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Design and Construction of Driven Pile Foundations – Volume II

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and

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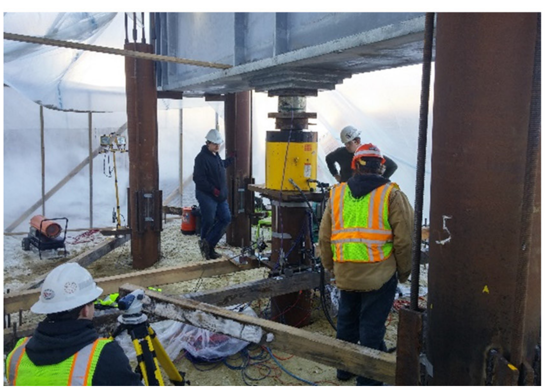

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NOTICE

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16. ABSTRACT

This document presents information on the analysis, design, and construction of driven pile foundations for highway structures. This document updates and replaces FHWA NHI-05-042 and FHWA NHI-05-043 as the primary FHWA guidance and reference document on driven pile foundations. The manual addresses design aspects including subsurface exploration, laboratory testing, pile selection, aspects of geotechnical and structural limit states, as well as technical specifications. Construction aspects including static load tests, dynamic tests, rapid load tests, wave equation analyses, dynamic formulas and development of driving criteria, as well as pile driving equipment, pile driving accessories, and monitoring of pile installation inspection are also covered. Step by step procedures are included for most analysis procedures and design examples.

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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When You Know	**Multiply By**	**To Find**	**When You Know**	**Multiply By**	**To Find**
(a) Length					
inch (in.)	25.4	millimeter (mm)	millimeter (mm)	0.039	inch (in.)
foot (ft)	0.305	meter (m)	meter (m)	3.28	foot (ft)
yard (yd)	0.914	meter (m)	meter (m)	1.09	yard (yd)
mile (mi)	1.61	kilometer (km)	kilometer (km)	0.621	mile (mi)
(b) Area					
square inches (in^2)	645.2	square millimeters (mm^2)	square millimeters (mm^2)	0.0016	square inches (in^2)
square feet (ft^2)	0.093	square meters (m^2)	square meters (m^2)	10.764	square feet (ft^2)
Acres (ac)	0.405	hectares (ha)	hectares (ha)	2.47	Acres (ac)
square miles (mi^2)	2.59	square kilometers (km^2)	square kilometers (km^2)	0.386	square miles (mi^2)
square inches (in^2)	645.2	square millimeters (mm^2)	square millimeters (mm^2)	0.0016	square inches (in^2)
(c) Volume					
fluid ounces (oz)	29.57	milliliters (mL)	milliliters (mL)	0.034	fluid ounces (oz)
Gallons (gal)	3.785	liters (L)	liters (L)	0.264	Gallons (gal)
cubic feet (ft^3)	0.028	cubic meters (m^3)	cubic meters (m^3)	35.32	cubic feet (ft^3)
cubic yards (yd^3)	0.765	cubic meters (m^3)	cubic meters (m^3)	1.308	cubic yards (yd^3)
(d) Mass					
ounces (oz)	28.35	grams (g)	grams (g)	0.035	ounces
pounds (lb)	0.454	kilograms (kg)	kilograms (kg)	2.205	pounds
short tons (2000 lb) (T)	0.907	megagrams (tonne) (Mg)	megagrams (tonne) (Mg)	1.102	short tons (2000 lb)
(e) Force					
pound (lb)	4.448	Newton (N)	Newton (N)	0.2248	pound (lb)
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot (psf)	47.88	Pascals (Pa)	Pascals (Pa)	0.021	pounds per square foot (psf)
pounds per square inch (psi)	6.895	kiloPascals (kPa)	kiloPascals (kPa)	0.145	pounds per square inch (psi)
(g) Density					
pounds per cubic foot (pcf)	16.019	kilograms per cubic meter (kgm^3)	kilograms per cubic meter (kgm^3)	0.0624	pounds per cubic feet (pcf)
(h) Temperature					
Fahrenheit temperature (^oF)	$5/9(^oF - 32)$	Celsius temperature (^oC)	Celsius temperature (^oC)	$9/5(^oC) + 32$	Fahrenheit temperature (^oF)

Notes:

1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second (s), Newton (N), and Pascal ($Pa=N/m^2$).
2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

PREFACE

The purpose of this manual is to provide updated, state-of-the-practice information for the design and construction of driven pile foundations in accordance with the Load and Resistance Factor Design (LRFD) platform. Engineers and contractors have been designing and installing pile foundations for many years. During the past three decades, the industry has experienced several major improvements including newer and more accurate methods of predicting and measuring geotechnical resistance, vast improvements in design software, highly specialized and sophisticated equipment for pile driving, and improved methods of construction control. Previous editions of the FHWA Design and Construction of Driven Pile Foundations manual were published 1985, 1996, and 2006 and chronical the many changes in design and construction practice over the past 30 years. This two volume edition, GEC-12, serves as the FHWA reference document for highway projects involving driven pile foundations.

Volume I, FHWA-NHI-16-009, covers the foundation selection process, site characterization, geotechnical design parameters and reporting, selection of pile type, geotechnical aspects of limit state design, and structural aspects of limits state design. Volume II, FHWA-NHI-16-010, addresses static load tests, dynamic testing and signal matching, rapid load testing, wave equation analysis, dynamic formulas, contract documents, pile driving equipment, pile accessories, driving criteria, and construction monitoring. Comprehensive design examples are presented in publication FHWA-NHI-16-064.

Throughout this manual, numerous references will be made to the names of software or technology that are proprietary to a specific manufacturer or vendor. Please note that the FHWA does not endorse or approve commercially available products, and is very sensitive to the perceptions of endorsement or preferred approval of commercially available products used in transportation applications. Our goal with this development is to provide recommended technical guidance for the safe design and construction of driven pile foundations that reflects the current state of practice and provides information on advances and innovations in the industry. To accomplish this, it is necessary to illustrate methods and procedures for design and construction of driven pile foundations. Where proprietary products are described in text or figures, it is only for this purpose.

The primary audience for this document is: agency and consulting engineers specialized in geotechnical and structural design of highway structures; engineering geologists and consulting engineers providing technical reviews, or who are engaged in the design, procurement, and construction of driven pile foundations This document is also intended for management, specification and contracting specialists, as well as for construction engineers interested in design and contracting aspects of driven pile systems.

This document draws material from the three earlier FHWA publications in this field; FHWA-DP-66-1 by Vanikar (1985), FHWA HI 97-013 and FHWA HI 97-014 by Hannigan et al. (1998), and FHWA NHI-05-042 and FHWA NHI-05-043 by Hannigan et al. (2006). Photographs without specific acknowledgement in this two volume document are from these previous editions, their associated training courses, or from the consulting practice of GRL Engineers, Inc.

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Mr. Jerry DiMaggio – Applied Research Associates, Inc.
Mr. Van Komurka – Wagner Komurka Geotechnical Group, Inc.
Mr. Billy Camp – S&ME, Inc.
Dr. Brian Anderson – Auburn University

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A	-	Pile cross sectional area (9.2) (9.5) (10.4) (10.5) (10.6) (12.8); Slope of tangent modulus line (9.5).
A_c	-	Cross sectional area of concrete (9.5).
a	-	Acceleration of pile (11.4).
a(t)	-	Acceleration measured at the gage location (10.4).
B	-	Y-intercept of the tangent modulus line (9.5).
b	-	Pile width or diameter (9.2);
bpm	-	Blow per minute (15.13) (18.7).
C	-	Wave speed of pile material (10.5) (10.6) (11.3) (12.8).
C_4	-	Damping constant for Statnamic test (11.4).
D	-	Pile width for WEAP quake calculation(12.5);
D_w	-	Wave length (11.3).
E	-	Elastic modulus of pile material (9.2) (9.5) (10.4) (10.5) (10.6) (12.8).
E_c	-	Elastic modulus of concrete (9.5).
E_d	-	Developed hammer energy (13.3).
E_p	-	Energy computed at the gage location (10.6).
E_{sm}	-	Secant modulus (9.5).
E_{st}	-	Elastic modulus of steel (9.5).
F	-	Axial force in plane of gage (9.5); Force (10.5).
$F(t)$	-	Force computed at gage location (10.4) (10.6).
F_a	-	Statnamic inertia load (11.4).
F_{eff}	-	Hammer efficiency factor (13.3).
F_{max}	-	Maximum measured force applied during testing (11.4).
F_p	-	Statnamic pore water pressure force (11.4).
F_{stn}	-	Statnamic induced load (11.4).
F_t	-	Force measured on pile head (11.4).
F_u	-	Statnamic static soil resistance force (11.4).
F_v	-	Statnamic dynamic soil resistance force (11.4).
f'_c	-	Concrete compressive strength at 28 days (14.2).
f_{pe}	-	Effective prestress in concrete (14.2).
g	-	Acceleration due to gravity (15.8).
h	-	Ram stroke (13.1) (15.8) (15.13) (18.7).
J	-	Dimensionless damping factor based on soil type (10.6).
KE	-	Kinetic Energy (15.8).

L - Total pile length (10.5) (10.6) (11.3);
Pile length below dial gages or gage location (9.2) (10.6);
Length of pile between two measuring points under no load condition (9.5).

M_t - Tangent modulus (9.5).

m - Mass of pile (11.4); Ram mass (15.8).

N_b - Pile penetration resistance with units of (blows/inch) (13.3).

N_{ft} - Pile penetration resistance with units of (blows/foot) (13.3).

N_t - Toe bearing capacity coefficient (12.7).

N_w - Wave number (11.3).

Q - Test load (9.2) (9.5).

Q_{avg} - Average load in the pile between two points (9.5).

Q_f - Failure load (9.2) (9.3).

RSP - Case method static nominal resistance (10.6).

RTL - Total static and dynamic resistance on pile during driving (10.6).

R_n - Nominal resistance (11.4) (13.1)

R_{ndr} - Nominal driving resistance (13.3).

R_p - Nominal toe resistance (9.5).

R_1 - Deflection reading at upper measurement location (9.5).

R_2 - Deflection reading at lower measurement location (9.5).

s_b - Set per blow (13.1); Permanent pile set (13.3).

s_f - Measured pile head movement at failure (9.2) (9.3).

t_L - Load duration (11.4).

t_{umax} - Time of maximum displacement for Statnamic test (11.4).

t_1 - Time of initial impact (10.6).

t_2 - Time of reflection of initial impact from pile toe (t_1+2L/C) (10.6).

t_4 - Time at beginning of Stage 4 for Statnamic test (11.4).

V - Velocity of pile (11.4).

$V(\mathrm{t})$ - Velocity measured at gage location (10.4).

V_i - Impact velocity (15.8).

V_o - Normalizing constant (11.4).

V_{static} - Velocity used to determine parameters α and β for Sheffield Method (11.4).

W - Ram weight (13.1).

$WD(t)$ - Downward traveling wave, wave down (10.6).

WD_m - Wave down, measured (10.6).

$WU(t)$ - Upward traveling wave, wave up (10.6).

WU_c - Wave up, computed (10.6).

WU_m - Wave up, measured (10.6).

α - Sheffield Method rate parameter (11.4).
β - Sheffield Method rate parameter (11.4).
ξ - Loading rate reduction factor (11.4).
Δ - Elastic deformation of pile (9.2).
$\Delta\varepsilon$ - Change of strain from one load increment to the next (9.5).
$\Delta\sigma$ - Change of stress from one load increment to the next (9.5).
ε - Strain measured in gage (9.5).
$\varepsilon(t)$ - Measured strain at time, t (10.4).
σ - Normal stress (pressure) on plane of failure, stress (9.5).
ϕ_{dyn} - Resistance factor (based on the construction control method) (13.2).

LIST OF ACRONYMNS

AASHTO	-	American Association of State Highway and Transportation Officials
ASTM	-	American Society for Testing and Materials
BL	-	Blast load
BOR	-	Beginning of Restrike
BR	-	Vehicular braking force
CD	-	Consolidated Drained triaxial test
CE	-	Vehicular centrifugal force
CED	-	Closed End Diesel hammer
CEP	-	Closed End Pipe
COR	-	Coefficient of Restitution
CPT	-	Cone Penetration Test
CPTu	-	Piezo Cone Penetration Test
CR	-	Force effects due to creep
CT	-	Vehicular collision force
CU	-	Consolidated Undrained triaxial test
CV	-	Vessel collision force
DA	-	Design Angular Distortion
DC	-	Dead load components and attachments
DD	-	Downdrag
DMT	-	Dilatometer test
DW	-	Wearing surface and utilities
DWT	-	Deadweight tonnage
EH	-	Horizontal earth pressure
EL	-	Locked-in stress
EOD	-	End of Drive
EQ	-	Earthquake load
ER	-	SPT hammer efficiency as determined by energy measurements
ES	-	Earth surcharge
EV	-	Vertical earth pressure
FHWA	-	Federal Highway Administration
FR	-	Friction load
I.D.	-	Inner diameter
IC	-	Ice load
IM	-	Vehicular dynamic load allowance
KE	-	Kinetic Energy

LL	-	Liquid Limit; Vehicular Live Load
LS	-	Live Load Surcharge
LVDT	-	Linear Variable Differential Transformer
MUP	-	Modified Unloading Point Method
NHI	-	National Highway Institute
O.D.	-	Outer Diameter
OED	-	Open Ended Diesel hammer
OEP	-	Open Ended Pipe
PE	-	Potential Energy
PGA	-	Peak Ground Acceleration coefficient
PI	-	Plasticity Index
PL	-	Plastic Limit; Pedestrian Live Load
PS	-	Secondary forces from post-tensioning
RSA	-	Residual Stress Analysis
SE	-	Force effect due to settlement
SH	-	Force effects due to shrinkage
SPT	-	Standard Penetration Test
SRD	-	Soil Resistance to Driving
SUP	-	Segmental Unloading Point Method
TG	-	Force effect due to temperature gradient
TU	-	Force effect due to uniform temperature
UPM	-	Unloading Point Method
UU	-	Unconsolidated Undrained triaxial test
VST	-	Vane shear test
WA	-	Water load and steam pressure
WD	-	Downward traveling wave, Wave Down
WEAP	-	Wave Equation Analysis Program
WL	-	Wind on live load
WS	-	Wind load on structure
WU	-	Upward traveling wave, Wave Up

CHAPTER 9

STATIC LOAD TESTING

9.1 GENERAL

Static load testing of piles is the most accurate method of determining load capacity. Depending upon the size of the project and other project variables, 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 static load testing, reference should be made to publications listed at the end of this chapter including the most recent ASTM test standards as well as the NYSDOT Static Pile Load Test Manual, GCP-18 (2007).

9.1.1 Reasons and Prerequisites for a Static Load Test Program

Static load testing provides the engineer with valuable design verification information on the nominal resistance and deformation response of test or production piles. Reasons to perform static load tests include the following:

1. Provide information for design verification or design refinement as well as information for construction verification or construction procedure modification based on measured load-deflection or measured load-transfer data.

2. Confirm the ability of the subsurface materials to provide the nominal geotechnical resistance.

3. Determine or calibrate resistance factors for new static design methods, local or regional geologic conditions, or dynamic or rapid load test analysis methods or procedures.

4. Determine p-y response of laterally loaded piles.

Static load test program development and oversight should be performed by an experienced foundation engineer. Static load tests are frequently used to establish a comparison baseline for design, testing, and construction techniques. The quality of static load tests is always important, but should be of utmost importance when performing research or updating current LRFD resistance factors. Poorly performed tests or analyses can result in unsafe or grossly uneconomical foundations.

In order to adequately plan and implement a static load testing program, the following information should first 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 parameters of geomaterials and identification of groundwater conditions.

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

9.1.2 Developing a Static Load Test Program

The goal and objectives of a static load test program should be clearly established, while the type and frequency of tests should be selected to provide the required knowledge for final design purposes or for construction verification. A significantly different level of effort and instrumentation is required if the goal of the load test program is simply to confirm the nominal geotechnical resistance or if detailed load-transfer information is desired for final design or improvement of design efficiency. 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 test pile(s) may be loaded to the geotechnical nominal resistance. A loading apparatus designed to load a pile to only to the anticipated nominal resistance is usually insufficient to obtain geotechnical failure. Hence, the true nominal geotechnical resistance cannot be determined, and the full benefit from performing a static load test to improve design efficiency is not realized. The jack capacity should be between 120% and 150% of the anticipated maximum load to be applied.

2. Specifications should require the use of a load cell and spherical bearing plate as well as linear variable displacement transducers (LVDT's) or dial gages with sufficient travel to allow accurate measurements of load and movement at the pile head. LVDT's or dial gages with insufficient travel that must be shimmed and reset during the test should be prohibited. 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 resistances. The load cell capacity should be between 120% and 150% of the anticipated maximum load to be applied.

