	ECH
179	CONMACO	C 100E5	67.82	44.50	1.52	ECH
180	CONMACO	C 115E5	77.99	51.17	1.52	ECH
181	CONMACO	C 125E5	84.77	55.62	1.52	ECH
182	CONMACO	C 140	56.97	62.30	.91	ECH
183	CONMACO	C 160	66.12	72.31	.91	ECH
184	CONMACO	C 200	81.38	89.00	.91	ECH
185	CONMACO	C 300	122.07	133.50	.91	ECH
186	CONMACO	C 5200	135.64	89.00	1.52	ECH
187	CONMACO	C 5300	203.45	133.50	1.52	ECH
188	CONMACO	C 5450	305.18	200.25	1.52	ECH
189	CONMACO	C 5700	474.73	311.50	1.52	ECH
190	CONMACO	C 6850	691.74	378.25	1.83	ECH
191	CONMACO	C 160 **	70.23	76.81	.91	ECH
201	VULCAN	VUL V15	36.77	12.26	3.00	OED

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
202	VULCAN	VUL V18	52.97	17.66	3.00	OED
203	VULCAN	VUL V25	78.51	24.53	3.20	OED
204	VULCAN	VUL 01	20.35	22.25	.91	ECH
205	VULCAN	VUL 02	9.85	13.35	.74	ECH
206	VULCAN	VUL 06	26.45	28.92	.91	ECH
207	VULCAN	VUL 08	35.27	35.60	.99	ECH
208	VULCAN	VUL010	44.08	44.50	.99	ECH
209	VULCAN	VUL012	52.90	53.40	.99	ECH
210	VULCAN	VUL014	56.97	62.30	.91	ECH
211	VULCAN	VUL016	66.12	72.31	.91	ECH
212	VULCAN	VUL020	81.38	89.00	.91	ECH
213	VULCAN	VUL030	122.07	133.50	.91	ECH
214	VULCAN	VUL040	162.76	178.00	.91	ECH
215	VULCAN	VUL060	244.14	267.00	.91	ECH
220	VULCAN	VUL30C	9.85	13.35	.74	ECH
221	VULCAN	VUL50C	20.48	22.25	.92	ECH
222	VULCAN	VUL65C	26.01	28.92	.90	ECH
223	VULCAN	VUL 65CA	26.54	28.92	.92	ECH
224	VULCAN	VUL80C	33.20	35.60	.93	ECH
225	VULCAN	VUL85C	35.25	37.91	.93	ECH
226	VULCAN	VUL 100C	44.62	44.50	1.00	ECH
227	VULCAN	VUL 140C	48.80	62.30	.78	ECH
228	VULCAN	VUL 200C	68.09	89.00	.77	ECH
229	VULCAN	VUL 400C	154.08	178.00	.87	ECH
230	VULCAN	VUL 600C	243.01	267.00	.91	ECH
231	VULCAN	VUL320	81.38	89.00	.91	ECH
232	VULCAN	VUL330	122.07	133.50	.91	ECH
233	VULCAN	VUL340	162.76	178.00	.91	ECH
234	VULCAN	VUL360	244.14	267.00	.91	ECH
235	VULCAN	VUL505	33.91	22.25	1.52	ECH
236	VULCAN	VUL506	44.08	28.92	1.52	ECH
237	VULCAN	VUL508	54.25	35.60	1.52	ECH
238	VULCAN	VUL510	67.82	44.50	1.52	ECH
239	VULCAN	VUL512	81.38	53.40	1.52	ECH
240	VULCAN	VUL520	135.64	89.00	1.52	ECH
241	VULCAN	VUL530	203.45	133.50	1.52	ECH
242	VULCAN	VUL540	271.27	182.01	1.49	ECH
243	VULCAN	VUL560	406.91	278.13	1.46	ECH
245	VULCAN	VUL 3100	406.91	445.00	.91	ECH
246	VULCAN	VUL 5100	678.18	445.00	1.52	ECH
247	VULCAN	VUL 5150	1017.27	667.50	1.52	ECH

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
248	VULCAN	VUL 6300	2441.45	1335.00	1.83	ECH
251	RAYMOND	R 1	20.35	22.25	.91	ECH
252	RAYMOND	R 1S	26.45	28.92	.91	ECH
253	RAYMOND	R 65C	26.45	28.92	.91	ECH
254	RAYMOND	R 65CH	26.45	28.92	.91	ECH
255	RAYMOND	R 0	33.06	33.38	.99	ECH
256	RAYMOND	R 80C	33.20	35.60	.93	ECH
257	RAYMOND	R 80CH	33.20	35.60	.93	ECH
258	RAYMOND	R 2/0	44.08	44.50	.99	ECH
259	RAYMOND	R 3/0	55.10	55.62	.99	ECH
260	RAYMOND	R 150C	66.12	66.75	.99	ECH
261	RAYMOND	R 4/0	66.12	66.75	.99	ECH
262	RAYMOND	R 5/0	77.14	77.88	.99	ECH
263	RAYMOND	R 30X	101.73	133.50	.76	ECH
264	RAYMOND	R 8/0	110.20	111.25	.99	ECH
265	RAYMOND	R 40X	135.64	178.00	.76	ECH
266	RAYMOND	R 60X	203.45	267.00	.76	ECH
270	9K DROP	9K DROP	122.07	40.05	3.05	ECH
271	MENCK	MH 68	66.70	34.35	1.94	ECH
272	MENCK	MH 96	94.17	49.04	1.92	ECH
273	MENCK	MH 145	142.15	73.56	1.93	ECH
274	MENCK	MH 195	191.41	98.12	1.95	ECH
275	MENCK	MHU 220	215.76	111.83	1.93	ECH
276	MENCK	MHU 400	392.74	225.66	1.74	ECH
277	MENCK	MHU 600	588.17	343.36	1.71	ECH
278	MENCK	MHU 1000	1000.58	565.02	1.77	ECH
279	MENCK	MHU 1700	1666.80	922.17	1.81	ECH
280	MENCK	MHU 2100	2099.09	1138.31	1.84	ECH
281	MENCK	MHU 3000	2945.54	1618.73	1.82	ECH
282	MENCK	MRBS 500	61.13	49.04	1.25	ECH
283	MENCK	MRBS 750	91.92	73.56	1.25	ECH
284	MENCK	MRBS 800	126.53	84.37	1.50	ECH
285	MENCK	MRBS 850	126.53	84.37	1.50	ECH
286	MENCK	MRBS1100	167.42	107.91	1.55	ECH
287	MENCK	MRBS1502	183.90	147.16	1.25	ECH
288	MENCK	MRBS1800	257.46	171.68	1.50	ECH
289	MENCK	MRBS2500	355.52	284.49	1.25	ECH
290	MENCK	MRBS2502	306.47	245.24	1.25	ECH
291	MENCK	MRBS2504	306.47	245.24	1.25	ECH
292	MENCK	MRBS3000	441.30	294.28	1.50	ECH
293	MENCK	MRBS3900	696.28	386.53	1.80	ECH

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
294	MENCK	MRBS4600	676.74	451.27	1.50	ECH
295	MENCK	MRBS5000	735.60	490.52	1.50	ECH
296	MENCK	MRBS6000	1029.79	588.60	1.75	ECH
297	MENCK	MRBS7000	856.41	685.30	1.25	ECH
298	MENCK	MRBS8000	1176.97	784.85	1.50	ECH
299	MENCK	MRBS8800	1294.69	863.34	1.50	ECH
300	MENCK	MBS12500	2145.53	1226.33	1.75	ECH
301	MKT	No. 5	1.36	.89	1.52	ECH
302	MKT	No. 6	3.39	1.78	1.90	ECH
303	MKT	No. 7	5.63	3.56	1.58	ECH
304	MKT	9B3	11.87	7.12	1.67	ECH
305	MKT	10B3	17.78	13.35	1.33	ECH
306	MKT	C5-Air	19.26	22.25	.87	ECH
307	MKT	C5-Steam	21.97	22.25	.99	ECH
308	MKT	S-5	22.04	22.25	.99	ECH
309	MKT	11B3	25.97	22.25	1.17	ECH
310	MKT	C826 Stm	33.10	35.60	.93	ECH
311	MKT	C826 Air	28.75	35.60	.81	ECH
312	MKT	S-8	35.27	35.60	.99	ECH
313	MKT	MS-350	41.78	34.35	1.22	ECH
314	MKT	S 10	44.08	44.50	.99	ECH
315	MKT	S 14	50.89	62.30	.82	ECH
316	MKT	MS 500	59.68	48.95	1.22	ECH
317	MKT	S 20	81.38	89.00	.91	ECH
320	IHC Hydh	S 35	35.01	32.35	1.08	ECH
321	IHC Hydh	S 40	41.18	24.52	1.68	ECH
322	IHC Hydh	S 60	60.04	58.86	1.02	ECH
323	IHC Hydh	S 70	70.05	34.35	2.04	ECH
324	IHC Hydh	S 90	90.01	44.14	2.04	ECH
325	IHC Hydh	S 200	199.63	97.90	2.04	ECH
326	IHC Hydh	S 250	250.44	122.82	2.04	ECH
327	IHC Hydh	S 280	280.11	132.61	2.11	ECH
328	IHC Hydh	S 400	399.58	197.13	2.03	ECH
329	IHC Hydh	S 500	499.54	246.08	2.03	ECH
330	IHC Hydh	S 800	800.05	363.00	2.20	ECH
331	IHC Hydh	S 1000	999.99	451.26	2.22	ECH
332	IHC Hydh	S 1600	1597.52	694.20	2.30	ECH
333	IHC Hydh	S 2300	2298.99	1008.37	2.28	ECH
334	IHC Hydh	S 3000	2997.72	1477.40	2.03	ECH
341	IHC Hydh	SC 30	30.02	16.20	1.85	ECH
342	IHC Hydh	SC 40	39.98	24.52	1.63	ECH

TABLE D-3 COMPLETE HAMMER LISTING
(sorted by GRLWEAP ID Numbers)

GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
343	IHC Hydh	SC 60	60.00	34.35	1.75	ECH
344	IHC Hydh	SC 80	79.89	50.02	1.60	ECH
345	IHC Hydh	SC 110	105.01	67.68	1.55	ECH
346	IHC Hydh	SC 150	140.12	107.91	1.30	ECH
347	IHC Hydh	SC 200	204.81	134.39	1.52	ECH
348	IHC Hydh	SC 250	240.04	174.62	1.37	ECH
350	HERA	1250	34.38	12.50	2.75	OED
351	HERA	1500	41.25	15.00	2.75	OED
352	HERA	2500	68.75	25.00	2.75	OED
353	HERA	2800	77.00	28.00	2.75	OED
354	HERA	3500	96.25	35.00	2.75	OED
355	HERA	5000	137.50	50.00	2.75	OED
356	HERA	5700	156.75	57.00	2.75	OED
357	HERA	6200	170.50	62.00	2.75	OED
358	HERA	7500	206.25	75.00	2.75	OED
359	HERA	8800	242.00	88.00	2.75	OED
371	FAIRCHLD	F-45	61.04	66.75	.91	ECH
372	FAIRCHLD	F-32	44.15	48.28	.91	ECH
381	BSP	HH 3	35.29	29.42	1.20	ECH
382	BSP	HH 5	58.83	49.04	1.20	ECH
383	BSP	HH 7	82.44	68.65	1.20	ECH
384	BSP	HH 8	94.27	78.50	1.20	ECH
385	BSP	HH 9	106.03	88.29	1.20	ECH
386	BSP	HH 11	129.59	107.91	1.20	ECH
387	BSP	HH 14	164.92	137.33	1.20	ECH
388	BSP	HH 16	188.35	156.96	1.20	ECH
389	BSP	HH 20	235.44	196.20	1.20	ECH
390	BSP	HH 20S	235.44	196.20	1.20	ECH
391	BSP	HA 30	353.16	294.30	1.20	ECH
392	BSP	HA 40	470.88	392.40	1.20	ECH
401	BERMINGH	B23	31.18	12.46	2.50	CED
402	BERMINGH	B200	24.41	8.90	2.74	OED
403	BERMINGH	B225	39.67	13.35	2.97	OED
404	BERMINGH	B300	54.68	16.69	3.28	OED
405	BERMINGH	B400	72.90	22.25	3.28	OED
410	BERMINGH	B300 M	54.68	16.69	3.28	OED
411	BERMINGH	B400 M	72.90	22.25	3.28	OED
412	BERMINGH	B400 4.8	58.59	21.36	2.74	OED
413	BERMINGH	B400 5.0	61.04	22.25	2.74	OED
414	BERMINGH	B23 5	31.18	12.46	2.50	CED
415	BERMINGH	B250 5	48.02	13.35	3.60	OED

TABLE D-3 COMPLETE HAMMER LISTING (sorted by GRLWEAP ID Numbers)						
GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
416	BERMINGH	B350 5	64.02	17.80	3.60	OED
417	BERMINGH	B400 5	80.03	22.25	3.60	OED
418	BERMINGH	B450 5	105.63	29.37	3.60	OED
419	BERMINGH	B500 5	124.84	34.71	3.60	OED
420	BERMINGH	B550 5	144.05	40.05	3.60	OED
421	BERMINGH	B550 C	119.36	48.95	2.44	OED
441	MENCK	MHF5-5	52.48	49.05	1.07	ECH
442	MENCK	MHF5-6	62.98	58.86	1.07	ECH
443	MENCK	MHF5-7	73.48	68.67	1.07	ECH
444	MENCK	MHF5-8	83.97	78.48	1.07	ECH
445	MENCK	MHF5-9	94.47	88.29	1.07	ECH
446	MENCK	MHF5-10	104.97	98.10	1.07	ECH
447	MENCK	MHF5-11	115.46	107.91	1.07	ECH
448	MENCK	MHF5-12	125.96	117.72	1.07	ECH
449	MENCK	MHF3-3	33.59	31.39	1.07	ECH
450	MENCK	MHF3-4	41.99	39.24	1.07	ECH
451	MENCK	MHF3-5	52.48	49.05	1.07	ECH
452	MENCK	MHF3-6	62.98	58.86	1.07	ECH
453	MENCK	MHF3-7	73.48	68.67	1.07	ECH
454	MENCK	MHF10-15	157.39	147.12	1.07	ECH
455	MENCK	MHF10-20	209.81	196.11	1.07	ECH
461	MENCK	MHUT 200	199.90	117.75	1.70	ECH
462	MENCK	MHUT 400	400.29	234.51	1.71	ECH
463	MENCK	MHUT 500	499.89	264.95	1.89	ECH
464	MENCK	MHUT700U	700.06	413.09	1.69	ECH
465	MENCK	MHUT700A	839.83	413.09	2.03	ECH
466	MENCK	MHUT1000	999.52	588.73	1.70	ECH
481	JUNTTAN	HHK 3	36.01	29.46	1.22	ECH
482	JUNTTAN	HHK 4	47.97	39.25	1.22	ECH
483	JUNTTAN	HHK 5	59.99	49.08	1.22	ECH
484	JUNTTAN	HHK 6	71.96	58.87	1.22	ECH
485	JUNTTAN	HHK 7	83.98	68.71	1.22	ECH
486	JUNTTAN	HHK 10	119.93	98.12	1.22	ECH
487	JUNTTAN	HHK 12	143.92	117.75	1.22	ECH
488	JUNTTAN	HHK 14	167.90	137.37	1.22	ECH
489	JUNTTAN	HHK 5A	59.79	49.04	1.22	ECH
490	JUNTTAN	HHK 7A	83.71	68.66	1.22	ECH
491	JUNTTAN	HHK 9A	107.64	88.29	1.22	ECH
501	HPSI	110	59.68	48.95	1.22	ECH
502	HPSI	150	81.38	66.75	1.22	ECH
503	HPSI	154	83.55	68.53	1.22	ECH

TABLE D-3 COMPLETE HAMMER LISTING
(sorted by GRLWEAP ID Numbers)

GRLWEAP ID	Hammer Mfgr	Hammer Name	Max. Energy kN-m	Ram Weight kN	Eq. Max. Stroke m	Hammer Type
504	HPSI	200	108.51	89.00	1.22	ECH
505	HPSI	225	122.07	100.12	1.22	ECH
506	HPSI	650	44.08	28.92	1.52	ECH
507	HPSI	1000	67.82	44.50	1.52	ECH
508	HPSI	1605	112.58	73.87	1.52	ECH
509	HPSI	2005	128.99	84.64	1.52	ECH
510	HPSI	3005	209.32	137.35	1.52	ECH
511	HPSI	3505	239.16	156.93	1.52	ECH
515	UDDCOMB	H3H	33.75	29.37	1.15	ECH
516	UDDCOMB	H4H	45.00	39.16	1.15	ECH
517	UDDCOMB	H5H	56.25	48.95	1.15	ECH
518	UDDCOMB	H6H	67.50	58.74	1.15	ECH
519	UDDCOMB	H8H	94.06	78.32	1.20	ECH
520	UDDCOMB	H10H	117.84	98.12	1.20	ECH
521	DAWSON	HPH 1200	11.73	10.20	1.15	ECH
522	DAWSON	HPH 2400	23.49	18.64	1.26	ECH
541	BANUT	3 Tonnes	23.53	29.41	.80	ECH
542	BANUT	4 Tonnes	31.39	39.25	.80	ECH
543	BANUT	5 Tonnes	39.22	49.04	.80	ECH
544	BANUT	6 Tonnes	47.09	58.87	.80	ECH
545	BANUT	7 Tonnes	54.92	68.66	.80	ECH
551	ICE	110-SH	51.16	51.17	1.00	ECH
552	ICE	115-SH	51.47	51.17	1.01	ECH
553	ICE	160-SH	86.81	71.20	1.22	ECH
701	ICE	1412	27.13	4.45	6.10	VIB
702	ICE	815	36.21	4.45	8.14	VIB
703	ICE	812	32.59	4.01	8.14	VIB
704	ICE	416	18.11	2.23	8.14	VIB

APPENDIX E

Student Exercise - Solutions

STUDENT EXERCISE #9 SOLUTION - GATES FORMULA ULTIMATE CAPACITY

Use the Gates formula described in Section 16.3 to calculate the ultimate pile capacity of a 356 mm O.D. pipe pile driven with an ICE 42-S single acting diesel hammer to the driving resistances given in the table below. The field observed hammer strokes and corresponding manufacturer's rated energy are also included in the table. The Gates formula is presented below:

$$R_u = [\ 7\ \sqrt{E_r}\ \log\ (10\ N_b)]\ -\ 550$$

Where: R_u = ultimate pile capacity (kN).
E_r = manufacturer's rated energy at field stroke (joules).
N_b = number of hammer blows for 25 mm penetration.

For 168 blows / 250 mm at a 3.05 m stroke the solution is as follows:

$$R_u = [\ 7\ \sqrt{55480}\ \log\ (10\ (16.8))]\ -\ 550$$

$$R_u = [\ 7\ (235.5)\ \log\ (168)]\ -\ 550 = 3119\ \text{kN}$$

The table on the following page provides the problem solutions at other driving resistances and field observed strokes.

Group Number	Pile Driving Resistance (blows / 250 mm)	Field Observed Stroke (m)	Manufacturer's Rated Energy (joules)	Gates Ultimate Pile Capacity (kN)
1	3	1.67	30,377	32
2	7	2.43	44,202	693
3	18	2.88	52,387	1461
4	37	3.10	56,389	2057
5	53	3.13	56,935	2330
6	72	3.02	54,934	2497
7	87	3.04	55,298	2643
8	107	3.04	55,298	2791
9	133	3.05	55,480	2952
10	168	3.05	55,480	3119

STUDENT EXERCISE #10 SOLUTION - GATES FORMULA DRIVING CRITERION

The Gates formula is to be used for construction control on a new bridge project. The piles have a design load of 620 kN and are to be driven through 5 meters of scourable soils that were calculated to provide 90 kN of resistance at the time of driving. A Kobe K 25 single acting diesel hammer will be used to drive the piles. First determine the required ultimate pile capacity. Then use the Gates formula provided below and described in Section 16.3 to calculate the required driving resistance for the ultimate pile capacity at the hammer strokes shown in the table below.

STEP 1 Calculate the required ultimate pile capacity:

$$R_u = (\text{ design load })(\text{ factor of safety }) + \text{scour resistance}$$

$$= (620 \text{ kN})(3.5) + 90 \text{ kN} = 2260 \text{ kN}.$$

STEP 2 Calculate x: (Solution provided for a stroke of 2.85 m, E_r = 69882 joules)

$$x = [(R_u + 550)/(7\sqrt{E_r})] - 1$$

$$x = [(2260 + 550)/(7\sqrt{69882})] - 1$$

$$x = [(2810)/(7)(264.3)] - 1 = 0.518$$

STEP 3 Calculate N_{qm}:

$$N_{qm} = 10(10^{X}) = 10(10^{0.518}) = 10(3.29)$$

$$= 33 \text{ blows / 250 mm}$$

Where: N_{qm} = Number of hammer blows for 250 mm penetration.
R_u = Ultimate pile capacity (kN).
E_r = Manufacturer's rated energy at field stroke (joules).

Solutions for other field observed hammer strokes and corresponding rated hammer energies are provided in table on following page.