3. The load test program should be supervised by a foundation engineer 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/energy setting, hammer cushion materials and dimensions, pile cushion material and cushion thickness when new and replaced, accurately determined final set, details on any installation aids used such as predrilling and their depths, start and ending driving times, stopping depths and associated time durations for splicing, equipment maintenance, etc., should be recorded.

5. Use of dynamic monitoring on the load test pile is recommended for estimates of the nominal resistance at the time of driving including the soil resistance distribution, evaluation of drive system performance, calculation of driving stresses, and refinement of wave equation input parameters.

9.1.3 When and Where to Load Test

Static load testing during either the design or construction phase should be performed to achieve a desired goal. This usually involves determining the nominal geotechnical resistance or load-transfer information at a representative site location. The number of load tests needed and their location should be selected based on the site variability. The AASHTO resistance factor for determination of the nominal resistance by static load testing is based on performing one load test per site condition. Site variability is discussed in Section 5.5.3 and in AASHTO (2014).

The following criteria, adapted from FHWA-SA-91-042 by Kyfor et al. (1992), summarize conditions when static pile load testing can be effectively utilized:

1. When substantial cost savings can be realized. This is often the case on large projects involving either friction piles (to determine that estimated pile lengths can be reduced) or end bearing piles (to determine that a higher nominal resistance can be used).

2. When the nominal geotechnical resistance is uncertain 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 nominal geotechnical resistance is contemplated relative to typical practice.

5. When significant time dependent changes in nominal resistance are anticipated as a result of soil setup or relaxation.

6. When a reliable assessment of axial tension resistance or lateral deflection is important.

7. Verification of new design or testing methods.

8. When new pile types, large diameter open end pipe piles, and/or pile installation procedures are utilized.

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

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

11. When developing LRFD calibration based on local and regional geologic conditions.

9.1.4 Effective Use of Load Test Information

9.1.4.1 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 disciplines is necessary. The following are the advantages of load testing during the design stage.

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

b. Confirm drivability to minimum penetration requirements and suitability of nominal geotechnical resistance at the estimated pile penetration depths.

c. Establishes preliminary driving criteria for production piles.

d. The availability of pile driving information to construction project bidders should reduce their bid "contingency."

e. Reduces potential for contract disputes related to pile driving problems.

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

9.1.4.2 Construction Stage

Load testing at the start of construction may be the only practical time for testing on some projects that cannot justify the cost of a design stage program. Construction stage static load tests are invaluable to confirm that the design loads are appropriate, to establish the final design lengths for production piles, and to assure that the pile installation procedure is satisfactory. Perhaps most importantly, these results can be used to refine the estimated pile lengths shown on the plans and establish minimum pile penetration requirements.

9.1.4.3 Load Test Databases

As mentioned in Paikowsky (2004) and Abu-Hejleh (2015), load test databases are needed for design method reliability calibrations and to improve the geotechnical design of production foundations. High quality, complete, load test databases offer the ability to compare design and construction verification methods with full scale load test results. The information may allow the increase of the resistance factor for a specific design method, or provide designers with preferable foundation alternatives and construction methods given similar site conditions.

One such study was performed in 2010 by the Oregon Department of Transportation (DOT), where they evaluated driven pile resistance factors based upon wave equation analyses (Smith et al. 2011). At that time, the AASHTO recommended resistance factor, φ_{dyn}, for wave equation analysis was 0.40. This resistance factor resulted in increased pile foundation costs within the LRFD framework. Therefore, the Oregon DOT performed a calibration study to assess the reliability of a higher resistance factor for wave equation analysis. After reviewing several databases, the study recommended a wave equation analysis resistance factor of 0.55 for initial driving and 0.40 for restrikes. As a result of this research and other efforts, the recommended resistance factor for wave equation analysis in the AASHTO (2010) design specifications was changed from φ_{dyn} =0.40 to φ_{dyn} =0.50.

The FHWA Deep Foundation Load Test Database (DFLTD) contains data from several state highway agencies, FHWA offices and international sources. Although it is no longer updated, the DFLTD is available upon request by contacting the FHWA and contains approximately 1300 load test results from 1985 to 2003. Additional load test databases have been developed by several state transportation agencies.

It is recommended that load test results and accompanying geotechnical and pile installation information be stored in databases. The respective report section of the ASTM standard for axial compression, axial tension, and lateral load tests identifies the complete site, installation, and load test details that should maintained in a database. Maintaining this information allows the database to be effective for development and calibration of new design or construction control methods.

9.1.5 Resistance Factors for Static Load Testing

Resistance factors applicable to the nominal axial geotechnical resistance determined from a static load test are provided in Table 9-1. The resistance factor varies depending on whether the load test is in axial compression or tension as well as whether dynamic testing is performed. A resistance factor is not provided for lateral load testing as this is generally controlled by deformation criteria in the service limit state rather than by the strength limit state.

Table 9-1 Resistance Factors Based on Static Load Testing of Driven Piles (modified from AASHTO 2014)

Condition	Resistance Determination Method	Resistance Factor
Nominal Axial Compression Resistance of Single Pile, ϕ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site condition, but no less than 2% of the production piles.	0.80
Nominal Axial Compression Resistance of Single Pile, ϕ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing.	0.75
Nominal Axial Tension Resistance of Single Pile, ϕ_{up}	Static load test.	0.60

9.2 AXIAL COMPRESSION LOAD TEST

Piles are most frequently tested in axial compression. However, they can also be tested in axial tension as well as for their lateral resistance. Figure 9-1 illustrates the basic mechanism of performing an axial compression 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. During the test, readings are recorded of the time, applied load, and pile head movement. Strain measurements at other points along the pile shaft can be obtained concurrently to yield load transfer details and unit shaft resistances.

3. A load movement curve is plotted.

4. The geotechnical nominal resistance and the movement at the nominal resistance are determined by one of the several methods of interpretation.

5. Movement is usually measured only at the pile head. However, the pile can also be instrumented with telltales to determine movement anywhere along the pile. Telltales (solid rods protected by tubes) are shown in Figure 9-1.

6. Telltales can be used to determine the strain between telltale locations. Strain gages, attached to, or embedded within the pile can be used to determine the strain at discrete locations along the pile length. Both of these strain measurements can be used to estimate load transfer along the pile shaft.

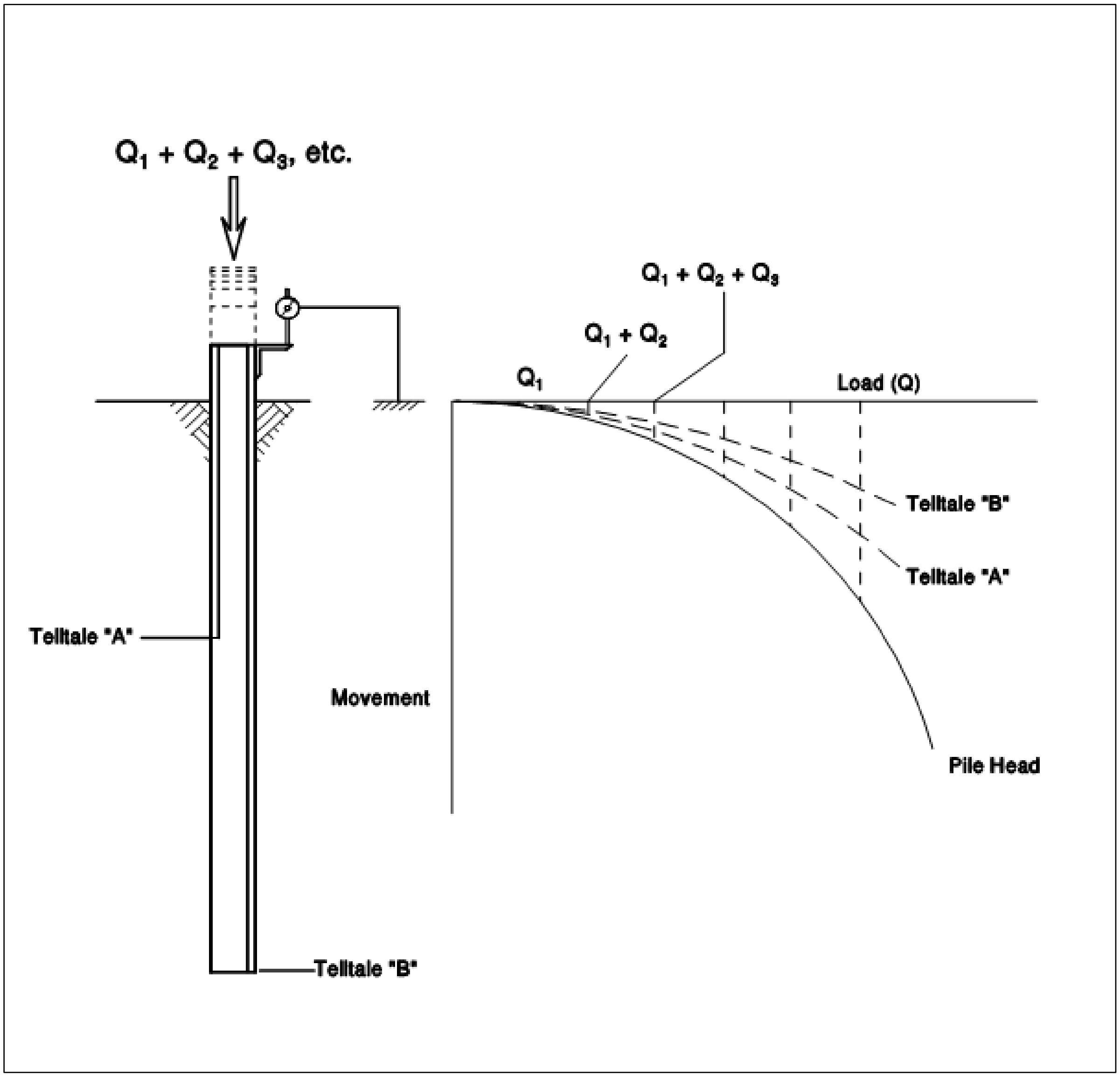


Figure 9-1 Axial compression static load test diagram.

9.2.1 Compression Load Test Equipment

ASTM D1143-07 (re-approved 2013) recommends several alternative systems for (1) applying compression load to the pile, and (2) measuring movements. Most often, compression 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 pressure from a calibrated pressure gage should also be recorded and be used as a secondary means of calculating the load applied to the pile. 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 LVDT's or dial gages that measure movement between the pile head and independently supported reference beam. The ASTM standard requires dial gages or LVDT's to have a minimum of 2 inches of travel and a precision of at least 0.01 inches. However, it is preferable and highly recommended to have gages with a minimum travel of 4 inches particularly for long piles with large elastic deformations under load and with a precision of 0.001 inches. A minimum of two dial gages or LVDT's mounted equidistant from the pile head, equidistant from the center of the pile, and diametrically opposite should be used. In many instances, four dial gages or LVDT’s mounted in diametrically opposite pairs may be advantageous.

A redundant backup system consisting of a scale, mirror, and wire system should be provided with a scale precision of 0.01 inches. The backup system should also be mounted on two diametrically opposite pile faces. Both the reference beams and backup wire system are to be independently supported with a clear distance of at least 5 times the pile diameter and not less than 8 feet between their supports and the test pile or reaction piles. 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. The survey level can also monitor reaction pile movement where reaction piles are used.

ASTM D1143 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 8 feet. If a weighted platform is used, the clear distance between cribbing supporting the weighted platform and the test pile should exceed 5 feet.

A schematic of a typical compression load test setup is presented in Figure 9-2. Photographs of the typical load application and movement monitoring components are presented in Figures 9-3 and 9-4. To improve the accuracy of the load cell readings, the loading arrangement shown in Figure 9-3 should include steel bearing plates between the hydraulic jack ram and the load cell, and between the load cell and spherical bearing plates. These bearing plates are included in the loading arrangement shown in Figure 9-5, and are discussed in greater detail in Section 9.6. The MnDOT mobile load frame attached to reaction piles is shown in Figure 9-5 with the load test being setup enclosed in a temporary heated structure for winter weather control. Wood reference beams are used to minimize temperature fluctuations on the reference beams.

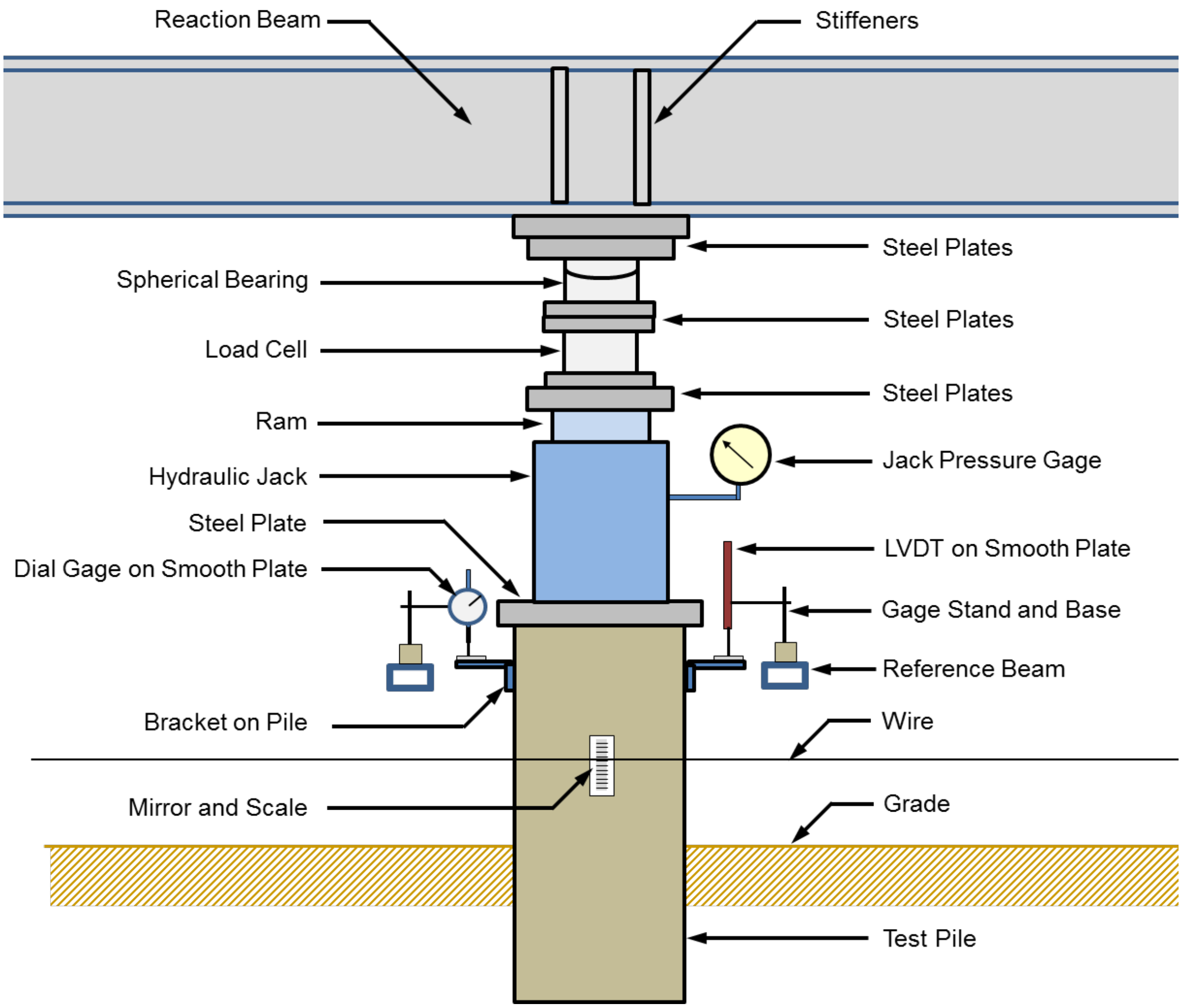


Figure 9-2 Static load test setup diagram.

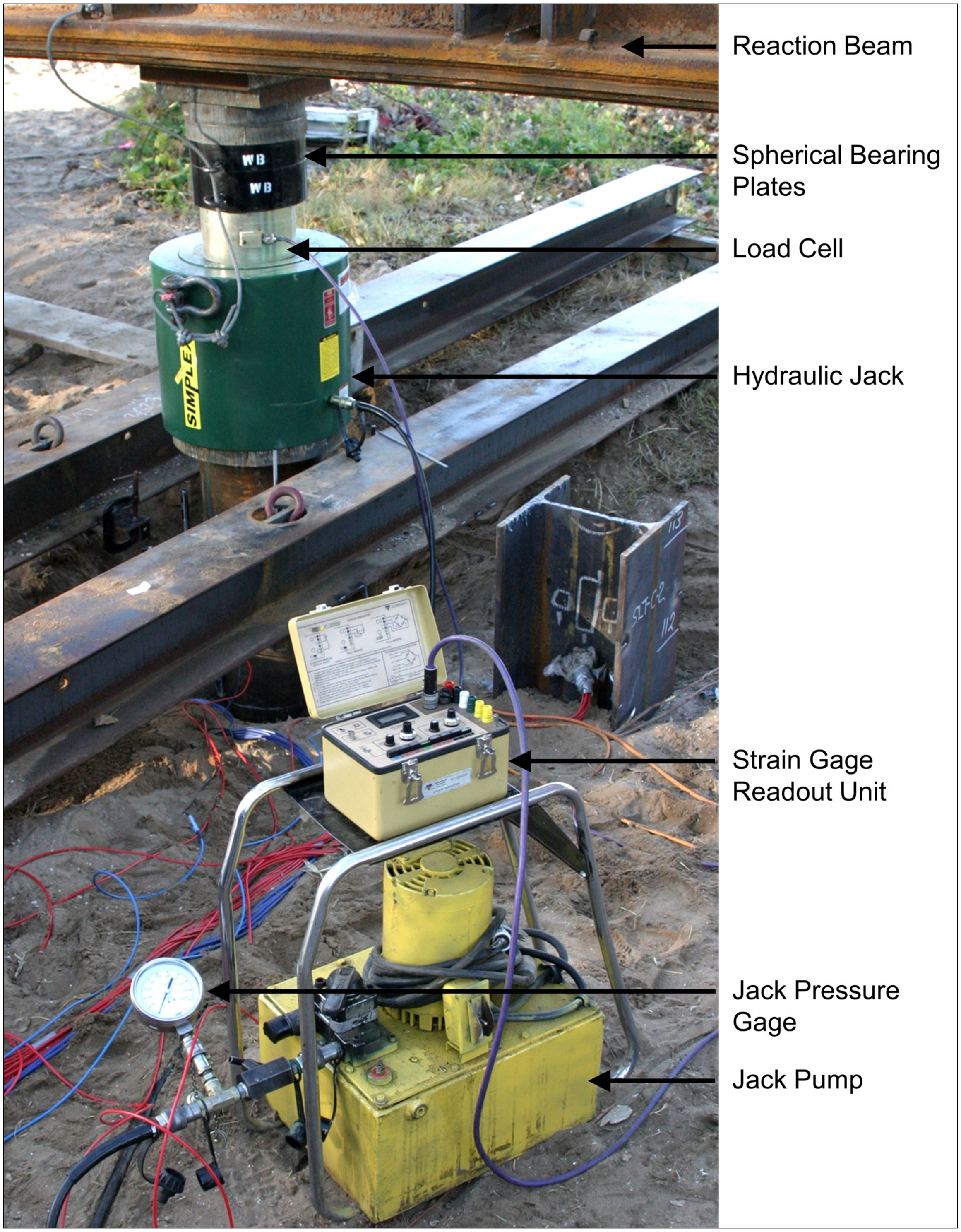


Figure 9-3 Load test application and monitoring system.

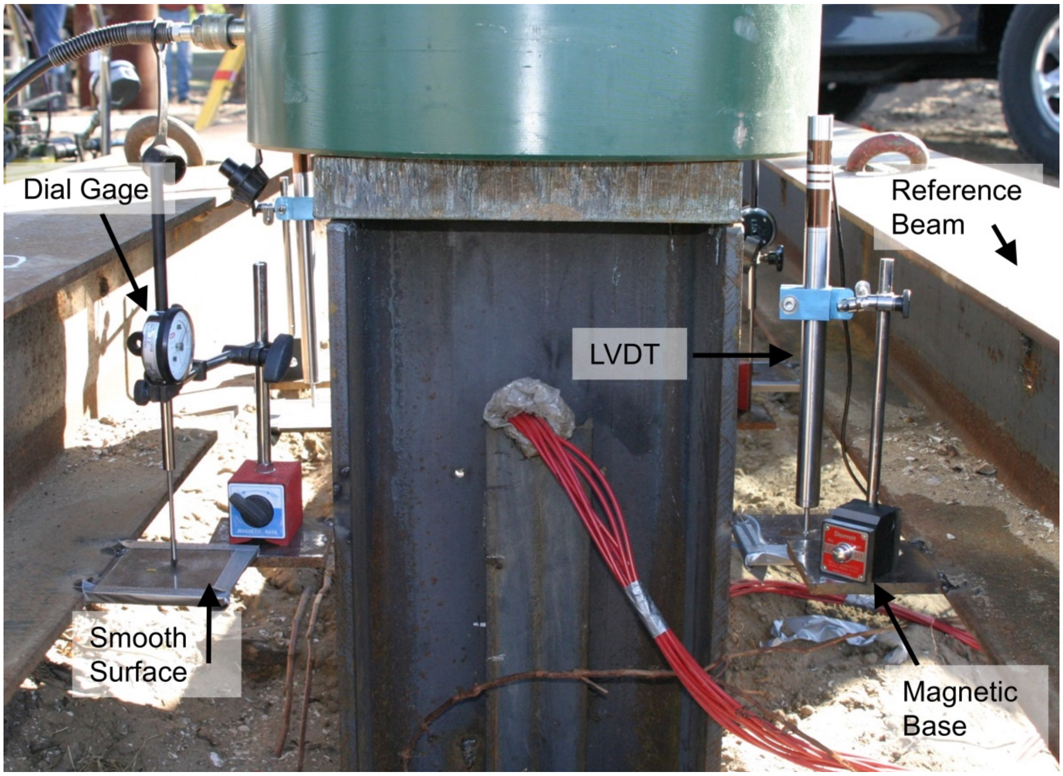


Figure 9-4 Load test movement monitoring components.

Figure 9-5 MnDOT reaction beam and load test setup (courtesy MnDOT).

A typical compression load test arrangement using reaction piles is presented in Figure 9-6 and a weighted platform arrangement is shown in Figure 9-7. It is recommended that load and movement measurements be recorded remotely using electronic instrumentation so that personnel are a significant distance from the load application / reaction system. Additional details on load application as well as pile head load and movement measurements may be found in ASTM D1143.

9.2.2 Recommended Axial Compression Test Loading Method

It is essential that standardized load testing procedures be followed. Several loading procedures are detailed in ASTM D1143, Standard Test Method for Deep Foundations Under Static Axial Compressive Load. The quick load test procedure is recommended by AASHTO (2014). This procedure is recommended because the load test can typically be performed in 2 to 4 hours thereby reducing construction delays and load test costs. However, alternative test procedures are also provided in ASTM D1143.

In the quick test procedure, the load is applied in increments of 5% of the anticipated nominal geotechnical resistance. This loading increment should produce on the order of 20 data points being recorded before reaching the geotechnical nominal resistance load thus allowing a detailed load-movement curve to be plotted.

Load increments should be held no more than 15 minutes and no less than 4 minutes, using the same time interval for all load increments. Readings of load and gross movement are to be recorded at 0.5, 1, 2, and 4 minutes after the load increment increase, with 8 and 15 minute readings if longer load increments have been selected. This procedure is to continue until the geotechnical nominal resistance or jack capacity is reached within the safe structural design limitations of the pile and reaction system. ASTM D1143 states that a longer hold time may be considered at the geotechnical nominal resistance to access creep. Note that this is not feasible if plunging movement under the applied load has occurred. If plunging movement occurs, the pump pressure should be locked off and the system should be allowed to come to equilibrium under the maximum resisted load. Upon reaching and holding the maximum load or reaching the load where plunging movement occurred, the pile is unloaded in 5 to 10 equal decrements which are each held for no more than 15 minutes and no less than 4 minutes. Readings of load and gross movement are to be recorded at 0.5, 1, 2, and 4 minutes (as well as 8 and 15 minutes) after each load reduction, including the zero load, which can be held longer to assess rebound.

Figure 9-6   4000-Ton mobile load test frame (courtesy Caltrans).

Figure 9-7   Static load test setup using weighted platform.

9.2.3 Presentation and Interpretation of Axial Compression Test Results

The results of axial compression load tests should be presented in a report conforming to the requirements of ASTM D1143. A load-movement curve similar to the one shown in Figure 9-8 should be plotted for interpretation of load test results.

The literature abounds with different methods of defining the nominal geotechnical resistance 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 are not recommended. These methods overestimate the nominal resistance of short piles and underestimate the nominal resistance of long piles. 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 (2014) design specifications Article 10.7.3.8.2 provides three interpretation criteria of axial compression test results based on pile size. All of the methods include consideration of the elastic deformation of the pile. For uniform presentation and interpretation of the axial compression load test results, the load and movement scales 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, for a pile of uniform cross section is computed from:

$$\Delta = \frac{QL}{AE} \qquad \text{Eq. 9-1}$$

Where:

Δ	=	elastic deformation of pile (inches).
Q	=	test load (kips).
L	=	pile length below dial gage or LVDT measurement location (inches).
A	=	pile cross sectional area (in^2).
E	=	elastic modulus of pile material (ksi).

AASHTO (2014) axial compression load test interpretation methods are based on an offset limit method as proposed by Davisson (1972). An offset limit line parallel to the elastic deformation line is plotted as shown in Figure 9-8. The point at which the observed load movement curve intersects the offset limit line is by definition the nominal geotechnical resistance or failure load. Note that this is a limiting deformation based criteria. In Figure 9-8, geotechnical plunging failure was not achieved, and additional geotechnical resistance develops beyond the defined failure load with additional pile head movement. If the load-movement curve does not intersect the offset limit line, the pile has a nominal geotechnical resistance in excess of the maximum applied test load.

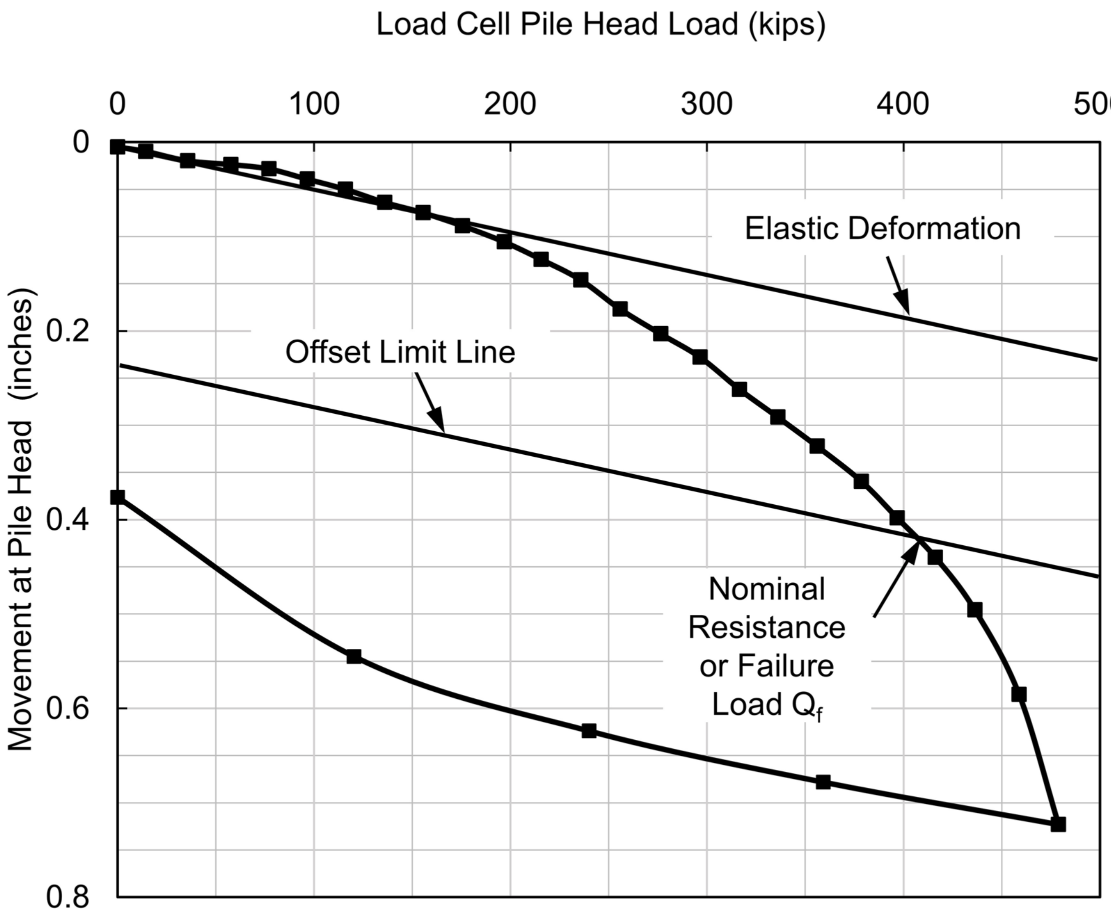


Figure 9-8 Typical load-movement curve for axial compression load test.

For piles 24 inches or less in diameter, the nominal geotechnical resistance, or failure load Q_f , is that load which produces a movement of the pile head equal to:

$$s_f = \Delta + \left(0.15 + \frac{b}{120}\right) \qquad \text{Eq. 9-2}$$

Where:

s_f = measured pile head movement at the nominal resistance (inches).

Δ = elastic deformation of pile per Equation 9-1 (inches).

b = pile width or diameter (inches).

For piles larger than 36 inches in diameter, additional pile toe movement is necessary to develop the toe resistance. For these larger diameter piles, the nominal geotechnical resistance, or failure load, can be defined as the load which produces at movement at the pile head equal to:

$$s_f = \Delta + \mathrm{b}/30 \qquad \text{Eq. 9-3}$$

Where:

- s_f = measured pile head movement at the nominal resistance (inches).
- Δ = elastic deformation of pile per Equation 9-1 (inches).
- b = pile width or diameter (inches).

For piles greater than 24 inches but less than 36 inches in diameter, linear interpolation should be performed between Eq. 9-2 and Eq. 9-3.

The pile diameter, *b*, used in Eq. 9-2 and Eq. 9-3 is defined by AASHTO as the pile diameter or width of side for square piles. However, for H-piles the value for b is not defined. For consistency with AASHTO treatment of square piles, it is recommended that the flange width be chosen for b.

It should be noted that the Davisson method is applicable for load tests in which the increment of load is held for not more than 1 hour. It is not applicable to load tests with long load holds and will underestimate the nominal geotechnical resistance in those cases due to the additional pile head movement that occurs during the extended load holds.

As discussed in Section 9.2.1, the load applied to the pile head is measured with a primary (load cell) and back-up (jack pressure) system. The applied load determined from both the primary and secondary system should be in general agreement, and should be compared to each other throughout the test to detect any major errors or discrepancies. If during initial loading increments, large discrepancies are observed between the applied load indicated by the primary and secondary systems, the test should be stopped, and the cause of the large discrepancy evaluated. Strain gage instrumentation above the ground surface can oftentimes quickly determine whether the load measured by the primary system or backup system is correct, and therefore which system should be used to control the test. If a load test is performed with large unresolved discrepancies, safety issues and/or later controversies may arise. If the lower value of applied load is used, and the actual applied load is the higher value, a safety issues can occur during the test by operating the jack beyond its intended range, or by exceeding the nominal structural resistance of the test pile or reaction system. If the higher value of applied load is used, and the actual applied load is the lower value, the test may be terminated prematurely or the nominal resistance determined from the test lower than anticipated. This may require the entire load test to be repeated, the use of longer pile lengths than necessary, a foundation redesign because of the lower load, or revisions to the driving criterion. Therefore whenever large discrepancies occur, the applied load should be removed from the test pile (unloaded), the cause of the discrepancy identified and corrected, and the load test restarted.

The factored compression resistance is 0.75 to 0.80 of the nominal resistance defined by the failure load. This range in the factored compression resistance depends on whether the driving criteria is developed solely from the static load test results (ϕ_{dyn} = 0.75), or if static load test results and dynamic testing with signal matching (ϕ_{dyn} = 0.80) is used.

An axial compression load test will not provide detailed load-transfer information in the penetrated geomaterials solely from the pile head load-movement results. Additional instrumentation as described in Section 9.5 is required if load-transfer information along the pile shaft or the delineation of shaft and toe resistances is desired.

9.2.4 Limitations of Axial Compression 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 nominal resistance. 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. If detailed load-transfer assessments are desired, the pile must have additional strain gage instrumentation attached or embedded in the pile, which can add significant cost.

9.3 TENSION LOAD TEST

Tension load tests are performed to determine axial tension (uplift) resistance of piles. The uplift resistance is often a significant factor in determining minimum pile penetration requirements for pile groups subject to large uplift loading demand including cofferdam seals that create large buoyancy forces, cantilever segmental bridge construction, as well as seismic, vessel impact, or debris loading. The basic mechanics of a tension test are similar to compression load testing, except the pile is loaded in tension. Tension load tests are often economical since they can frequently be performed by jacking against a reaction beam supported by crane mats rather than reaction piles.