Group Number	Field Observed Stroke (m)	Manufacturer's Rated Energy (joules)	Exponent (x)	Required Driving Resistance (blows / 250 mm)
1	1.50	36,870	1.091	123
2	1.65	40,458	0.996	99
3	1.80	44,136	0.911	81
4	1.95	47,814	0.836	69
5	2.10	51,492	0.769	59
6	2.25	55,170	0.709	51
7	2.40	58,848	0.655	45
8	2.55	62,526	0.605	40
9	2.70	66,204	0.561	36
10	2.85	69,882	0.518	33

STUDENT EXERCISE #11 SOLUTION - WAVE EQUATION HAMMER APPROVAL

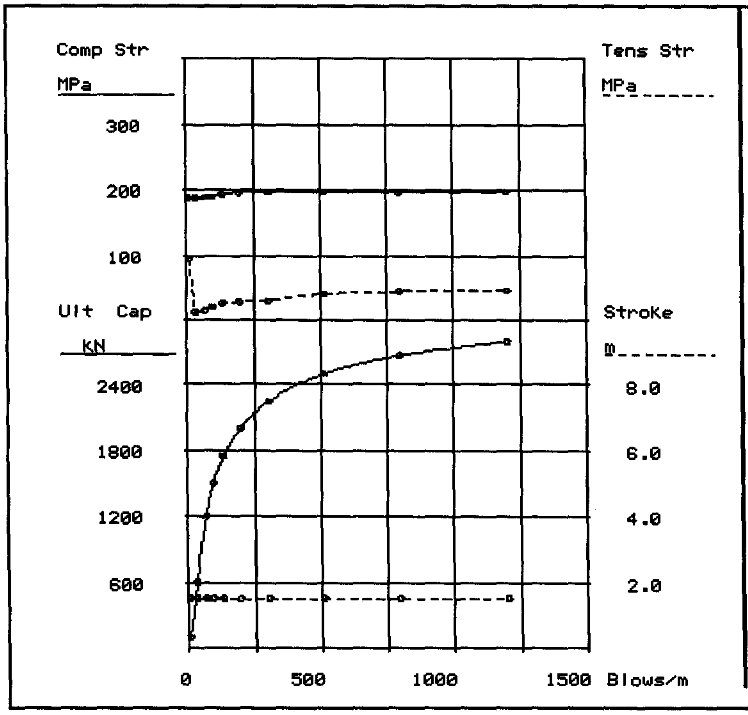
The wave equation results for the Vulcan 510 driving system indicate that a driving resistance of 797 blows per meter is required for the ultimate pile capacity. Maximum compression driving stresses are 197 MPa Based on these results, the compression driving stresses are below the maximum allowable of 279 MPa for Grade 3 steel but the driving resistance is greater than the recommended ranged of 120 to 480 blows per meter. Therefore, this hammer should not be approved.

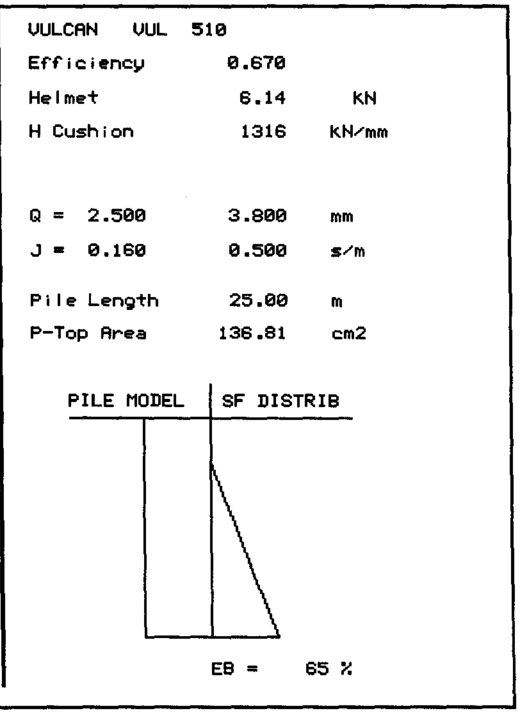
The wave equation results for the IHC S-70 driving system indicate that a driving resistance of 328 blows per meter is required for the ultimate pile capacity. Maximum compression driving stresses are 273 MPa Based on these results, the driving stresses are high, but within acceptable limits, and the driving resistance is within the recommended ranged of 120 to 480 blows per meter. The IHC hammer equivalent stroke could be slightly reduced, if necessary, to further decrease compression driving stress levels. This would increase the driving resistance but since the driving resistance is well below the maximum value this should not be a problem. An additional wave equation analysis should be performed if a reduced equivalent stroke will be used. Based on the above analysis, the IHC S-70 hammer should be approved.

HAMMER APPROVAL - VUL 510 SUBMITTAL 95/09/22

Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.994-1

Rut (kN)	Bl Ct (bpm)	Stroke(eq.) (m)	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)
100.0	7.5	1.52	-94.84	(5, 12)	188.83	(7, 4)	38.1
600.0	33.1	1.52	-12.54	(12, 47)	189.12	(8, 5)	41.9
1200.0	67.8	1.52	-15.71	(12, 33)	189.70	(9, 5)	41.5
1500.0	95.8	1.52	-21.81	(13, 31)	190.04	(8, 5)	40.6
1750.0	134.5	1.52	-25.95	(15, 45)	193.46	(25, 8)	39.9
2000.0	199.3	1.52	-28.52	(14, 30)	197.31	(25, 8)	39.7
2250.0	306.7	1.52	-30.44	(10, 41)	198.28	(25, 8)	39.7
2500.0	511.9	1.52	-40.46	(11, 40)	197.83	(25, 8)	39.6
2670.0	796.8	1.52	-44.19	(11, 40)	197.03	(25, 8)	39.6
2800.0	1202.6	1.52	-46.29	(11, 39)	198.47	(2, 13)	39.6

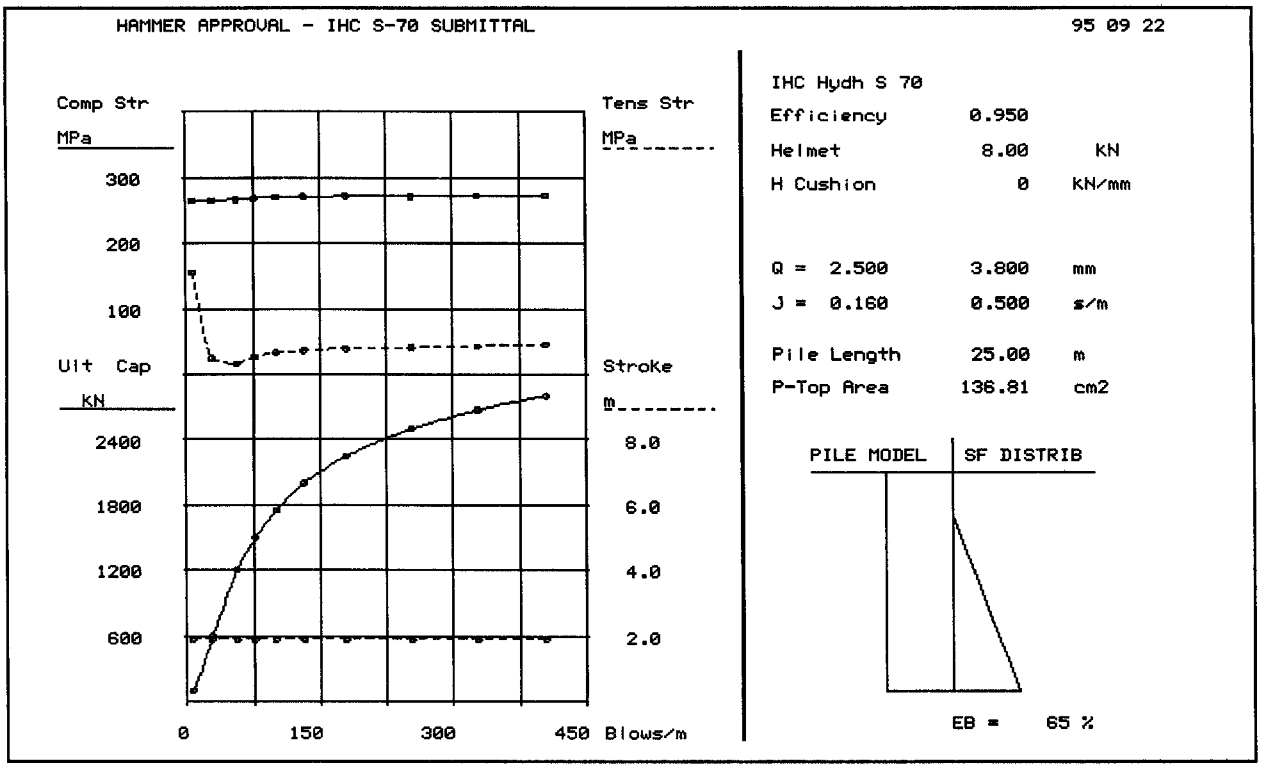




HAMMER APPROVAL - IHC S-70 SUBMITTAL 95/09/22

Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.994-1

Rut (kN)	Bl Ct (bpm)	Stroke(eq.) (m)	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)
100.0	6.5	1.90	-155.26(	6, 11)	265.77(	3, 3)	51.1
600.0	29.2	1.90	-25.68(	6, 47)	265.77(	3, 3)	55.0
1200.0	56.7	1.90	-17.12(	13, 32)	267.25(	4, 3)	54.9
1500.0	77.1	1.90	-27.35(	13, 29)	269.03(	7, 3)	54.0
1750.0	100.8	1.90	-33.58(	13, 28)	270.15(	7, 3)	53.5
2000.0	132.1	1.90	-37.35(	14, 28)	271.23(	7, 3)	53.2
2250.0	179.7	1.90	-39.84(	14, 28)	271.79(	7, 3)	53.2
2500.0	253.7	1.90	-41.38(	14, 27)	272.41(	8, 4)	53.1
2670.0	328.1	1.90	-43.87(	14, 26)	273.02(	7, 3)	53.1
2800.0	405.1	1.90	-46.17(	14, 26)	273.23(	7, 3)	53.1



STUDENT EXERCISE #12 SOLUTION - WAVE EQUATION INSPECTORS CHART

A contractor has chosen a Kobe K-35 for foundation installation of HP 360x174 H-piles. The H-piles are to be driven to a limestone bedrock for an ultimate pile capacity of 3250 kN. The H-piles are to be A-36 steel.

For hammer approval, a standard wave equation bearing graph analysis was performed. The results from this analysis are the next page and indicate that both the driving resistance (Chapter 12) and driving stresses (Chapter 11) are within specification limits for the ultimate capacity of 3250 kN. The standard bearing graph indicates a driving resistance of 255 blows per meter at a hammer stroke of 2.40 m should result in the required ultimate pile capacity.

A constant capacity wave equation analysis or inspectors chart was then performed to assist field personnel in the determining the required driving resistance at other field observed hammer strokes. The results of this constant capacity analysis for Pier 2 piles is presented on page 17-69. The analysis results have been furnished to the inspector in expanded form as presented on page 17-70 and should be used to answer the following questions.

1. Pile #1 has a field observed hammer stroke is 2.20 m and a driving resistance of 275 blows/m. Does this pile have the required ultimate capacity?

    No, at 2.2 m stroke a driving resistance of 304 blows/m is required for 3,250 kN.

    Any additional action required by the inspector?

    Yes, drive pile further.

2. Pile #2 has a field observed hammer stroke of 2.85 m and a driving resistance of 195 blows/m. Does this pile have the required ultimate capacity?

    Yes.

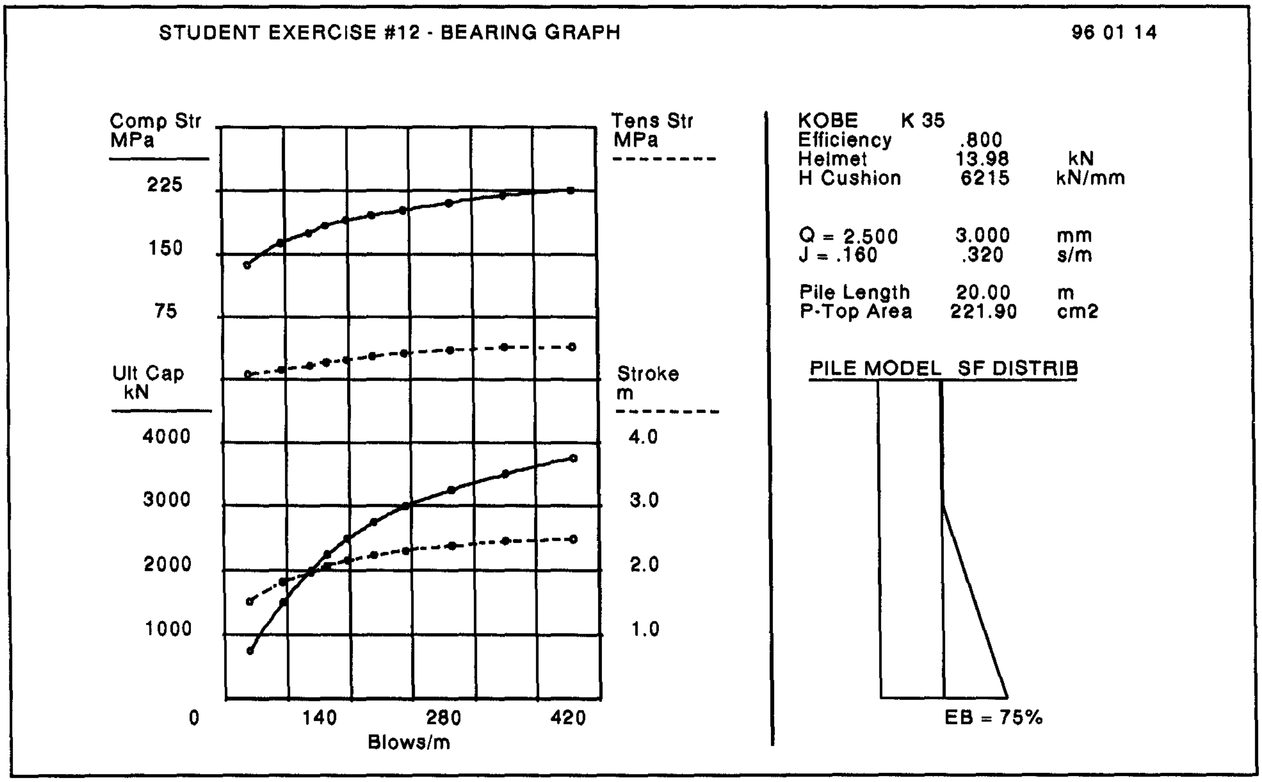
    Any additional action required by the inspector?

    Yes, driving stress are greater than 235 MPa which are too high since they are greater than 223 MPa (0.90 f_y). Drive piles with a reduced hammer stroke.

STUDENT EXERCISE #12 - BEARING GRAPH 96/01/14

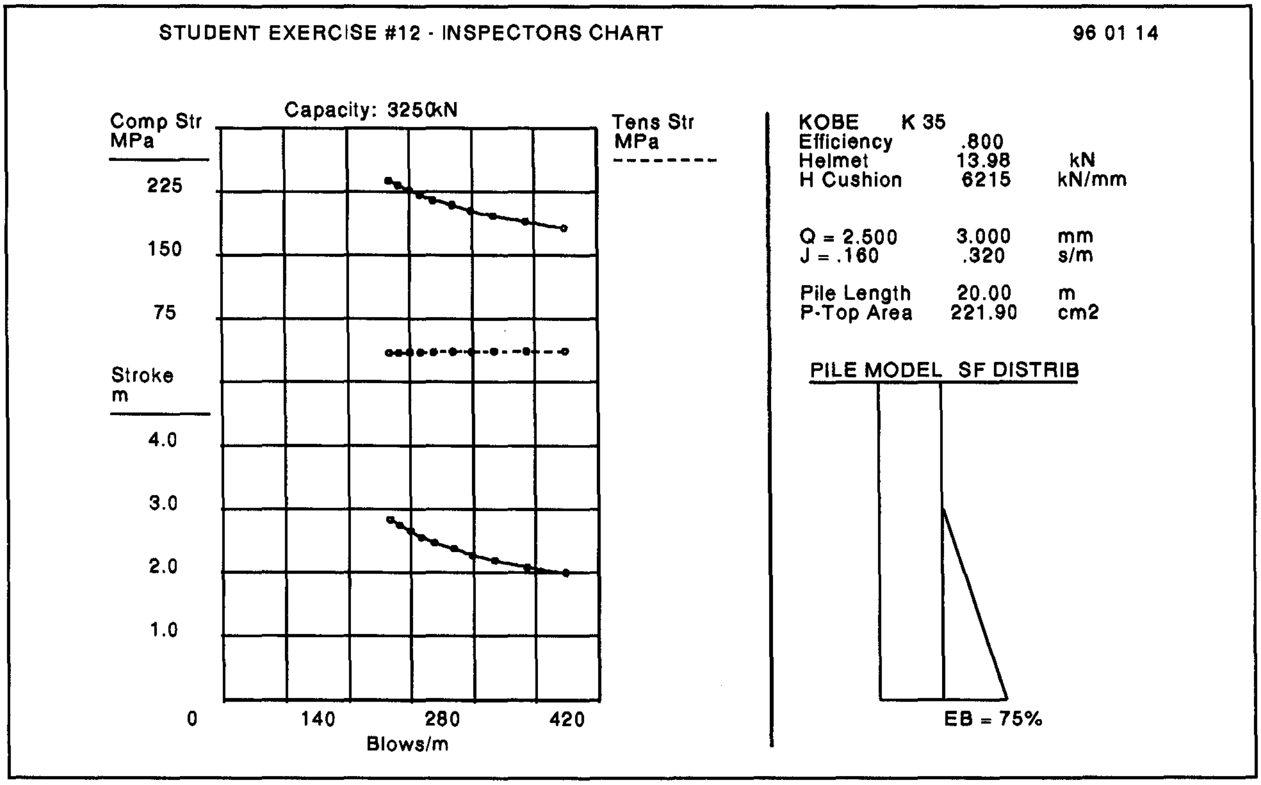
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.995-1

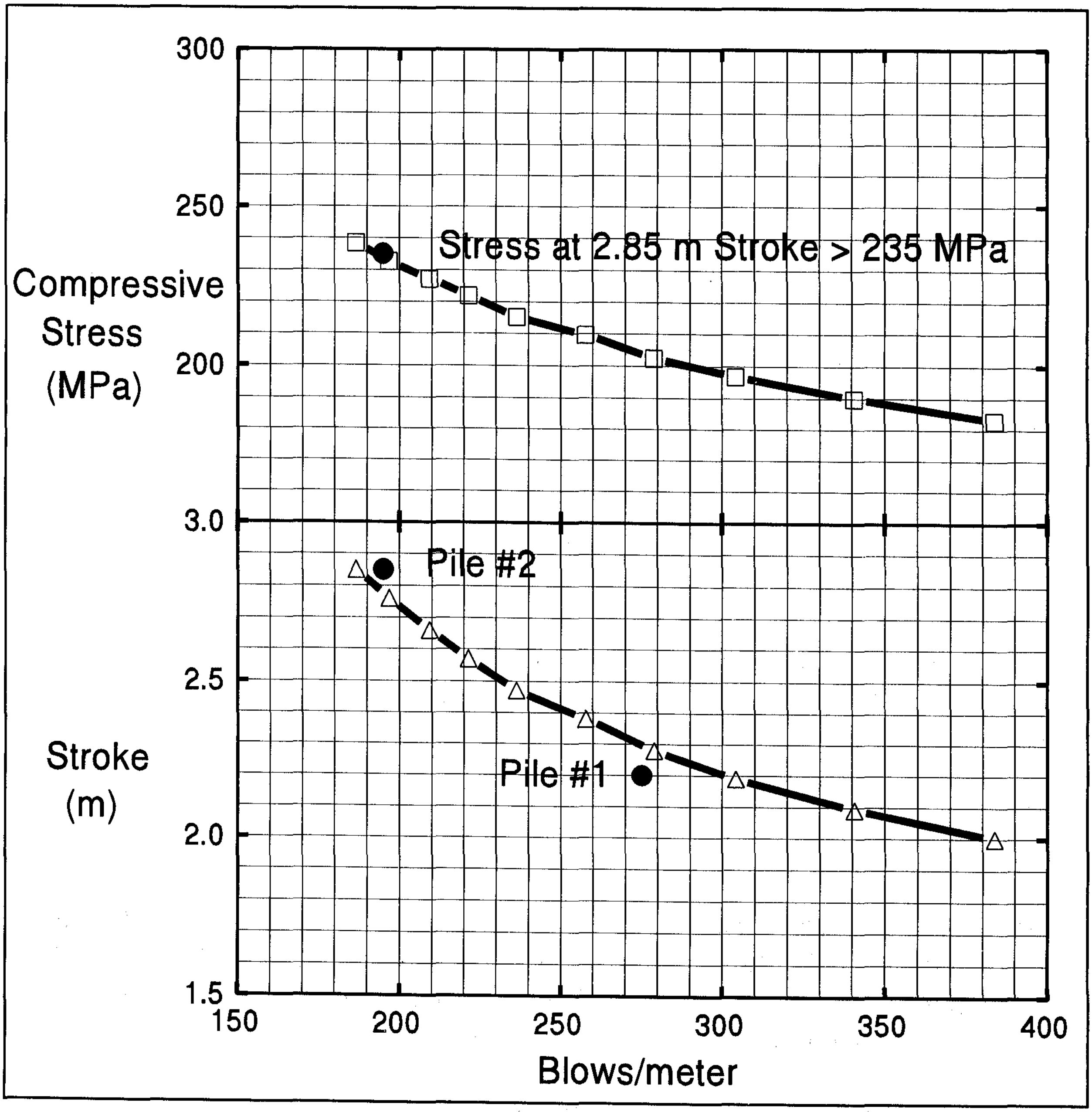
Rut (kN)	Bl Ct (bpm)	Stroke down	(m) up	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)	Bl Rt (b/min)
750.0	27.5	1.52	1.54	-7.39	(9, 46)	137.40	(4, 3)	44.7	52.7
1500.0	65.2	1.82	1.83	-12.30	(10, 30)	163.13	(10, 4)	41.7	48.2
2000.0	96.8	1.97	1.99	-17.55	(11, 27)	175.50	(10, 4)	41.9	46.2
2250.0	114.8	2.08	2.07	-21.69	(12, 26)	183.43	(10, 4)	43.2	45.2
2500.0	138.5	2.16	2.16	-24.44	(12, 24)	189.75	(11, 4)	44.1	44.3
2750.0	167.9	2.24	2.24	-28.96	(11, 23)	195.82	(11, 4)	45.1	43.6
3000.0	203.5	2.32	2.32	-32.91	(11, 23)	201.83	(10, 4)	46.2	42.8
3250.0	255.1	2.39	2.40	-36.08	(11, 22)	210.55	(20, 6)	47.0	42.2
3500.0	315.8	2.46	2.46	-39.09	(11, 22)	219.45	(20, 6)	48.2	41.6
3750.0	392.0	2.49	2.51	-40.49	(10, 22)	225.05	(20, 6)	48.9	41.3