9.3.1 Tension Load Test Equipment

ASTM D3689-07 (re-approved 2013) describes The Standard Method of Testing Deep Foundations Under Static Axial Tensile Load. Several alternative systems for applying tension load to the pile and measuring movements are provided in this standard. Most often, tension loads are applied by centering a hydraulic jack on reaction beams supported by reaction piles or by mats and cribbing supported by the ground. The jack can be a center hole jack allowing a rod embedded in the pile to pass in between the reaction beams and through the jack. This is a common arrangement for concrete piles with relatively low tension loads. Alternatively, a yoke type of connection can be used to transfer the tension load to the pile. In this arrangement, steel rods pass on either side of the reaction beam(s). The rods are connected to the pile and to a beam on top of the jack. This is a common arrangement for steel piles. 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 tension testing, ASTM requires that the dial gages or LVDT's have a minimum of 2 inches of travel and a precision of at least 0.01 inches. While not required, gages with 4 inches of travel are recommended. A minimum of two dial gages or LVDT's mounted equidistant from the point of load application, equidistant from the center of the pile, and diametrically opposite should be used. A redundant backup system consisting of a scale, mirror, and wire system should be provided with a scale precision of 0.01 inches. The backup system should also be mounted on two diametrically opposite pile faces. Both the reference beams and backup wire system are to be independently supported with a clear distance of at least 5 times the pile diameter and not less than 8 feet between supports and the test pile or reaction piles. 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. Additional details on load application, and pile head load and movement measurements may be found in ASTM D3689.

Photographs of tension load test arrangements are presented in Figure 9-9 and 9-10. In Figure 9-9, a tension load test arrangement for a nearshore situation using a reaction beam and reaction piles is presented. In Figure 9-10, a tension load test arrangement for a land situation using a reaction beam supported by crane mats and timber cribbing is illustrated.

9.3.2 Tension Test Loading Methods

Several loading procedures are detailed in ASTM D3689. The Quick Test method where load is applied in increments of 5% of the anticipated nominal geotechnical tension resistance is recommended. This loading increment should produce on the order of 20 data points being recorded before reaching the nominal geotechnical resistance thus allowing a detailed load-movement curve to be plotted. All load increments should be held for the same time interval of no more than 15 minutes and no less than 4 minutes.

Readings of load and gross movement are to be recorded at 0.5, 1, 2, and 4 minutes after the load increment increase, with 8 and 15 minute readings for longer hold times. This procedure is to continue until the nominal geotechnical resistance or jack capacity is reached within the safe structural design limitations of the pile and reaction system. Upon reaching and holding the maximum load or reaching the load where geotechnical pullout failure occurred, the pile is unloaded in 5 to 10 equal decrements which are each held for no more than 15 minutes and no less than 4 minutes. Load and gross movement are to be recorded at 0.5, 1, 2, and 4 minutes (as well as 8 and 15 minutes if applicable) after each load reduction, including the zero load. Additional optional loading procedures are detailed in ASTM D3689.

It is generally desirable to test a pile in tension loading to geotechnical pullout failure, particularly during a design stage test program. If construction stage tension 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 load testing.

Figure 9-9 Tension test on pipe pile using reaction piles (courtesy of Besix).

Figure 9-10 Tension test on H-pile using mats and cribbing (courtesy of WKG[2]).

9.3.3 Presentation and Interpretation of Tension Test Results

The results of tension load tests should be presented in a report conforming to the requirements of ASTM D3689. A load movement curve similar to the one shown in Figure 9-11 should be plotted for interpretation of tension load test results.

A widely accepted method for determining the nominal geotechnical resistance in tension loading has not been published. Fuller (1983) reported that acceptance criteria for tension 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.

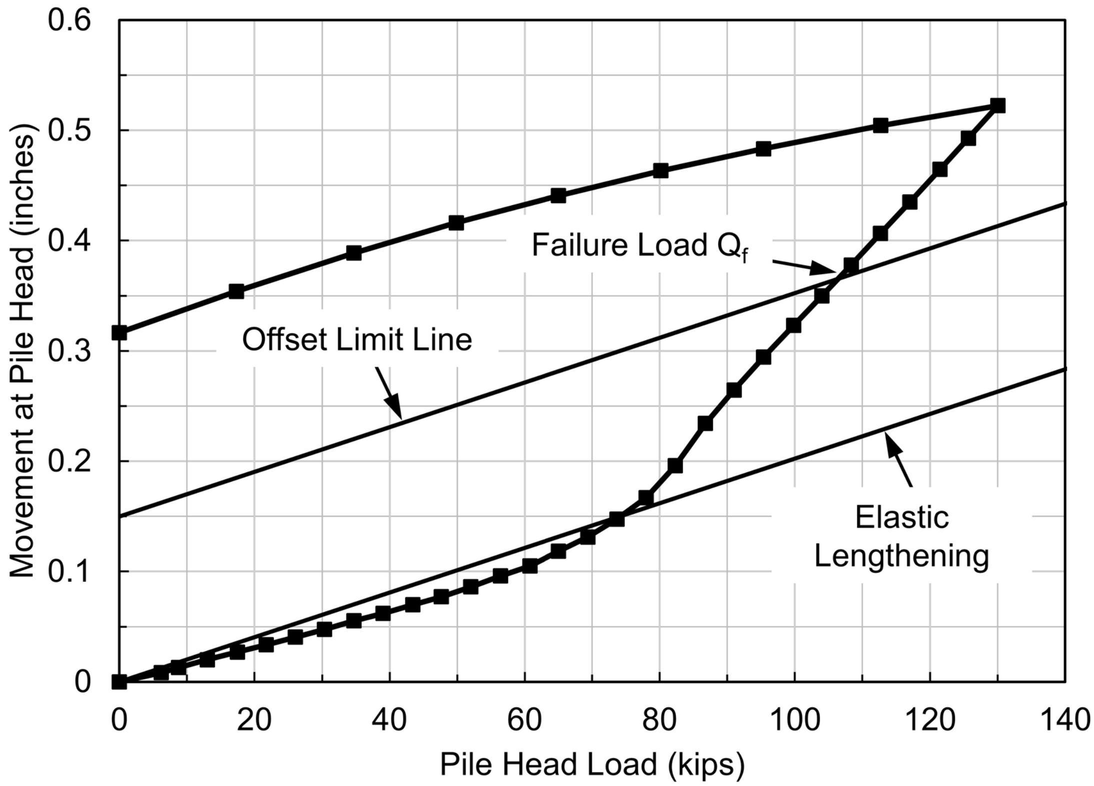


Figure 9-11 Typical tension load test load-movement curve.

AASHTO (2014) design specifications recommend tension load tests be evaluated using a modified Davisson Method that considers the elastic lengthening of the pile plus an offset limit. For tension loading, the suggested offset is 0.15 inches. Hence, the load at which the load movement curve intersects the elastic lengthening plus 0.15 inches is then defined as the nominal tension resistance or failure load. The factored tension resistance is then 0.60 of the nominal tension resistance defined by the failure load.

9.4 LATERAL LOAD TEST

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, particularly with regard to extreme event limit states consideration such as seismic and vessel impact. Lateral load testing has also increased due to the greater use of noise walls and large overhead signs. Most lateral load tests are performed on a single pile with a free head condition. In service, most piles are installed in groups having a fixed head condition. Hence, lateral load test results on a single pile with a free head pile condition may not be directly applicable to design. Accordingly, the primary purpose of lateral load testing is to determine the p-y curves modelling soil behavior to be used in the design or to verify the appropriateness of the p-y curves on which the design is based. Lateral load tests are often economical since they can frequently be performed by jacking or pulling between two piles. For design stage tests, the two piles can be different pile types or pile sections enabling lateral deformation behavior for two pile types to be assessed with one lateral load test.

9.4.1 Lateral Load Test Equipment

ASTM D3966-07 (re-approved 2013) describes The Standard Method of Testing Deep Foundations Under Lateral Load. Several alternative systems for applying the lateral load to the pile and measuring movements are provided in this standard. Most often, lateral loads are applied by a hydraulic jack acting against a reaction system (e.g. piles, deadman, or weighted platform), or by a hydraulic jack acting between two piles. When jacking between two piles, pulling the piles toward one another is the preferred arrangement as the system comprises a two-force member and so alignment is naturally maintained. However, with this arrangement ASTM requires a significant distance between the test piles clear distance of not less than 20 feet or 20 pile diameters. 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 D3966 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 3 inches of travel and a precision of at least 0.01 inches. 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 0.01 inches. 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 either by installing an inclinometer casing on or in the test pile to the depth where it is reasonable certain that no lateral movement will occur and recording inclinometer readings immediately after application or removal of a load increment held for a duration of 30 minutes or longer. Alternatively, a Shape Accel Array (SSA) can be embedded in or attached to the test pile. 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 D3966 and FHWA-SA-91-042 (Kyfor et al. 1992).

A photograph of a typical lateral load test arrangement is presented in Figure 9-12. This figure shows a 14 and a 16 inch O.D. concrete filled pipe pile being pushed apart. The jack is located adjacent to the right pile and the load cell and spherical bearing plate are located adjacent to the left pile. Figures 9-13 and 9-14 present close-ups of these devices. Both piles were also equipped with a string of in-place inclinometers for prompt readout of the deflected pile shape with each load increment. A photograph of the multiple inclinometer string components is presented in Figure 9-15.

As an alternative means to measure horizontal deflection, a Shape Accel Array (SSA) can be embedded in or placed along the pile length (Rollins et al. 2009). These triaxial accelerometers are typically installed in 1 inch diameter PVC casing, are recoverable and reusable, and can output displacements for both static and dynamic loads (Measureand 2015). SSAs are currently produced in 12 inch segments and therefore provide a more detailed assessment of deflections as compared with typical inclinometers. Figure 9-16 shows a typical SSA.

In cases where it is not feasible to acquire lateral deflection measurements versus depth, it may be useful to measure pile head rotation during the lateral load test. The pile head rotation can be obtained by measuring the lateral pile deflection at two different elevations above the point of load application or by attaching an inclinometer probe or tiltmeter to the pile head. The measured pile head rotation may be used to assess appropriate p-y modelling.

Figure 9-12 Lateral load test on 14 inch and 16 inch O.D. concrete filled pipe piles (courtesy of WKG[2]).

Figure 9-13 Jack for lateral load test (courtesy of WKG[2]).

Figure 9-14 Spherical bearing plate and load cell for lateral load test (courtesy of WKG[2]).

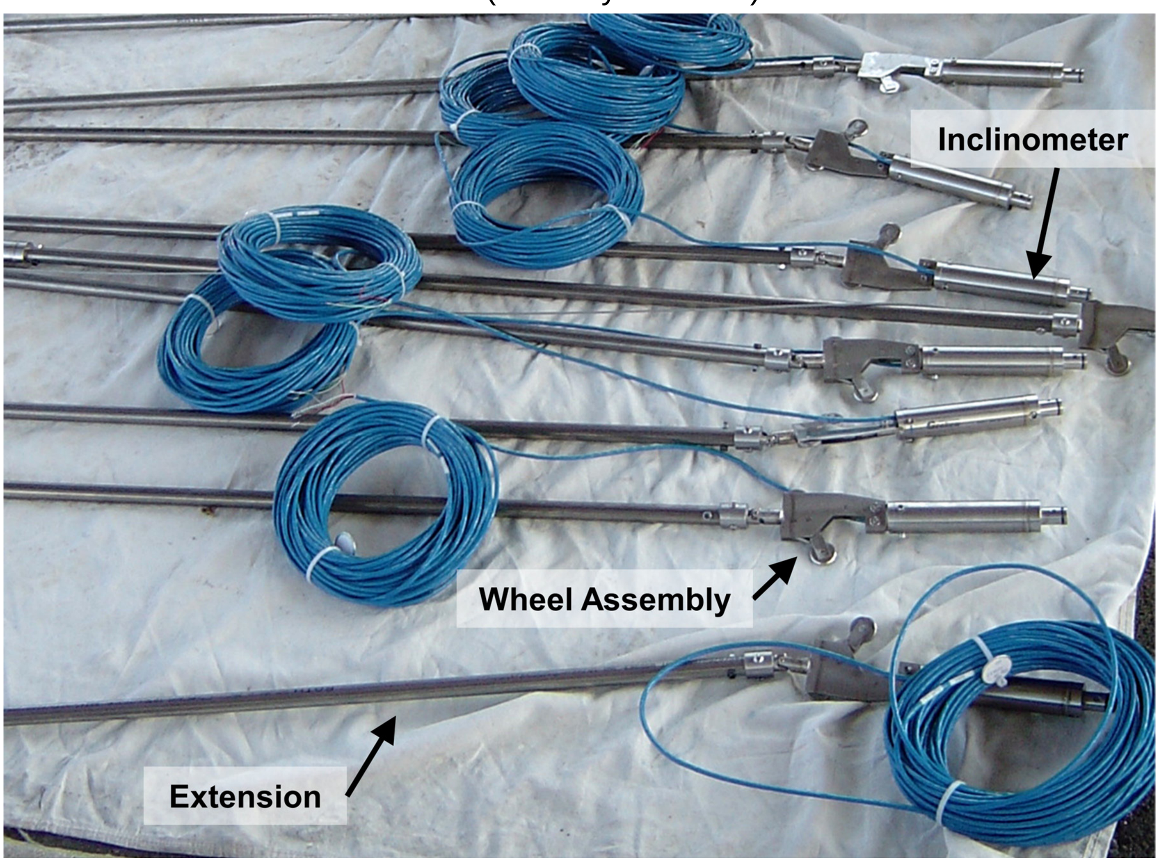


Figure 9-15 Multiple inclinometer string components for lateral load test (courtesy of WKG[2]).

Figure 9-16 Typical Shape Accel Array (SSA) (courtesy of Measurand).

9.4.2 Lateral Test Loading Methods

Several loading procedures are detailed in ASTM D3966. 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 50% of the maximum load, with 10 minute holds between load decrements. This loading schedule is shown in Table 9-2.

Table 9-2 Standard Loading Schedule (after ASTM D3966)

Percent of Design Load	Load Duration, minutes
0	-
25	10
50	10
75	15
100	20
125	20
150	20
170	20
180	20
190	20
200	60
150	10
100	10
50	10
0	30

Readings of load and gross movement are recorded immediately before and after each change in load, with additional readings taken at 5 minute intervals between load increments. Upon load removal, readings should be taken at 15 and 30 minutes.

9.4.3 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 D3966. The interpretation and analysis of lateral load test results is much more complicated than those for compression and tension load tests. Figure 9-17 presents a typical pile head load-movement curve from a lateral load test. A lateral deflection versus depth curve similar to the one shown in Figure 9-18 should also be reported for lateral load tests that include lateral deflection measurements versus depth. The measured lateral deflection behavior versus depth should also be plotted and compared to design analysis results from p-y computer software as indicated in Figure 9-19. 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.

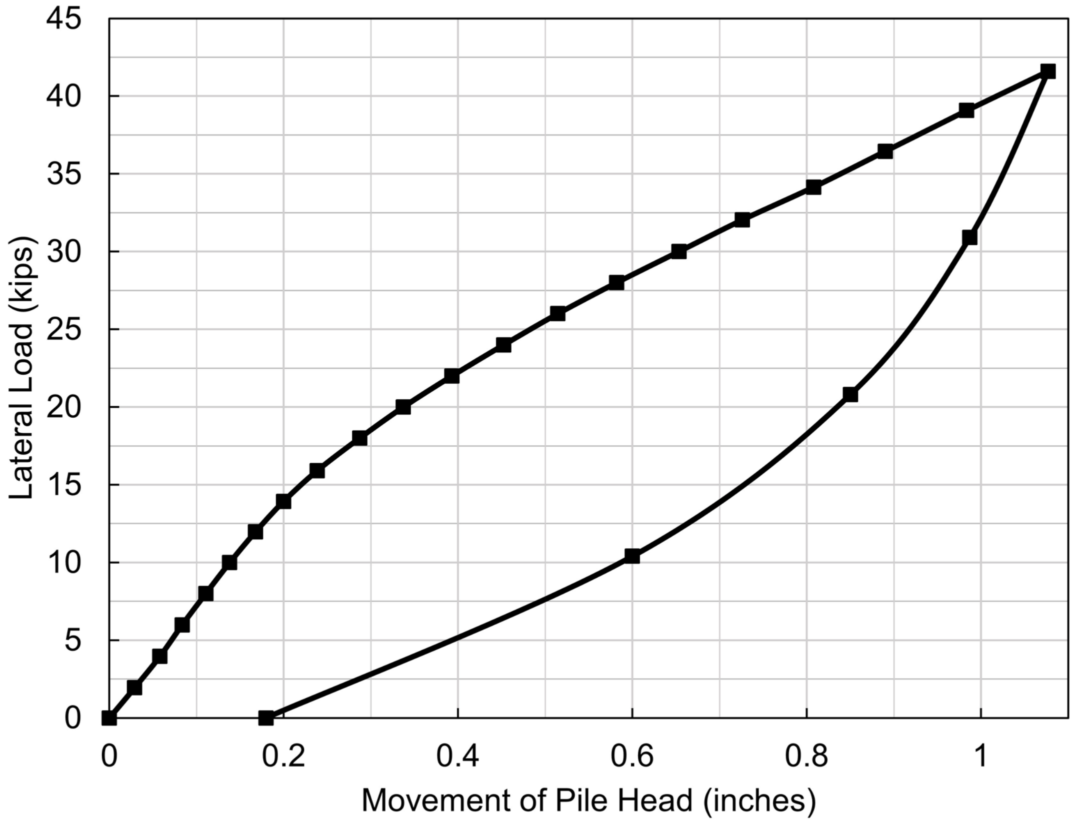


Figure 9-17 Typical lateral load test, pile head load-deflection curve.