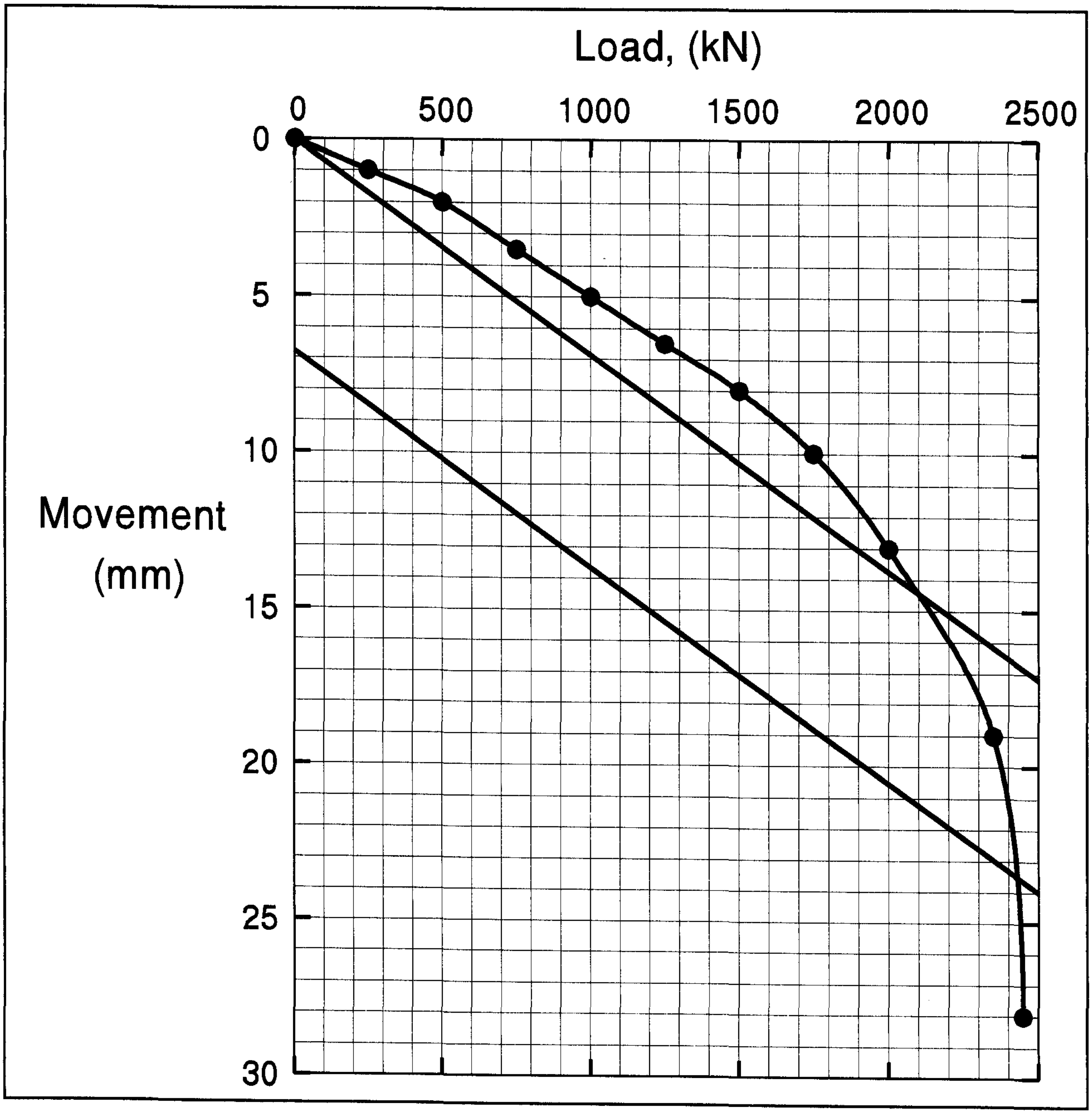


STUDENT EXERCISE #12 - INSPECTORS CHART 96/01/14
Goble Rausche Likins & Associates, Inc. GRLWEAP(TM) Version 1.995-1

Rut (kN)	Bl Ct (bpm)	Stroke down	(m) up	min Str (MPa)	i,t	max Str (MPa)	i,t	ENTHRU (kJ)	Bl Rt (b/min)
3250.0	383.7	2.00	2.40	-37.17	(11, 22)	182.91	(20, 6)	39.2	44.0
3250.0	340.7	2.09	2.40	-36.94	(11, 22)	189.80	(20, 6)	41.1	43.5
3250.0	304.2	2.19	2.39	-36.76	(11, 22)	196.77	(20, 6)	43.1	43.1
3250.0	278.9	2.28	2.39	-36.44	(11, 22)	202.57	(20, 6)	45.0	42.6
3250.0	257.7	2.38	2.39	-36.13	(11, 22)	209.78	(20, 6)	46.8	42.2
3250.0	236.4	2.47	2.39	-35.92	(11, 22)	215.22	(20, 6)	48.8	41.8
3250.0	221.3	2.57	2.39	-35.61	(11, 22)	221.89	(20, 6)	50.6	41.4
3250.0	209.4	2.66	2.40	-35.26	(11, 22)	227.07	(20, 6)	52.3	41.0
3250.0	196.8	2.76	2.40	-35.01	(11, 23)	232.65	(20, 6)	54.2	40.7
3250.0	186.6	2.85	2.40	-34.69	(11, 23)	238.40	(20, 6)	56.0	40.3




Compressive Stress (MPa)
300
250
200
Stress at 2.85 m Stroke > 235 MPa
3.0
2.5
2.0
1.5
Stroke (m)
Pile #2
Pile #1
150
200
250
300
350
400
Blows/meter


Load, (kN)
0
500
1000
1500
2000
2500
Movement
(mm)
0
5
10
15
20
25
30

STUDENT EXERCISE #14 SOLUTION - EQUIPMENT SUBMITTAL REVIEW

Project specifications require the contractor to use a pile driving hammer having a minimum rated energy of 20.0 kJ to install the 20 m long, 305 mm square, prestressed concrete piles on this project. The piles have a required ultimate pile capacity of 1200 kN. Soil conditions consist of 15 m of soft clay over 20 meters of medium dense to dense sands. Static analyses indicate the piles should develop the required ultimate capacity at a penetration depth of 19 m. The Gates dynamic formula will be used for construction control.

The following pages contain the contractor's submittal package on this project. Based on the submittal, the final driving resistance required by the Gates formula is 56 blows per 0.25 m for the 1200 kN ultimate capacity. Review the submittal information and decide if the submittal should be approved. Do you have any questions or concerns ?

STEP 1 Check if hammer meets minimum energy requirements.

Yes, the rated energy of 20.5 kJ for the Vulcan 50-C is greater than the 20.0 kJ required.

STEP 2 Determine line pressure loss in air hose between compressor and hammer by entering hose detail table on page E-20 at compressor air delivery of 28 m^3/min. (Note, this table indicates the line loss in 15.2 m of hose.)

At 28 m^3 / min and a line pressure of 827 kPa the expected pressure loss in the hose is 18.6 kPa per 15.2 m. Therefore for 61 m of hose, the pressure loss is (61m / 15.2 m)(18.6 kPa) or 74.6 kPa. The actual pressure at the hammer is then 827 kPa - 74.6 kPa or 752.3 kPa.

STEP 3 Check if the pressure at the hammer meets manufacturer's requirements.

No, the required pressure at the hammer is 827 kPa in order to develop the full rated energy.

STEP 4 Determine the rated energy based on the pressure at the hammer using the following manufacturer's formula for a differential hammer:

$$E_r = [W + A_{np} (p_h)]h$$

Based on the pressure at the hammer, the rated energy is:

$$\begin{aligned} E_r &= [22.25 \text{ kN} + 0.036 \text{ m}^2 (752.3 \text{ kPa})] \ 0.39 \text{ m} \\ &= [22.25 \text{ kN} + 27.08 \text{ kN}] \ 0.39 \text{ m} = 19.2 \text{ kJ} \end{aligned}$$

Note: At this rated energy, the Gates formula would require 64 blows / 0.25 m for the 1200 kN ultimate pile capacity. In addition more than half the rated energy is due to the pressure at the hammer.

Equipment Submittal

Hammer:	Vulcan 50-C differential acting air hammer. Rated energy = 20.5 kJ at 0.39 m stroke. (additional hammer details on page 22-49)
Hammer Cushion:	152 mm of Aluminum and Micarta. Hammer Cushion Area = 641 cm^2.
Helmet:	4.6 kN
Pile Cushion:	100 mm of Plywood. Pile Cushion Area = 930 cm^2.
Air Compressor:	Model 1000 Rated Delivery: 28.3 m^3 / min. Rated Pressure: 827 kPa.
Hose:	61 m of 51 mm I.D. (additional details on page 22-49).
Pile:	20 m long, 305 mm square precast, prestressed concrete Compressive Strength: 40 MPa. Effective Prestress after losses: 6 MPa.

Equipment Submittal

Hammer Details:

Ram Weight: 22.25 kN
Normal Stroke: 0.39 m
Rated Operating Pressure at Hammer: 827 kPa
Air Consumption: 24.9 m^3 / min
Required Air Compressor Size: 25.5 m^3 / min
Net Area of Piston: 0.036 m^2

Hose Details:

Hose		Pressure Loss in Hose (kPa)						
Inside Dia. (mm)	Length (m)	Air Delivery (m^3 / min)	Line Pressure (kPa) 414	552	690	827	1034	1378
51	15.2	16.8	13.1	------	------	------	------	------
		22.4	22.1	17.2	14.5	------	------	------
		28.0	34.5	26.9	22.1	18.6	12.2	11.7
		33.6	48.3	37.9	31.0	26.2	21.4	16.5
		39.2	64.1	51.0	42.1	35.9	29.0	22.1
		44.8	------	66.2	54.5	46.2	37.9	29.0
		50.4	------	83.4	69.3	57.9	47.6	36.5
		56.0	------	------	84.1	71.7	58.6	44.8

STUDENT EXERCISE #15 SOLUTION - HAMMER INSPECTION

You are inspecting the pile driving operations on two bridge projects. On the first project, Bridge #1, the contractor is using a single acting diesel hammer. The driving criteria with this hammer has been established as follows:

Minimum Toe Elevation:	EL 96.5 m
Minimum Driving Resistance:	80 blows / 250 mm at a 3.0 m stroke.

The driving record for the first pile driven is attached. The hammer operating speed was timed at 40 blows per minute at final driving. Has this pile met the driving criteria ?

STEP 1. Calculate the hammer stroke based on the recorded hammer operating speed using the formula on page 24-22.

$$\text{Stroke, } h = [4400/\{BPM^2\}] - 0.09$$
$$= [4400/\{40^2\}] - 0.09 = 2.66 \text{ m}$$

STEP 2. Determine the pile toe elevation.

Toe elevation = reference elevation - pile penetration depth
= 109.5 - 13.5 = 96.0 m

STEP 3. Based on hammer stroke, driving resistance and pile toe elevation, determine if the pile has met the driving criteria.

The pile has met the required driving resistance and toe elevation. However, the stroke is less than required. Therefore, the pile has not met the driving criteria, so continue driving.

PILE DRIVING LOG

STATE PROJECT NO.: Bridge #1 DATE: 5-29-98

JOB LOCATION: Bogalusa

PILE TYPE: 457 mm PCC LENGTH: 15 m BENT/PIER NO.: 1 PILE NO.: 1

HAMMER: D-30-32 ENERGY/BLOW: 99.9 kJ OPERATING RATE: 36-52 BPM HELMET WEIGHT: 14.5 kN

REF. ELEV.: 109.5 m PILE TOE ELEV.: PILE CUTOFF ELEV.: 108.3 m

PILE CUSHION THICKNESS AND MATERIAL: 190 mm of plywood

WEATHER: sunny TEMP.: 80° START TIME: 8:23 am STOP TIME: 8:58 am

METERS	BLOWS	STROKE / PRESSURE	REMARKS	METERS	BLOWS	STROKE / PRESSURE	REMARKS
0 - 0.25	W.O.P			8.00 - 8.25	25		
0.25 - 0.50	W.O.P			8.25 - 8.50	21	51 BPM	
0 50 - 0.75	W.O.P			8.50 - 8.75	23		
0.75 - 1.00	W.O.P			8.75 - 9.00	26		
1.00 - 1.25	W.O.P			9.00 - 9.25	22	51 BPM	
1.25 - 1.50	W.O.P			9.25 - 9.50	21		
1.50 - 1.75	W.O.H			9.50 - 9.75	23		
1.75 - 2.00	W.O.H			9.75 - 10.00	24	51 BPM	
2.00 - 2.25	W.O.H			10.00 - 10.25	22		
2.25 - 2.50	5		Fuel #2	10.25 - 10.50	26		
2.50 - 2.75	6	52 BPM		10.50 - 10.75	30	44 BPM	
2.75 - 3.00	8			10.75 - 11.00	34		
3.00 - 3.25	10			11.00 - 11.25	40		
3.25 - 3.50	12			11.25 - 11.50	51	43 BPM	
3.50 - 3.75	17	50 BPM		11.50 - 11.75	38	42 BPM	Fuel #4
3.75 - 4.00	22			11.75 - 12.00	41		
4.00 - 4.25	30	49 BPM		12.00 - 12.25	42	42 BPM	
4.25 - 4.50	21	47 BPM	Fuel #3	12.25 - 12.50	53		
4.50 - 4.75	24			12.50 - 12.75	58	41 BPM	
4.75 - 5.00	27			12.75 - 13.00	65		
5.00 - 5.25	29			13.00 - 13.25	77	40 BPM	
5.25 - 5.50	31	45 BPM		13.25 - 13.50	80	40 BPM	
5.50 - 5.75	32			13.50 - 13.75			
5.75 - 6.00	32			13.75 - 14.00			
6.00 - 6.25	35	45 BPM		14.00 - 14.25			
6.25 - 6.50	31			14.25 - 14.50			
6.50 - 6.75	25			14.50 - 14.75			
6.75 - 7.00	21	47 BPM		14.75 - 15.00			
7.00 - 7.25	18			15.00 - 15.25			
7.25 - 7.50	20			15.25 - 15.50			
7.50 - 7.75	19	51 BPM		15.50 - 15.75			
7.75 - 8.00	22			15.75 - 16.00			

On the second project, Bridge #2, the contractor is using a double acting diesel hammer. The bounce chamber - equivalent energy correlation for the hammer as provided by the contractor in the equipment submittal is attached. The driving criteria on the second project has been established as follows:

Minimum Toe Elevation:	EL 80
Minimum Driving Resistance:	60 blows / 250 mm at a bounce chamber pressure of 180 kPa. (Based on 15.2 m of hose)

The driving record for the first production pile driven on this project is attached. The hose between the bounce chamber pressure is 24.4 m long. Has this pile met the driving criteria?

STEP 1. Determine the equivalent hammer energy based on the bounce chamber pressure on the driving log.

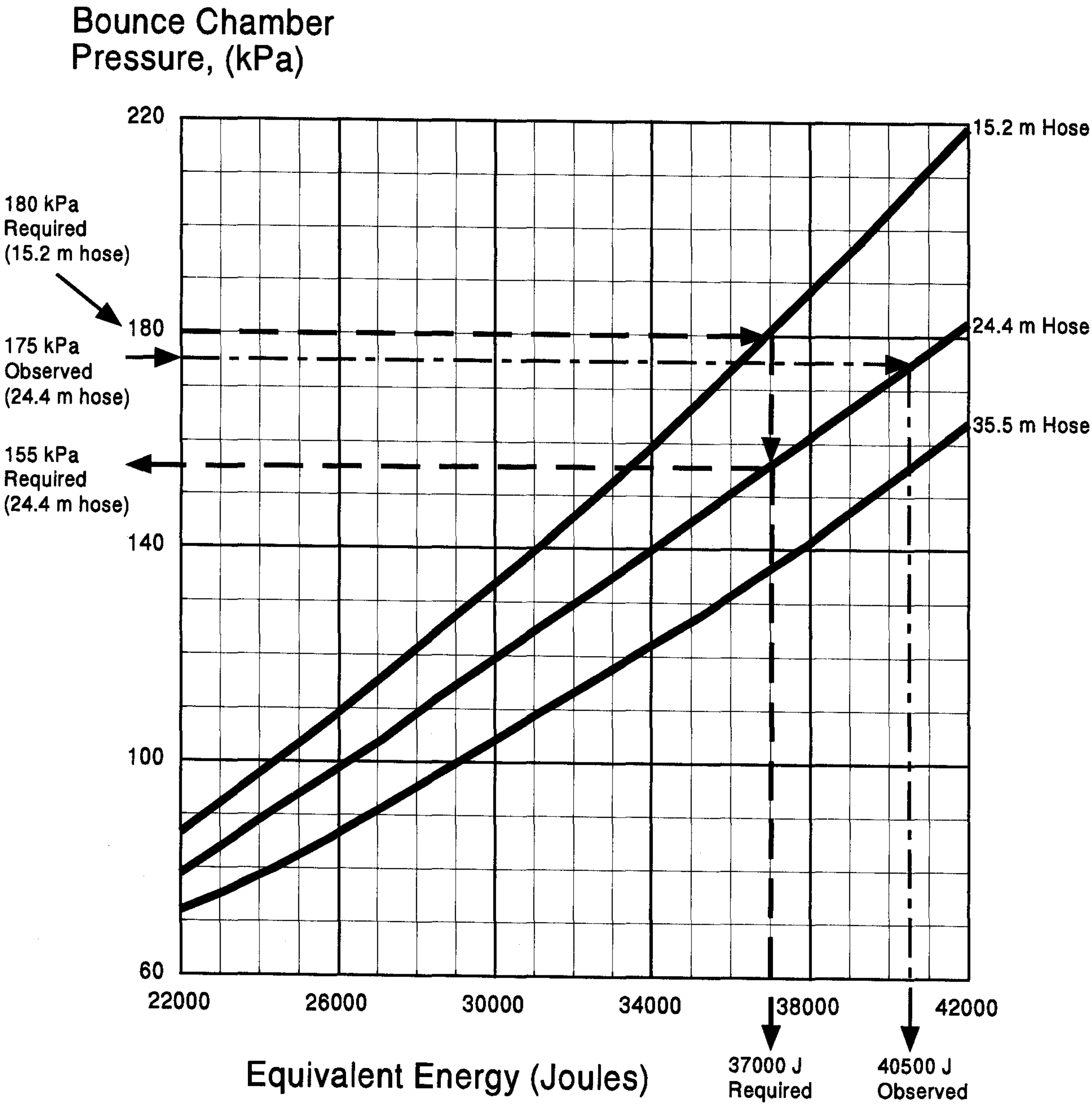
At a bounce chamber pressure of 175 kPa and a hose length of 24.4 m, the equivalent hammer energy is 40,500 joules.

STEP 2. Compare observed equivalent hammer energy with required energy.