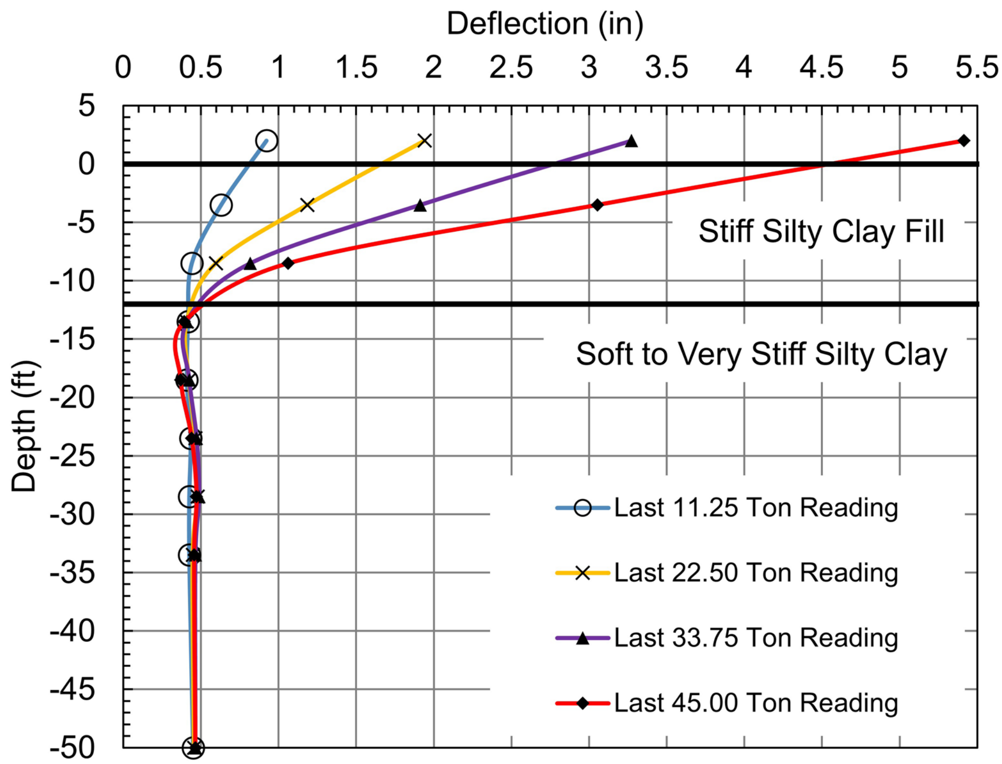


Figure 9-18 Plot of lateral load test measured deflected shape versus depth.

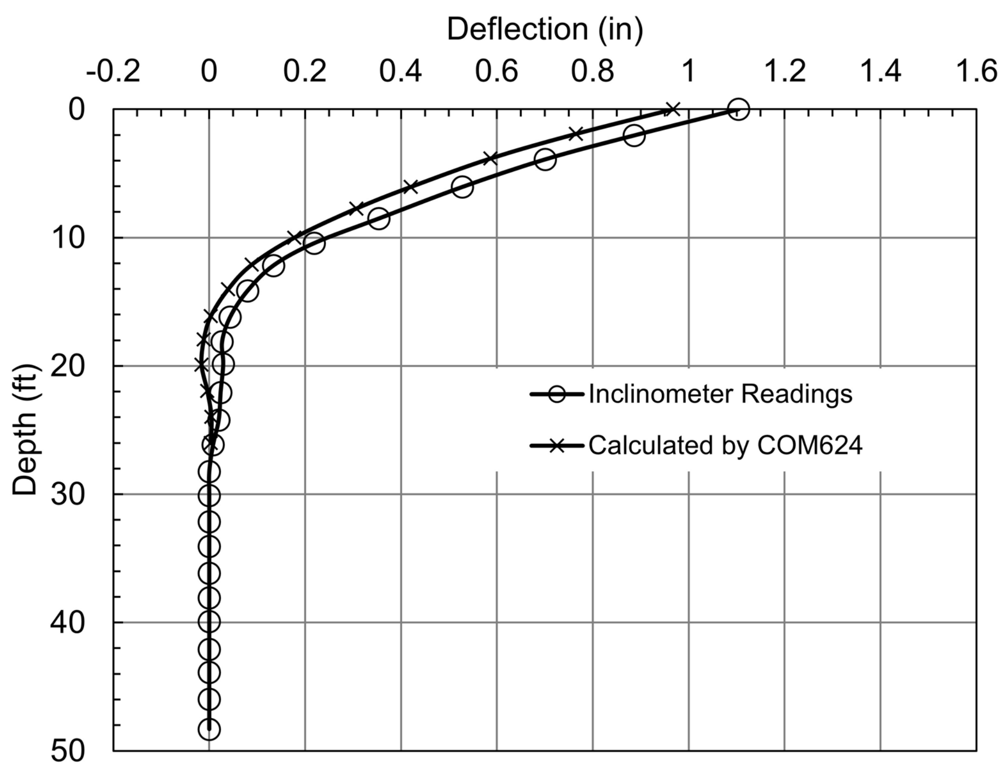


Figure 9-19 Comparison of measured and COM624P predicted load deflection behavior versus depth (after Kyfor et al. 1992).

9.5 LOAD TRANSFER EVALUATIONS

The magnitude of shaft and toe resistances or the unit shaft resistance values can be determined though instrumented static load tests. Load transfer evaluation along the pile shaft may be determined through the use of telltale rods (extensometers) or through surface mounted or embedded strain gages. Details on the use of telltales or strain gages for load transfer determinations are discussed in the following sections.

9.5.1 Use of Telltales

Telltales are thin steel rods that extend from the pile head to a selected point in the pile. The rods are encased within a slightly larger tube. A schematic of telltale rod placement within a load test is shown in Figure 9-20. The NYSDOT Static Pile Load Test Manual, GCP-18 (2007) provides telltale details for pipe piles, H-piles, and timber piles.

LVDT's or dial gages attached to the top of the telltale rod measure the relative movement between the rod attachment locations on the pile and other points such as the independent reference beam or the pile head. When using multiple rods, the deflection difference between the rod attachment points can provide the load transfer between those fixed locations. According to Kyfor et al. (1992), the average load in the pile, Q_{avg}, between two measuring points can be determined as follows:

$$\mathrm{Q}_{avg} = A\, E\, \frac{R_1 - R_2}{L} \qquad \text{Eq. 9-4}$$

Where:

- Q_{avg} = average load in pile between two points (kips).
- A = pile cross sectional area (in^2).
- E = elastic modulus of pile material (ksi).
- R_1 = deflection reading at upper measurement location (inches).
- R_2 = deflection reading at lower measurement location (inches).
- L = length of pile between two measuring points under no load condition.

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

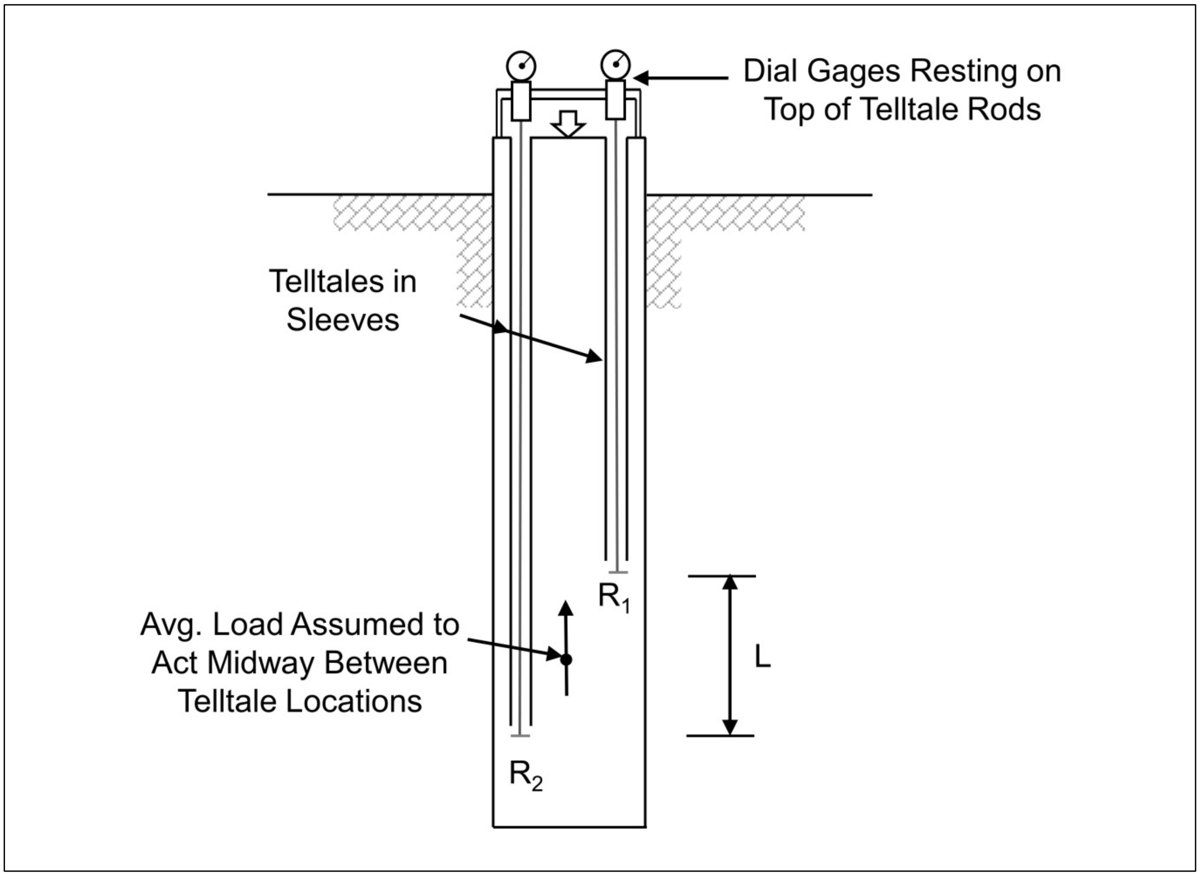


Figure 9-20 Diagram of telltale rods installed on pile (modified from Kyfor et al. 1992).

that an estimate of the toe resistance, R_p, can be computed from the applied pile test load, Q. The applied pile head load, Q, is chosen as close to the nominal resistance or failure load as possible.

$$R_p = 2\,Q_{avg} - Q \qquad \text{Eq. 9-5}$$

Where:

R_p = nominal toe resistance (kips).

Q_{avg} = average load in pile between two points (kips).

Q = test load, applied at the pile head (kips).

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

$$R_p = 3\,Q_{avg} - 2\,Q \qquad \text{Eq. 9-6}$$

Where:

R_p = nominal toe resistance (kips).

Q_{avg} = average load in pile between two points (kips).

Q = test load, applied at the pile head (kips).

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

9.5.2 Use of Strain Gages

When detailed load transfer data is desired, telltale measurements alone are insufficient, due to the presence of unaccounted for residual load. 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. Other gage types may also be used if load transfer measurements under both static and dynamic conditions are desired. A geotechnical instrumentation specialist should be used to select the appropriate instrumentation for the application, to select instrumentation that can withstand pile handling and pile driving, to determine the needed instrumentation redundancy, and to determine the appropriate data acquisition system for the program.

A sister bar vibrating wire strain gage is presented in Figure 9-21. The protective cover has been removed from the top gage in the box. Sister bar gages are often cast into prestressed concrete piles or embedded in concrete filled pipe piles during concrete placement. The sister bars are tied to the longitudinal rebar in the casting yard for prestressed concrete piles or attached to centralized steel bar or pipe when cast into a concrete filled pipe pile.

An arc-weldable vibrating wire strain gage attached to a steel H-pile is shown in Figure 9-22. The gage attachment blocks are welded to the pile and then the vibrating wire gage positioned between the mounting blocks.

Piles can also be instrumented with resistance type strain gages. These gages can be used for both static load-transfer and dynamic measurements on the pile. A bolt-on, waterproof, foil resistance strain gage attached to the side of a steel pipe pile is shown in Figure 9-23. The gages and instrumentation cables are covered and protected by a steel channel as the pile is driven below grade. Resistance type strain gages mounted on sister bars are shown in Figure 9-24. These gages are being cast into a prestressed concrete pile. The center sister bar also includes an accelerometer for dynamic testing purposes.

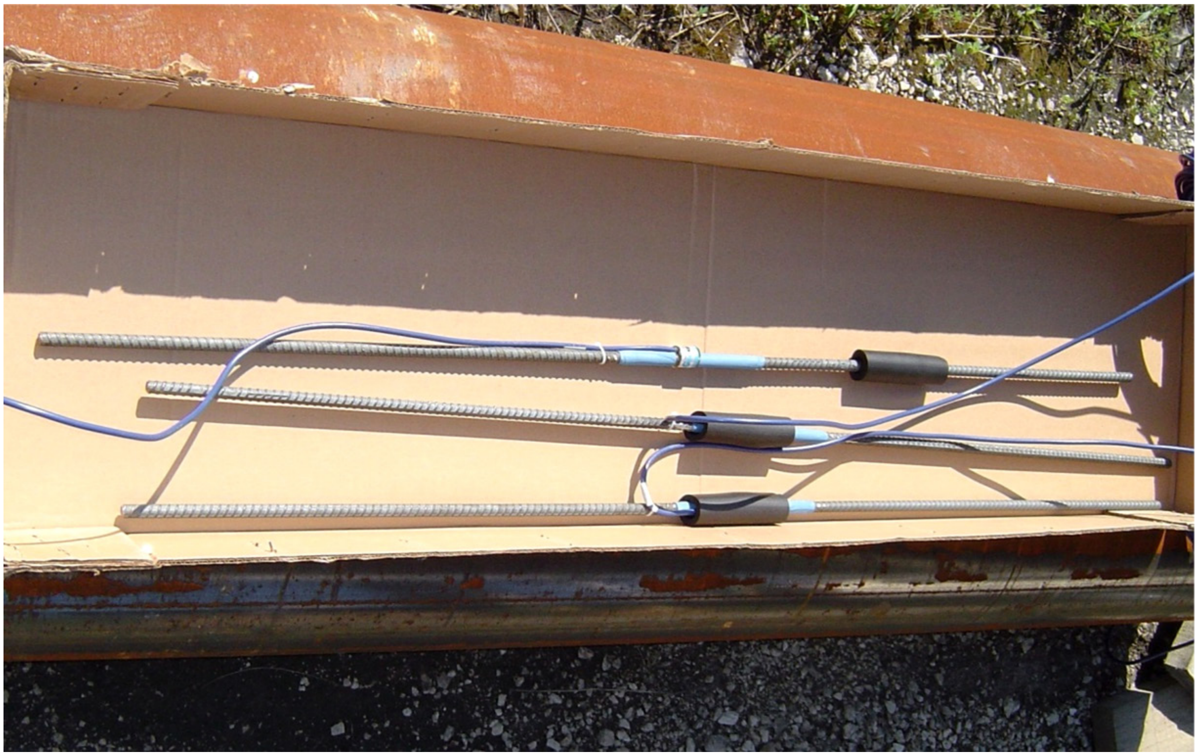


Figure 9-21 Vibrating wire strain gage sister bars for concrete embedment.