The driving criteria required a bounce chamber pressure of 180 kPa with a 15.2 m hose. Hence, an equvalent hammer energy of 37,000 joules was needed with the 60 blows / 250 mm driving criteria.

STEP 3. Based on observed hammer energy, driving resistance and pile toe elevation, determine if the pile has met the driving criteria.

The hammer is delivering 40,500 joules and only 37,000 joules are required. For the 24.4 m hose length used, a bounce chamber pressure of only 155 kPa is needed for 37,000 joules. The minimum pile toe elevation of 80 was exceeded at a penetration depth of 11.25 m. The required final driving resistance of 65 blows / 250 mm also exceeds the required driving resistance of 60 blows / 250 mm. Therefore, this pile has more than met the driving criteria and has actually been overdriven.



Bounce Chamber
Pressure, (kPa)
220
180
140
100
60
180 kPa
Required
(15.2 m hose)
175 kPa
Observed
(24.4 m hose)
155 kPa
Required
(24.4 m hose)
15.2 m Hose
24.4 m Hose
35.5 m Hose
22000
26000
30000
34000
38000
42000
Equivalent Energy (Joules)
37000 J
Required
40500 J
Observed

PILE DRIVING LOG

STATE PROJECT NO.: Bridge #2 DATE: 5-29-98

JOB LOCATION: Hoboken

PILE TYPE: 324 mm CEP LENGTH: 15.5 m BENT/PIER NO.: 4 PILE NO.: 1

HAMMER: LB 520 ENERGY/BLOW: 35.7 kJ OPERATING RATE: 80-84 BPM HELMET WEIGHT: 8.9 kN

REF. ELEV.: 91.25 PILE TOE ELEV.: ______ PILE CUTOFF ELEV.: 94.1 m

PILE CUSHION THICKNESS AND MATERIAL: none

WEATHER: cloudy TEMP.: 75° START TIME: 10:52 STOP TIME: 11:09

METERS	BLOWS	STROKE / PRESSURE	REMARKS	METERS	BLOWS	STROKE / PRESSURE	REMARKS
0 - 0.25	W.O.H.		24.4 m hose	8.00 - 8.25	38		
0.25 - 0.50	W.O.H.			8.25 - 8.50	37	BCP 160	
0 50 - 0.75	W.O.H.			8.50 - 8.75	39		
0.75 - 1.00	W.O.H.			8.75 - 9.00	41		
1.00 - 1.25	3			9.00 - 9.25	40		
1.25 - 1.50	5			9.25 - 9.50	39	BCP 160	
1.50 - 1.75	6			9.50 - 9.75	42		
1.75 - 2.00	5			9.75 - 10.00	41		
2.00 - 2.25	6			10.00 - 10.25	44	BCP 160	
2.25 - 2.50	4	BCP 110		10.25 - 10.50	50		
2.50 - 2.75	5			10.50 - 10.75	51		
2.75 - 3.00	6			10.75 - 11.00	53	BCP 165	
3.00 - 3.25	8	BCP 115		11.00 - 11.25	51		min pen
3.25 - 3.50	10			11.25 - 11.50	54		
3.50 - 3.75	12			11.50 - 11.75	55	BCP 170	
3.75 - 4.00	20	BCP 125		11.75 - 12.00	57		
4.00 - 4.25	22			12.00 - 12.25	58	BCP 170	
4.25 - 4.50	21			12.25 - 12.50	60		
4.50 - 4.75	20			12.50 - 12.75	65	BCP 175	
4.75 - 5.00	23	BCP 135		12.75 - 13.00			
5.00 - 5.25	21			13.00 - 13.25			
5.25 - 5.50	25			13.25 - 13.50			
5.50 - 5.75	28	BCP 150		13.50 - 13.75			
5.75 - 6.00	30			13.75 - 14.00			
6.00 - 6.25	33			14.00 - 14.25			
6.25 - 6.50	32	BCP 155		14.25 - 14.50			
6.50 - 6.75	33			14.50 - 14.75			
6.75 - 7.00	35			14.75 - 15.00			
7.00 - 7.25	33	BCP 155		15.00 - 15.25			
7.25 - 7.50	37			15.25 - 15.50			
7.50 - 7.75	36			15.50 - 15.75			
7.75 - 8.00	33	BCP 155		15.75 - 16.00			

STUDENT EXERCISE #16 SOLUTION - DETERMINING PILE TOE ELEVATIONS

Pile driving criteria often include obtaining a specified driving resistance in conjunction with a pile penetration requirements or pile toe elevation. For many land based driving situations determination of the pile toe elevation is a relatively straightforward task. For batter pile driving and pile installations over water, determination of the pile toe elevation can be more problematic.

The following pages contain pile installation illustrations where the reference elevation is given and the pile penetration shown. For each example calculate the final pile toe elevation and pile penetration depth.

16a. pile toe elevation = template elevation - length below reference
= 125.5 - 16.5 m = 109.0

pile penetration = ground elevation - pile toe elevation
= 124.25 - 109.0 = 15.25 m

16b. pile toe elevation = template elevation - length below reference
= 15.25 - 20.75 m = -5.5

pile penetration = ground elevation - pile toe elevation
= 14.0 - (-5.5) = 19.5 m

16c. corrected pile length = length below template (correction factor for 1H:4V)
= 15.25 (0.971) = 14.81 m

pile toe elevation = template elevation - length below reference
= 175.40 - 14.81 m = 160.59

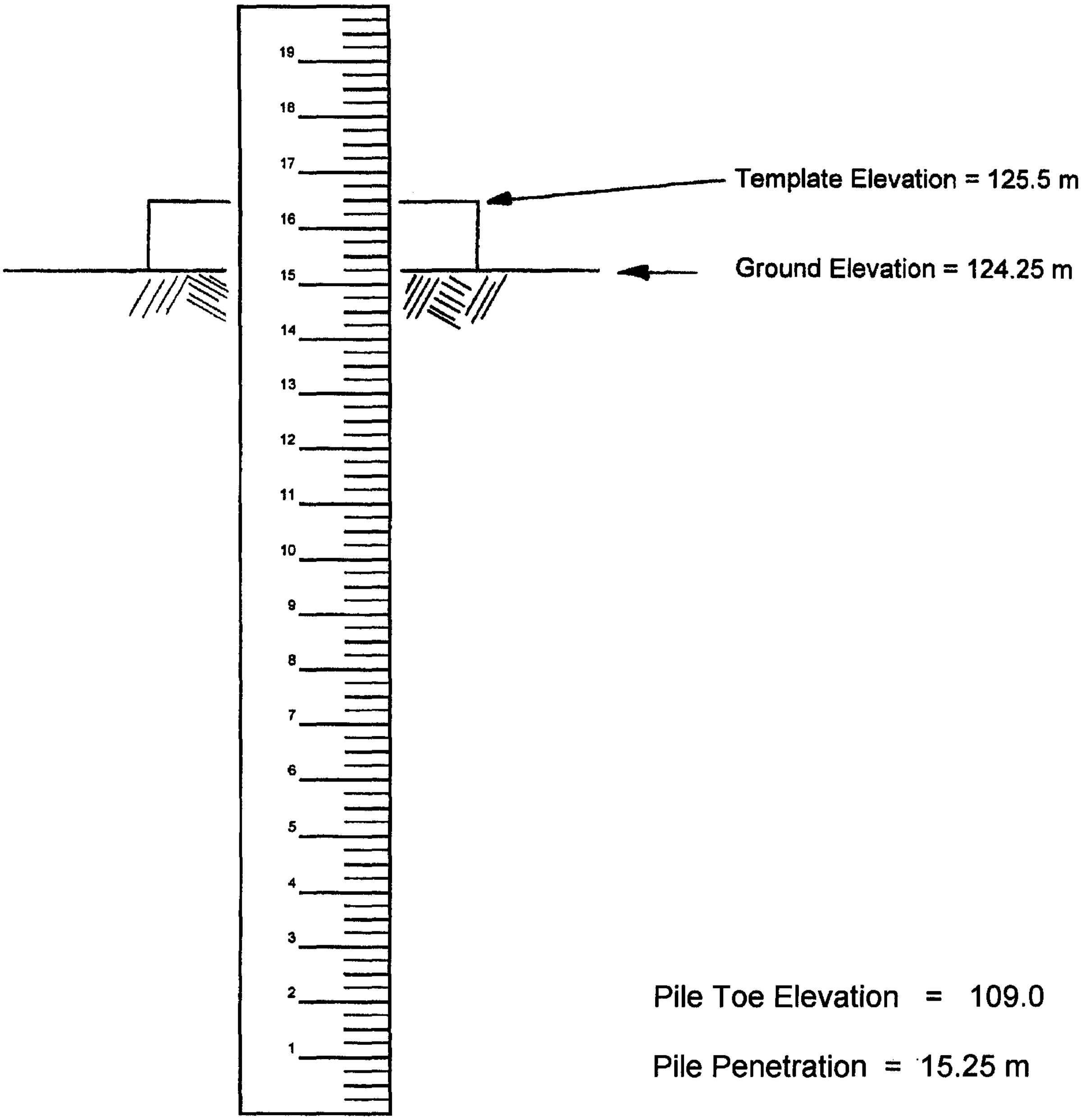
pile penetration = (ground elevation - pile toe elevation) / (correction factor for 1H:4V)

= (173.70 - 160.59) / (.971) = 13.50 m

		=	13.11 / (.971) = 13.5 m
16d.	pile toe elevation	=	template elevation - length below reference
		=	+1.3 - 18.75 m = -17.45
	pile penetration	=	mudline elevation - pile toe elevation
		=	-3.9 - (-17.45) = 13.55 m
	penetration below scour	=	scour elevation - pile toe elevation
		=	-9.8 - (-17.45) = 7.65 m