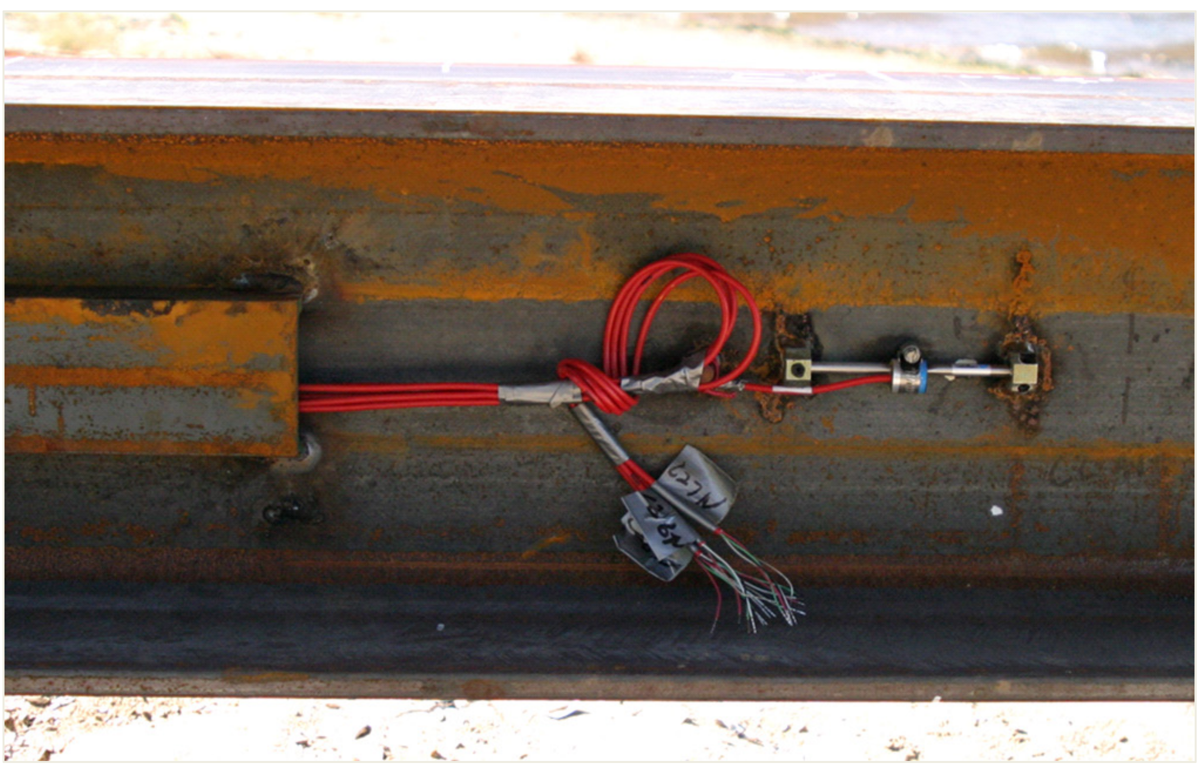


Figure 9-22 Vibrating wire strain gage with welded anchor blocks and protective channel.

Figure 9-23 Waterproof electrical resistance strain gage bolted on pipe pile with protective channel (courtesy of Besix).

Figure 9-24 Electrical resistance strain gage on sister bars in concrete pile casting bed.

Moisture is one of the leading causes of resistance strain gage failure. Therefore, all lead wires should be carefully checked for any nicks or abrasion and sealed as appropriate in these instrumentation situations. A multiple channel data acquisition system is also required as part of the instrumentation system.

Strain gages are generally attached at pre-selected points along the pile length to determine load-transfer and unit shaft resistances in specific strata or at prescribed locations. The closest soil boring to the load test location should be reviewed as part of the load-transfer instrumentation program planning so that the desired unit shaft resistance values for the selected geomaterials are determined. Additional gages can be distributed based upon the Engineer's experience, the need for instrumentation redundancy, site specific conditions, and project requirements. Cost effective designs can be finalized based on the determined geomaterial resistances.

The uppermost gage location should be below the pile head in an area where shaft resistance does not act on the pile. Dunnicliff (1988) recommends three undamaged pile diameters be left above the ground surface to simulate an unconfined compression specimen. At a minimum, the pile head strain gages should be at least two pile diameters below the pile head to allow for full load development across the pile section. At this location, a modulus determination can be made while a comparison of internal and external strain readings can indicate composite section action for concrete filled pipe piles (Sellers 1995; Komurka 2015). As depicted in Figure 9-25, it is also prudent to mitigate bending effects by using multiple external gage pairs equidistant from the head and center of the pile, or centralizers on single embedded gages.

Komurka (2015) also recommended another gage location; two pile diameters above the pile toe to be used for load transfer evaluation near the pile toe. He reported load transfer can be extrapolated from this location over the remainder of the pile to estimate the pile toe resistance. Sister bars near the pile toe should not rest on the toe when cast into concrete filled pipe piles but should instead be supported from the pile head during the concrete curing process.

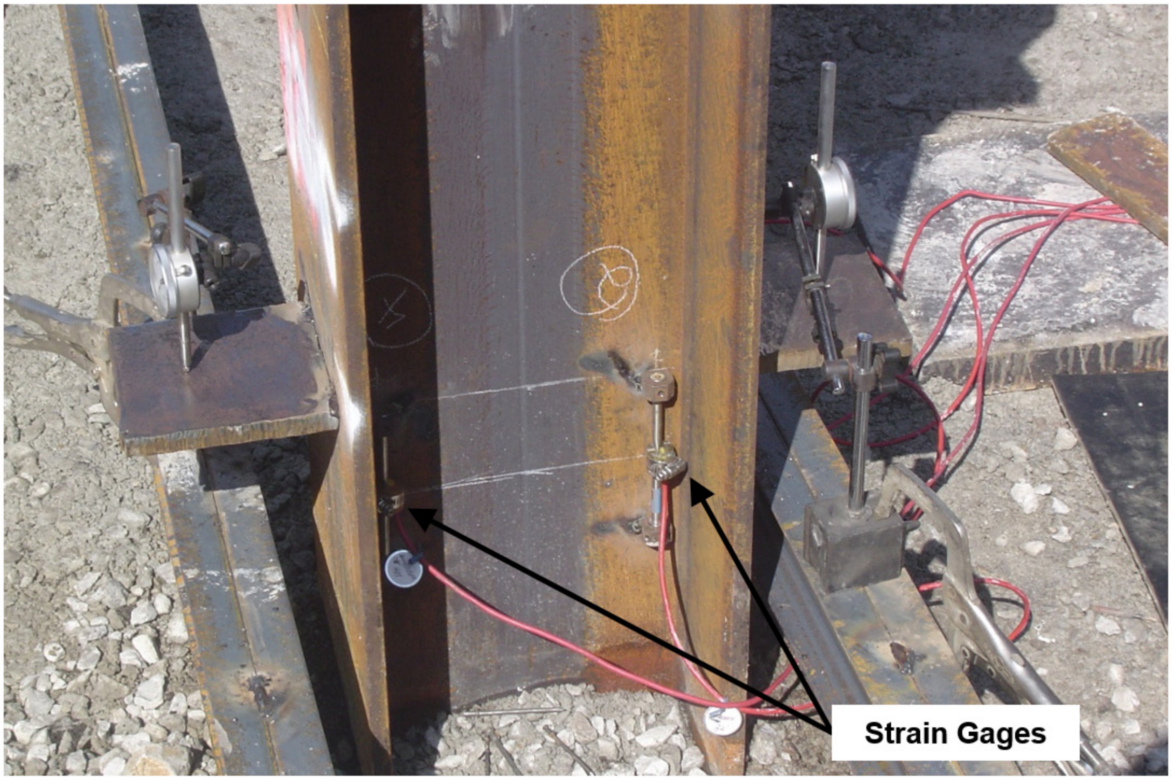


Figure 9-25 Multiple externally mounted strain gages (2 on each web face) located in soil resistance free area during static load test (courtesy WKG[2]).

Following strain gage attachment and pile installation, the axial force in the plane of the gage can be determined using Eq. 9-7.

$$F = \varepsilon\, E\, A \qquad \text{Eq. 9-7}$$

Where:

F	=	axial force in plane of gage (kips).
ε	=	strain measured in gage.
E	=	elastic modulus of pile material (ksi).
A	=	pile cross sectional area (in^2).

The modulus of elasticity for steel is well defined. However, for concrete filled steel piles, a composite modulus of the concrete and steel section must be calculated using Eq. 9-8.

$$\mathrm{E} = \frac{\mathrm{E}_{st}\,\mathrm{A}_{st} + \mathrm{E}_c\,\mathrm{A}_c}{\mathrm{A}_{st} + \mathrm{A}_c} \qquad \text{Eq. 9-8}$$

Where:

E = elastic modulus of pile material (ksi).
E_{st} = elastic modulus of steel (ksi).
A_{st} = cross sectional area of steel (in^2).
E_c = elastic modulus of concrete (ksi).
A_c = cross sectional area of concrete (in^2).

The modulus of elasticity of concrete also decreases with increasing strain, and this value should vary over the pile length due to load-transfer. Fellenius (2001), recommended the Tangent Modulus Method be used to determine a secant modulus, E_{sm}, for load transfer evaluations of concrete piles. The tangent modulus versus measured microstrain is plotted for all strain gage levels and all load increments. As shown in Figure 9-26, a best fit line through the data determines the slope, *A*, and y-axis intercept, *B*. Following determination of the tangent modulus line from Eq. 9-9, integration is used to calculate stress in Eq. 9-10.

$$\mathrm{M}_t = \left(\frac{\Delta\sigma}{\Delta\varepsilon}\right) = A\varepsilon + B \qquad \text{Eq. 9-9}$$

$$\sigma = \left(\frac{\mathrm{A}}{2}\right)\varepsilon^2 + B\varepsilon \qquad \text{Eq. 9-10}$$

Where:

M_t = tangent modulus (ksi).
$\Delta\sigma$ = change of stress from one load increment to the next (ksi).
$\Delta\varepsilon$ = change of strain from one load increment to the next.
A = slope of tangent modulus line.
ε = strain measured in gage.
B = Y-intercept of the tangent modulus line.
σ = stress (ksi).

Through the use of Hooke's law, the secant modulus is related to stress and strain in Eq. 9-11. By substituting terms from Eq. 9-10, the secant modulus can be expressed as shown in Eq. 9-12.

$$\sigma = \mathrm{E}_{sm}\varepsilon \qquad \text{Eq. 9-11}$$

$$\mathrm{E}_{sm} = 0.5\, A\, \varepsilon + B \qquad \text{Eq. 9-12}$$

Where:

σ	=	stress (ksi).
E_{sm}	=	secant modulus (ksi).
ε	=	strain measured in gage.
A	=	Slope of tangent modulus line.
B	=	Y-intercept of the tangent modulus line.

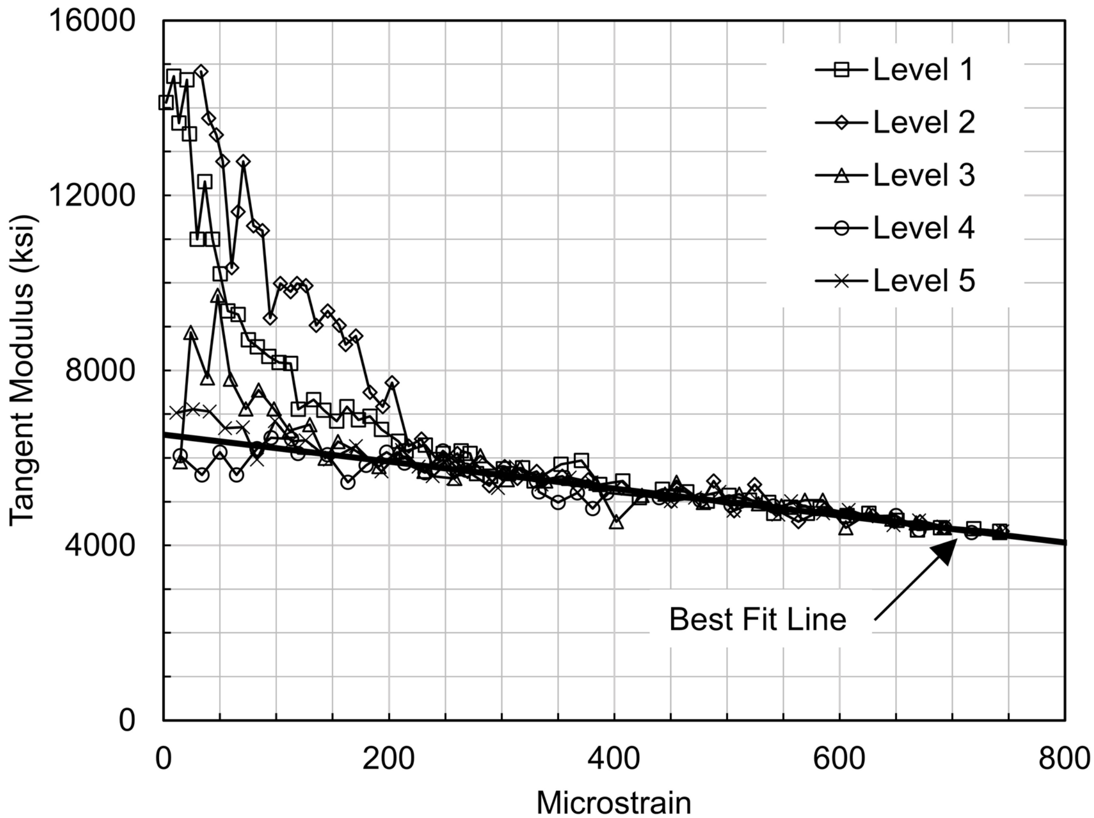


Figure 9-26 Tangent modulus method for determining elastic modulus (modified from Fellenius 2001).

Once the modulus for the pile material is determined, Eq. 9-7 can be used to determine the force or load at a given depth. A plot of axial load versus elevation from a static load test is presented in Figure 9-27. In this figure, seven embedded gages were installed to provide a detailed load transfer evaluation for a 14 inch diameter, concrete filled, steel pipe pile. The load transferred to the soil can be calculated from the difference in force between selected measurement locations. For example, when utilizing the final load increment, 16.3 tons of load (e.g. 53.9 tons - 37.6 tons equals 16.3 tons) is transferred to the soil between the strain gages located at elevations EL 549.7 and EL 537.4. The pile in Figure 9-27 is 3.67 feet in circumference, and because the final two strain gages are located 12.3 feet apart, the unit shaft resistance over this interval is 16.3 tons divided by 45.1 ft^2 (12.3 feet times 3.67 feet) or 0.36 tsf. It should be noted that this computation did not consider the effects of any residual load in the pile which is discussed in Section 9.5.3.

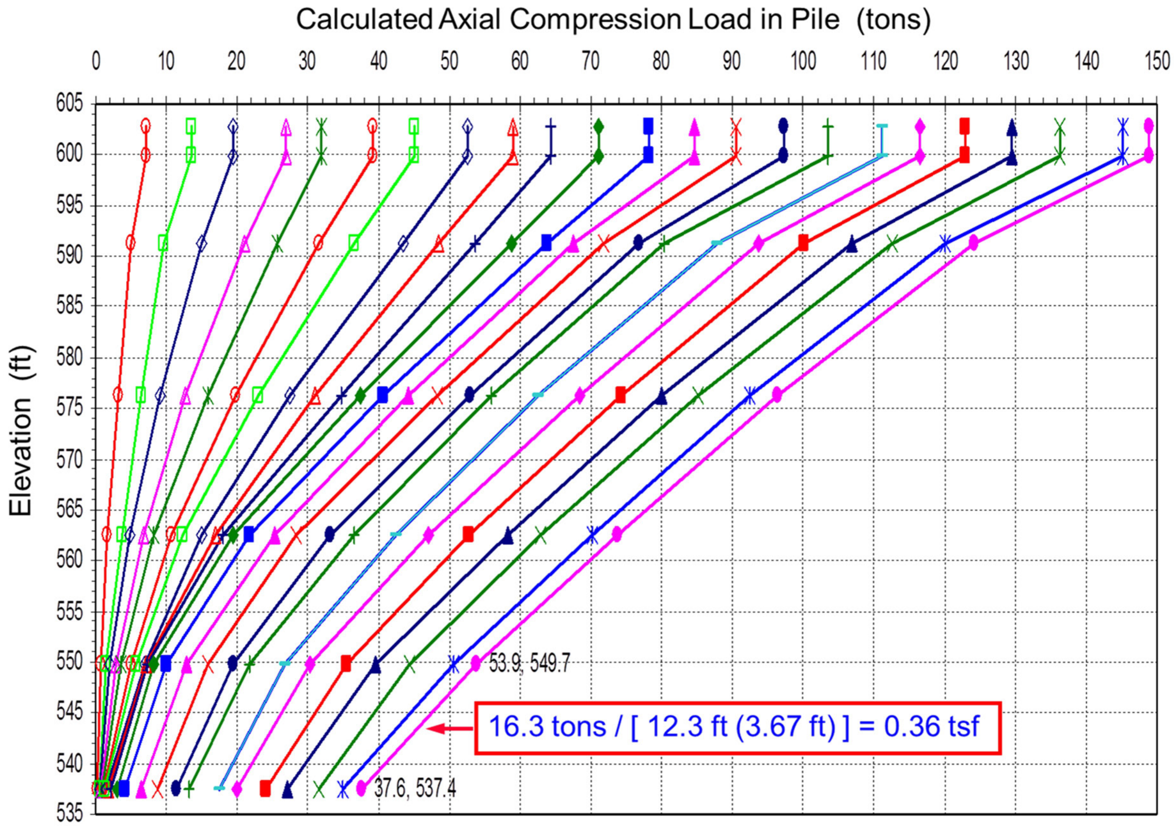


Figure 9-27 Plot of axial load versus strain gage elevation for load transfer assessment.