STUDENT EXERCISE 16a - DETERMINING PILE TOE ELEVATIONS

Land Pile Installation

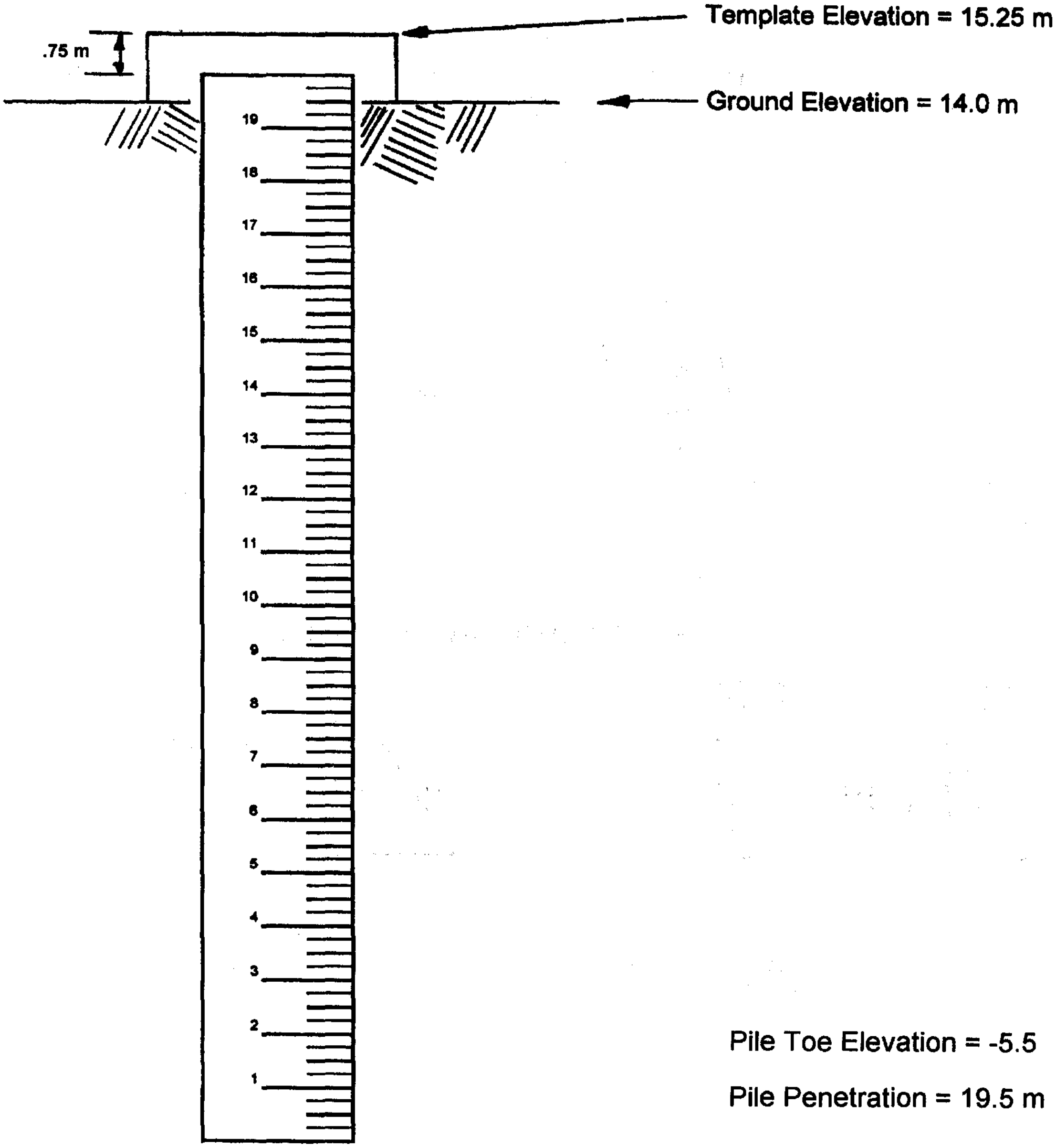


Pile Toe Elevation = 109.0

Pile Penetration = 15.25 m

STUDENT EXERCISE #16b - DETERMINING PILE TOE ELEVATIONS

Land Pile Installation

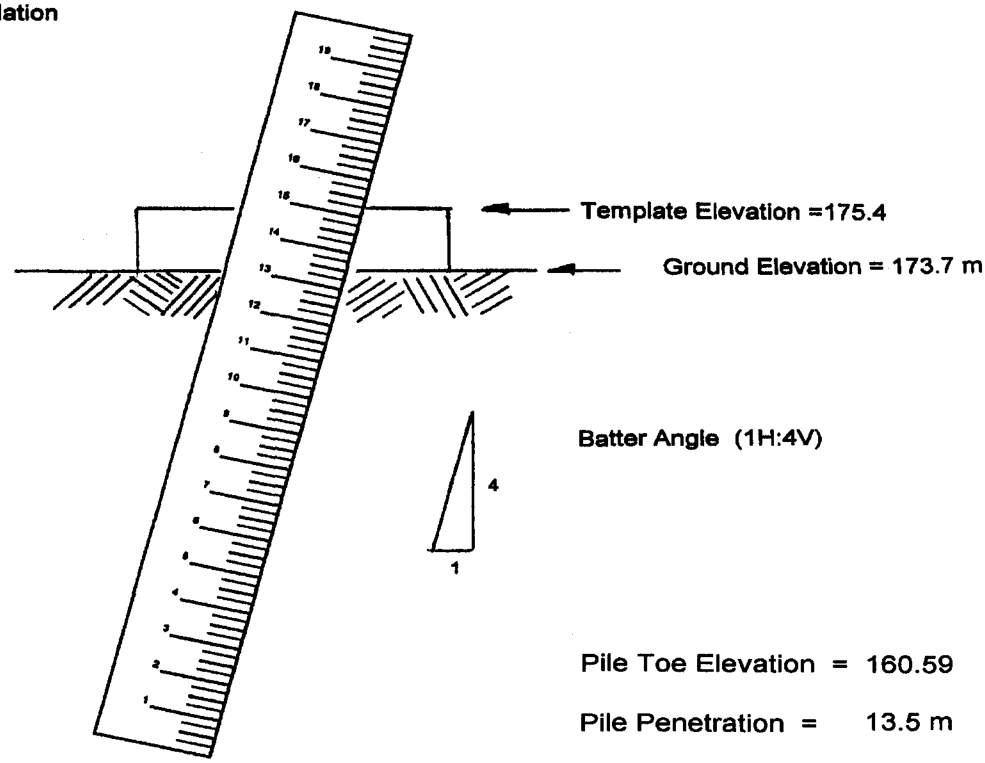


Pile Toe Elevation = -5.5

Pile Penetration = 19.5 m

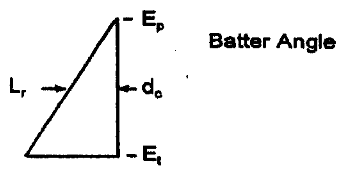
STUDENT EXERCISE #16c - DETERMINING PILE TOE ELEVATIONS

Batter Pile Installation



Calculating Pile Toe Elevation of Batter Piles

Batter Angle	Correction Factor, (B_c)
1H : 12V	.997
1.5H : 12V	.992
2H : 12V (1H : 6V)	.986
3H : 12V (1H : 4V)	.971
4H : 12V (1H : 3V)	.949
5H : 12V	.923



Definitions

L_r = Pile Length Below Reference Point (m)
E_p = Reference Point Elevation (m)
d_c = Corrected Pile Depth (m)
E_t = Pile Toe Elevation

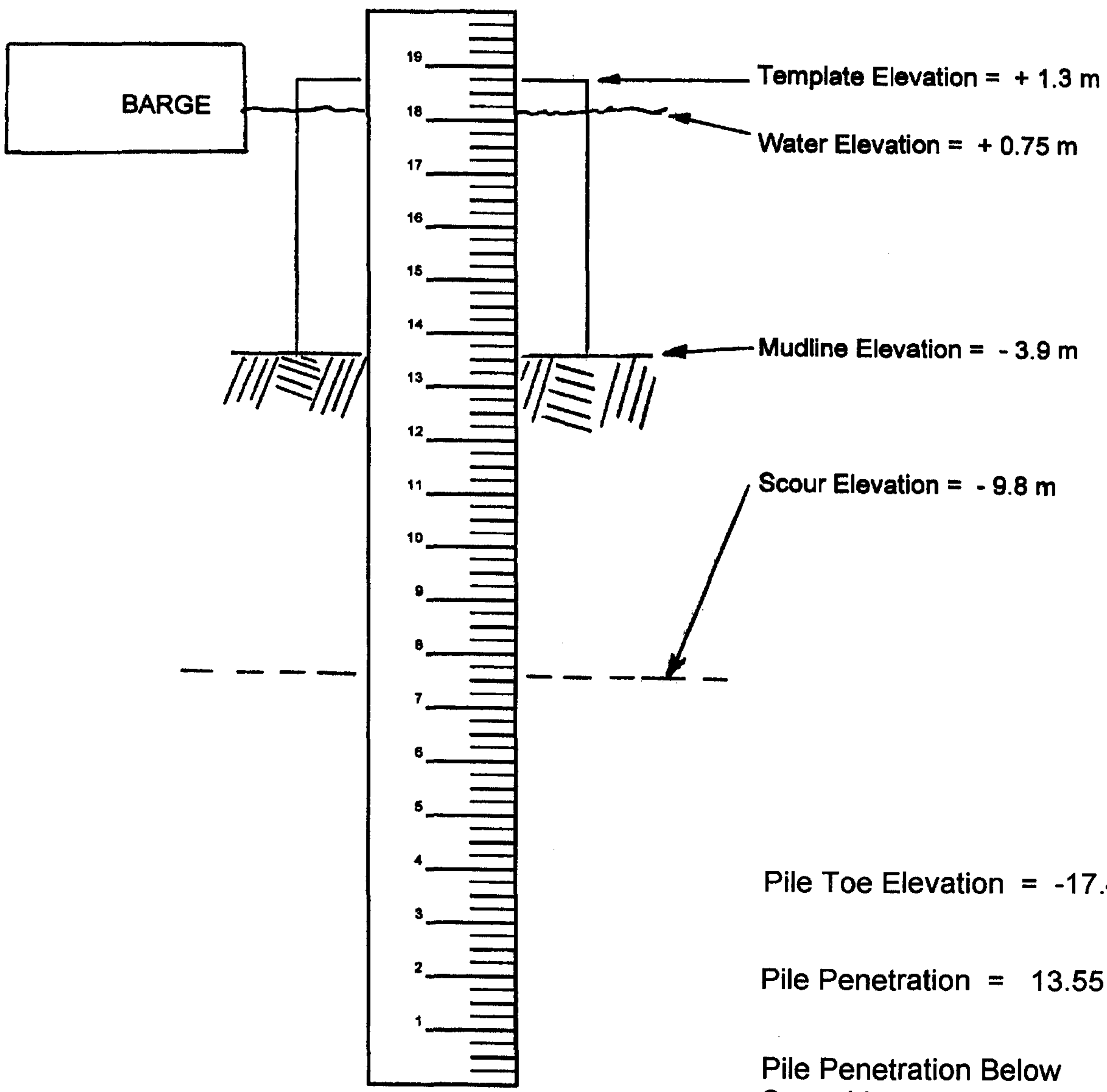
Formulas

$d_c = (L_r)(B_c)$

$E_t = E_p - d_c$

STUDENT EXERCISE #16d - DETERMINING PILE TOE ELEVATIONS

Pile Installation over Water



Pile Toe Elevation = -17.45

Pile Penetration = 13.55 m

Pile Penetration Below
Scour Line = 7.65 m