Unit shaft resistance values can be used to evaluate nominal resistances of other similar size and type of piles (e.g. 12 inch or 16 inch O.D. in the above case) or to refine the pile penetration depth for the required nominal resistance. When performed in the design phase, foundation design can be optimized through a more complete understanding of load transfer mechanisms in the geomaterials.

9.5.3 Determination of Residual Load

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 shaft 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 Eq. 9-5 to 9-7 would not include residual loads. If measuring points R_1 and R_2 in Figure 9-20 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 9-28.

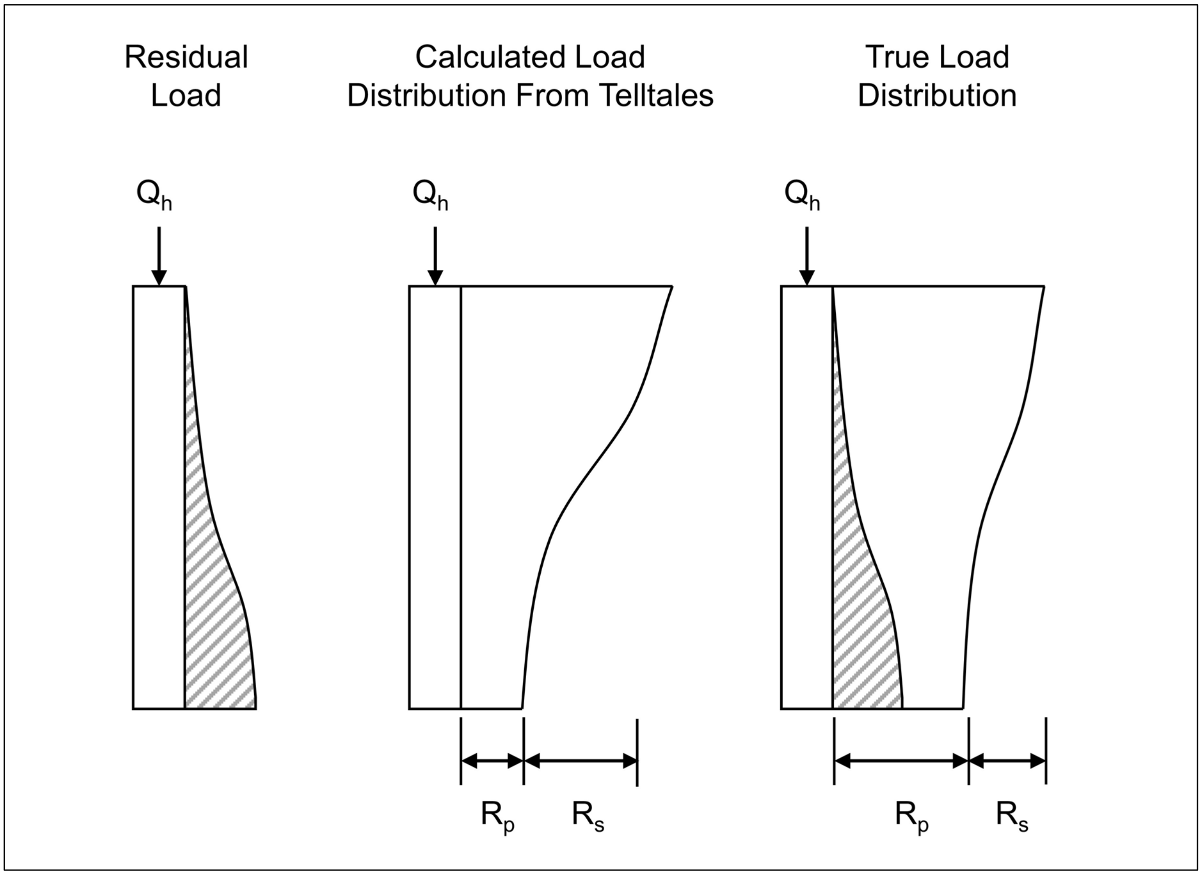


Figure 9-28   Example of residual load effects on load transfer evaluation.

In a static load test performed on a pile with residual loads, negative shaft resistance will be present along the upper portion of the pile. Figure 9-28 illustrates the minimal residual load in the upper portion of the pile which must be overcome before the true shaft resistance from the applied load can be measured. Moreover, residual loads are present along the pile shaft and at the pile toe before the static load test begins. This concept is illustrated for mobilized shaft resistance in Figure 9-29, Fellenius (2014). If no residual loads acted upon the pile prior to static load testing, Path O-B-C would represent the developed shaft resistance. However when residual loads are present, Point D is the origin of the residual load, which initially loads the pile along Path D-A. Thus, when applying load during the static load test, shaft resistance develops along Path A-O-B-C. If residual load is not accounted for, the loading path will be thought to move through Path A-B-C, and a shaft resistance twice as large as the true shaft resistance will be measured, Fellenius (2014).

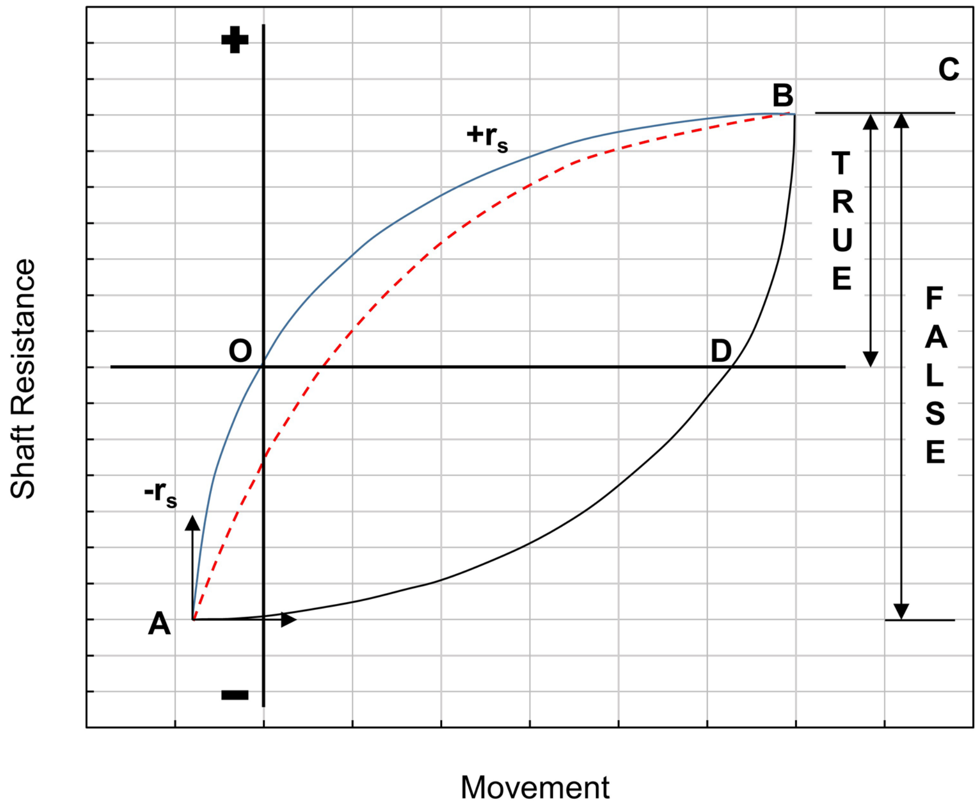


Figure 9-29 Mobilized shaft resistance hysteresis loop for pile during static load test (after Fellenius 2014).

Residual load can also develop at the pile toe. In Figure 9-30, Fellenius (2014), presents paths for toe resistance mobilization. Without the presence of residual load, toe resistance is developed along Path O-B-C. However when residual load exists within the pile, load develops along Path A-D-B-C. Toe resistance therefore develops prior to applying load during the static load test. In the above mentioned case of unloading, toe resistance is unloaded following the hysteresis loop of Path B-D'-A (when starting at Point B), where subsequent loading follows Path A-D-B-C. In summation, the residual load can result in underestimating the developed toe resistance.

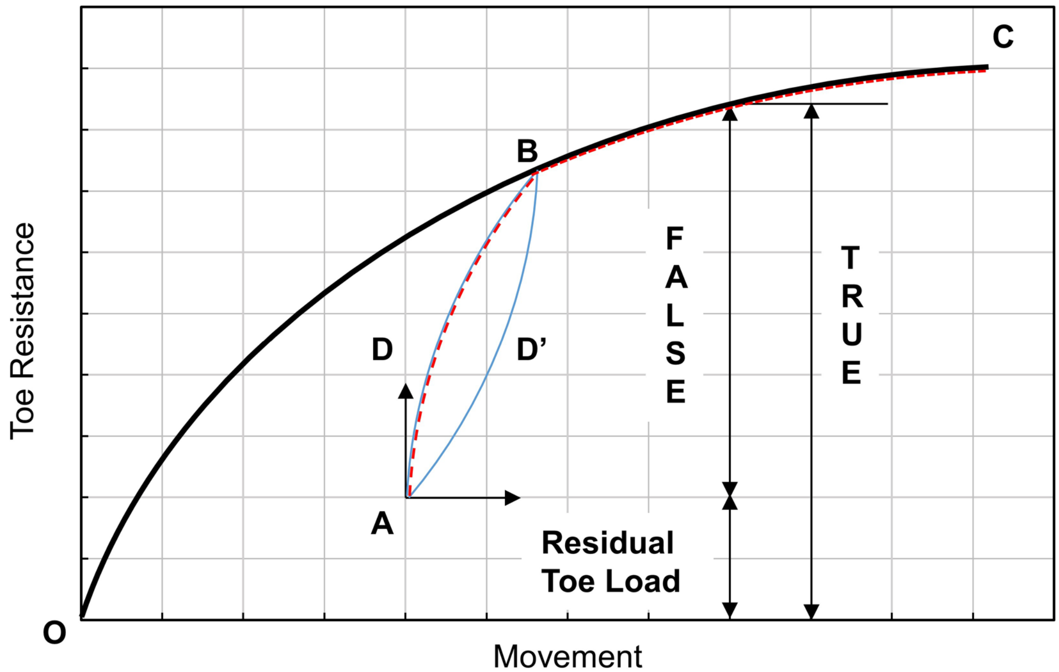


Figure 9-30 Mobilized toe resistance for pile during static load test. (after Fellenius 2014).

It is apparent from the above discussion that residual load locked into a pile can affect the apparent resistance distribution determined from a static load test. While the geotechnical nominal resistance is unaffected, the computed shaft and toe resistances are altered. Therefore, additional measures should be taken to determine the true resistance distribution and unit resistance values. This is particularly important when uplift resistance is being assessed from the shaft resistance determined in an instrumented compression load test or where unit resistance values are being used for design decisions.

Fellenius (2002b) presented a case where residual loads were evaluated from a strain gage instrumented static load test in conjunction with subsurface exploration results. When using strain gages for load-transfer in a static load test, it is important that the" zero reading" be made prior to pile installation (and for sister bars, the factory reading) such that no load exists in the pile. Fellenius et al. (2003) noted that a lower bound estimate of residual load may be obtained from the difference in strain measured just before commencement of the load test and at the true no load condition. This calculation of the residual load may however be an underestimate, in particular, when thermal strains occur in hydrating concrete.

Figure 9-31 presents strain gage instrumented static load test results for a prestressed concrete pile driven 48 feet into a loose to medium dense uniform sand. Reference should be made to Altaee et al. (1992) and Fellenius (2002b) for further details on the static load test or subsurface conditions. The "measured load" at a given depth is calculated using Eq. 9-7 and then that load subtracted from the applied pile head load to obtain the "measured load" versus depth.

In Figure 9-31 the measured load curve has an inflection point near a depth of 23 feet, and progresses downwards with increasing slope. This should not be the case in a uniform subsurface profile as it would require the unit shaft resistance to be smaller in the bottom third of the pile than in the middle portion of the pile. Therefore, residual load is likely to have influenced the true load distribution.

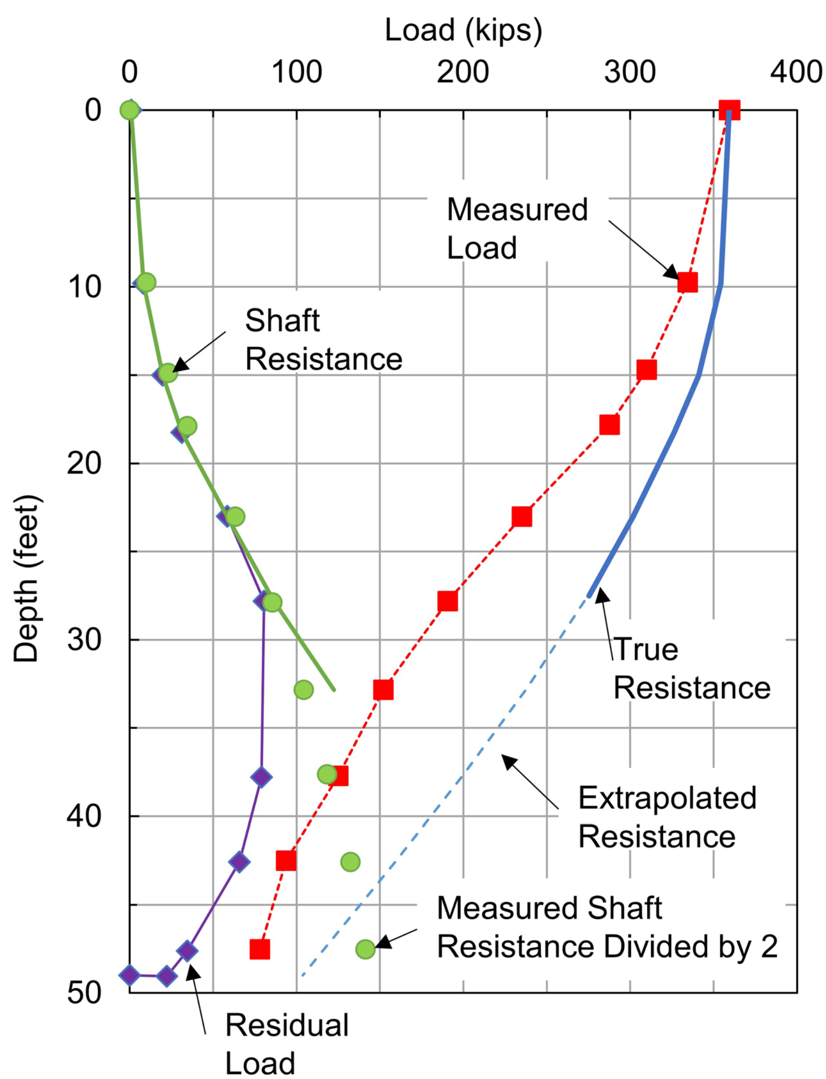


Figure 9-31 Instrumented static load test results and soil resistance distribution (modified from Fellenius 2002b with data from Altaee et al. 1992).

After plotting the measured load and calculating unit resistances along the pile, the measured shaft resistance divided by 2 (from the Measured Load) versus depth is added to the Figure 9-31 graph. The curve for the True Resistance then follows down to the inflection level (calculated as the difference in load applied at the pile head and the Measured Resistance divided by 2). Below this point, and because the uniform soil should exhibit similar shear strengths, the True Resistance curve may be extrapolated downward. In effect, it is assumed that the remaining unit shaft resistance is equal to that in the length just above this inflection, and does not follow the "false" measured values that indicate otherwise. The residual load is plotted as the difference of the True Resistance (with the extrapolated portion) and Measured Load. Engineering judgement and careful interpretation of the subsurface profile and load test results is necessary when determining residual load.

9.6 PRACTICAL ISSUES AND CONSIDERATIONS

Static load tests are the most reliable and most expensive method for determination of nominal geotechnical resistance or lateral deformation. Static load tests are sometimes avoided because of cost concerns or potential time delays in design or construction. While the economic benefits of performing a static load test should be carefully considered, cost alone should not be the deciding factor. Load transfer information from axial tests or deflected shape versus depth behavior in lateral load tests is extremely valuable when performed and evaluated by knowledgeable instrumentation specialists who should be involved in static load test program development and implementation.

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 load 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.

Fellenius (1984) reported on a static load test where the initial test was performed with the load determined from a load cell and jack pressure gage. In this test, the load determined from the load cell exceeded the load from the jack pressure gage and the accuracy of the test was in question. A second load test was performed after replacing the load cell and recalibrating the jack and load cell system. A comparison of the load from the jack and load cell in the second test is shown in Figure 9-32. The load determined from the jack pressure gage exceeded the load determined by the load cell by 10 to 20% in loading and underestimated the load by 5% in unloading. When load cell and jack pressure readings do not agree, the source of the discrepancy should be determined and the load test rerun if needed.

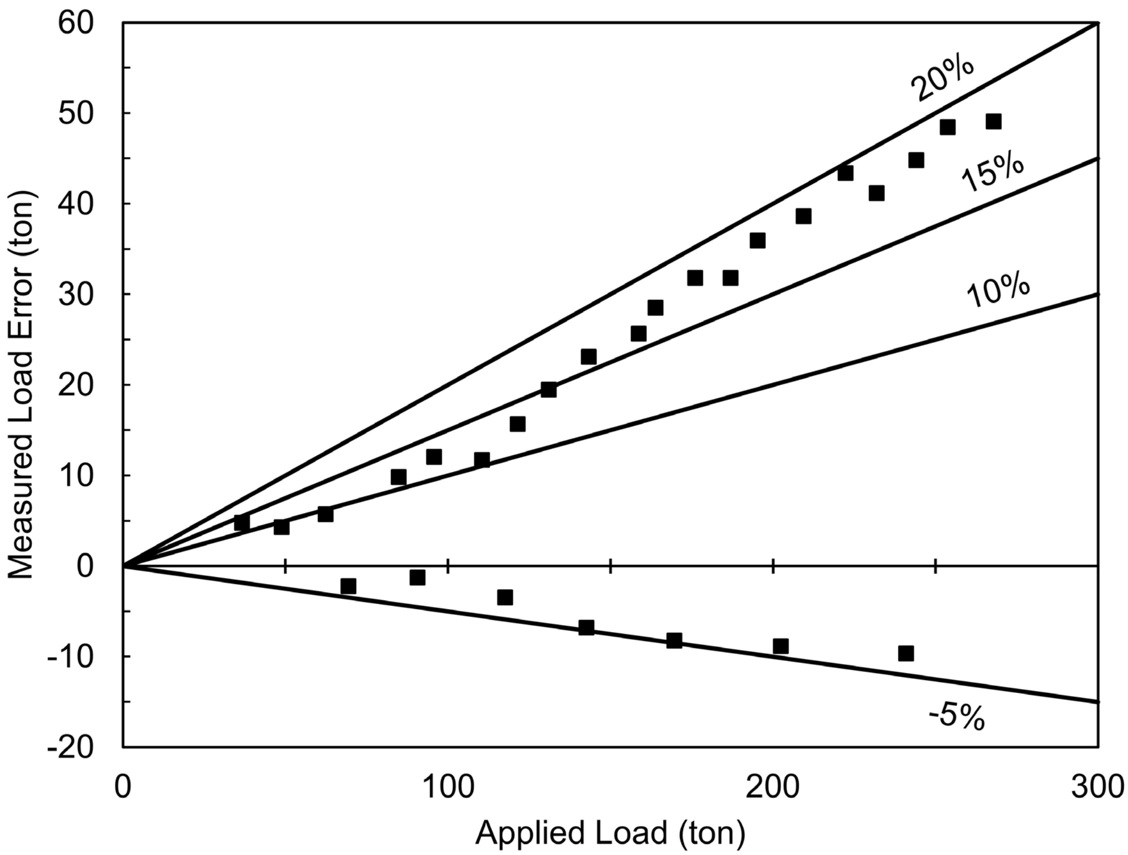


Figure 9-32 Variation between applied pile head load determined from load cell and jack pressure gage (after Fellenius 1984).

Load cells and jacks should also be properly sized to reduce measurement errors in the applied load. Geokon (2013) noted load cell error typically occurs if the load cell and hydraulic jack piston are not equal in diameter. The thickness of the bearing plates can further compound loading error, especially if the plate(s) is not sufficiently thick to reduce bending effects. In the Geokon study, three jack piston sizes were used while retaining the same load cell of size 4 inch I.D., 5-3/4 inch O.D with a maximum load of 300 kips. Furthermore, electrical resistance strain gages were bonded to the load cell's middle outside circumference, Sellers (2015). Jack A had dimensions of 2 inch I.D., 4 inch O.D., Jack B had dimensions of 4 inch I.D., 5-3/4 inch O.D., and Jack C had dimensions of 6 inch I.D, 8 inch O.D. Figure 9-33 summarizes size effects for the jack and bearing plate to the load cell response.

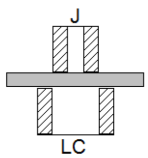
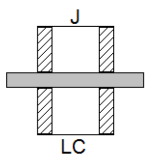
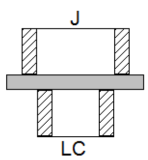
Jack		Load Cell response to applied load (100%)	
		1" thick plate	2" thick plate
A (smaller)		108%	102%
B (same size)		100%	100%
C (bigger)		96%	98%

Figure 9-33   Load cell response based on jack and bearing plate size (after Geokon 2013).

9.7 ADVANTAGES, DISADVANTAGES, AND LIMITATIONS

The advantages of performing static load tests are summarized below.

1. A static load test allows a more rational design. Confirmation of geomaterial resistance through static load testing is considerably more reliable than resistance estimates from static 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 factored pile load, either of which may result in potential savings in foundation costs.

3. With the improved knowledge of pile-soil behavior, a higher resistance factor may be used on the determined nominal resistance. Resistance factors between 0.75 and 0.80 are recommended when static load tests are utilized, as compared to a resistance factor of 0.40 when using the Modified Gates dynamic formula. Hence, a cost savings potential again exists.

4. The nominal resistance determined from load testing allows confirmation that the factored load may be adequately supported at the planned pile penetration depth.

Disadvantages include the load test cost, the time required to setup and perform a test, as well as the time required for complete result interpretation (particularly for tests with load transfer data). In addition, minimal information is obtained on driving stress levels or on the extent of pile damage (if any). Static load test results can also be misleading on projects with highly variable soil conditions. Site size and variability should therefore be considered when relying upon results for a single test pile.

As for limitations, 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.

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CHAPTER 10

DYNAMIC TESTING AND SIGNAL MATCHING 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 determine the performance of the pile driving system, calculate pile installation stresses, assess pile integrity, and evaluate the nominal geotechnical resistance.

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

10.1 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. The research work 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 transportation agencies, were also tested dynamically to obtain nominal resistance correlations. Finally, analysis method improvements were developed, including both field solutions (Case Method) in the Pile Driving Analyzer system (PDA) and an associated signal matching technique (CAse Pile Wave Analysis Program or CAPWAP). Additional information on the research project and its results may be found in Goble and Rausche (1970), Rausche et al. (1972), and Goble et al. (1975).

ODOT began to apply the results of this research to their construction projects in about 1968. Commercial use of the methods began in 1972 when the test equipment and analysis methods became practical for use in routine field testing by a trained engineer. Further implementation of dynamic testing methods in the 1980's resulted from FHWA Demonstration Project 66, in which additional correlation data was collected, and method benefits were demonstrated on transportation projects throughout the US.

Dynamic testing equipment and signal matching software have continued to be improved and enhanced over the 40 years since its original development. Other dynamic testing and analysis systems have also developed during that time period, primarily in Europe. One of the early European systems was advanced by the Netherlands Organization for Applied Scientific Research (TNO). This group developed the FPDS equipment and its associated signal matching technique, TNOWAVE, Reiding et al. (1988). Additional dynamic testing and analysis systems have also emerged over time including the Embedded Data Collector (EDC), Hererra et al. (2009), as well as the PDR dynamic testing system and Allwave-DLT signal matching software, Middendorp and Verbeek (2010).

10.2 APPLICATIONS FOR DYNAMIC TESTING METHODS

Samtani and Nowatzki (2006) note that dynamic testing costs much less and requires less time than static pile load testing. They also note that important information can be obtained regarding the behavior of the pile driving system and pile-soil response that is not available from a static pile load test. Determination of driving stresses and pile integrity with dynamic test methods has resulted in fewer, higher nominal resistance piles in foundation designs due to better pile installation control. Some of the applications for dynamic testing methods are discussed below.

10.2.1 Nominal Resistance

a. Assessments of nominal geotechnical resistance versus pile penetration depth can be obtained by testing from the start to the end of driving. This can be helpful in profiling the depth to the bearing stratum and thus the required production pile lengths.

b. Evaluation of the nominal geotechnical resistance at the time of testing. Soil setup or relaxation potential can be assessed by restrike testing several piles and comparing nominal resistance at restrike with that at end-of-initial driving.

c. Signal matching analysis provides refined estimates of nominal resistance, the soil resistance distribution, as well as insight into soil quake and damping parameter selection for wave equation analyses. Signal matching and wave equation analysis programs have different pile and soil models. Therefore, signal matching determined dynamic soil parameters may require adjustment, as described in Section 12.6.9, for input in wave equation analysis programs.

10.2.2 Hammer and Driving System Performance

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

b. Determination of drive system performance under different hammer strokes, operating pressures, batter angles, 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 issues, such as pre-ignition problems with diesel hammers or preadmission in air/steam hammers.

d. Determination of whether soil behavior or hammer performance is responsible for changes in the observed penetration resistance or blow count.

10.2.3 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. Compression and tension stress distribution throughout the pile obtained from signal matching.

10.3 RESISTANCE FACTORS FOR DYNAMIC TESTING

AASHTO (2014) design specifications provide resistance factors for dynamic pile testing with signal matching. When the driving criteria is established by dynamic testing with signal matching on two piles per site condition, but no less than 2% of the production piles, the AASHTO specified resistance factor is 0.65. When the driving criteria is established by dynamically testing with signal matching on 100% of the production piles, the AASHTO specified resistance factor is 0.75. The AASHTO resistance factor is 0.80 when the driving criteria is established by a successful static load test of one pile per site condition in conjunction with dynamic testing with signal matching on two piles per site condition but no less than 2% of the production piles. When a dynamic test with signal matching is used for determination of the nominal resistance in axial tension, the AASHTO specified resistance factor is 0.50.

AASHTO resistance factors for dynamic testing with signal matching were originally developed from data presented in NCHRP Report 507, Load and Resistance Factor Design (LRFD) for Deep Foundations, Paikowsky (2004). The database used to develop the recommended resistance factors defined the nominal resistance determined by a static load test as the load where the static load test load-deflection curve exceeded the Davisson offset limit. Similarly, the nominal resistance from dynamic testing with signal matching was defined as the nominal resistance during restrike determined by the CAPWAP signal matching program. If a different static load test interpretation criterion or a different signal matching analysis method is used, modification or local calibration of the resistance factor should be considered.

The Florida Department of Transportation sponsored a research effort by McVay and Wasman (2015) to determine the resistance factor for dynamic tests performed with the Embedded Data Collector (EDC) system. Resistance factors were calculated using First Order Second Moment (FOSM) principles, and using the UF and the Tran methods. The research study recommended that the calculated resistance values be considered as preliminary due to the limited size of the database.

10.4 DYNAMIC TESTING

A typical dynamic testing system consists of a minimum of two strain transducers and two accelerometers. The reusable gages are externally bolted to diametrically opposite sides of the pile at a location two to three diameters below the pile head. These gages measure strain and acceleration, and account for non-uniform hammer impacts and pile bending.

Two diametrically opposite mounted strain transducers are required for a valid dynamic test to average out and compensate for the influence of non-uniform impacts and bending. All driven pile types (prestressed concrete piles; steel pipe, H, Monotube, and Tapertube piles; timber piles; and composite piles) can be easily tested using external gages with the pile preparation and gage attachment procedures varying slightly for each pile type.

Figure 10-1 illustrates the typical pile preparation procedures required for dynamic testing using a reusable external gage system. In Figure 10-1(a), a prestressed concrete pile is being prepared for external gage attachment using a hammer drill to create holes in the concrete. The holes for the strain transducers are drilled through a template to maintain location tolerance. Concrete anchors are then set into the drilled holes and the gages are bolted to the concrete anchors. Removal of the concrete dust remaining in the drilled holes prior to setting the concrete anchors improves anchor bond with the concrete. For steel pipe, Monotube, and Tapertube piles, diametrically opposite holes are drilled into the steel as shown in Figure 10-1(b) and tapped. External gages are then attached using high strength bolts inserted in the threaded holes. For steel H-piles, holes are drilled holes through the web as shown in Figure 10-1(c) and bolts are used to attach gages on both opposite web faces. Wood lag screws are used to attach external gages on timber piles.

Pile preparation and gage attachment typically requires 15 to 20 minutes for each pile to be tested. After the gages are attached, the pile driving or restrike process continues following usual procedures. For restrike tests, the pile can be drilled and gages attached at any convenient location 2 or more diameters below the pile head. Drilling near the pile head or reusing the original gage holes at that location is not necessary. Most restrike tests are typically on the order of 20 blows or less.

Figure 10-1 Pile preparation for dynamic testing.

A photograph of an externally mounted accelerometer, strain transducer, and Wi-Fi transmitter bolted to a steel H-pile is shown in Figure 10-2(a). Signals can be transmitted from these gages either via the Wi-Fi transmitter shown, or by a splitter cable and main cable that collects and transfers signals from the individual gages. System manufacturers also offer a combined strain transducer and accelerometer as shown mounted on a pipe pile in Figure 10-2(b). The transmitter or signal collection cable is not visible in the photograph. Dynamic test records from either gage arrangement are acquired for every hammer blow and transmitted wirelessly or by main cable to the data acquisition system.

On concrete piles, dynamic testing can be performed using either reusable external gages or embedded gages. An embedded gage set typically consists of one strain transducer and one accelerometer cast into the pile at a distance of two to three diameters below the pile head. Only one strain transducer and one accelerometer is required in this situation, provided the embedded gage set is located on the central

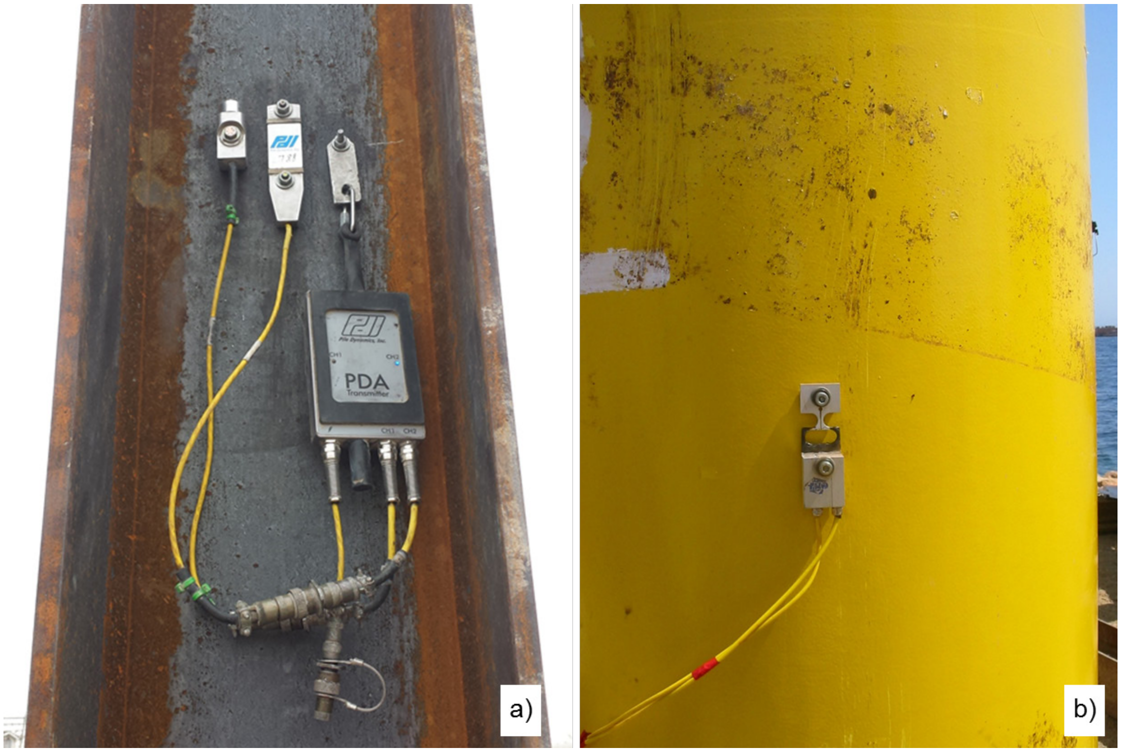


Figure 10-2 a) Accelerometer, strain gage, and WiFi transmitter (courtesy Pile Dynamics) and b) combined strain gage and accelerometer (courtesy Allnamics).

axis of the pile, such that bending and non-uniform impacts are eliminated. Non-uniform concrete piles, such as those with voided center sections, complicate use of embedded gages if the change in cross section occurs near the embedded gage location. Embedded gages should be located at least one diameter from the cross sectional change. An external gage system with the gages placed on the voided section below the cross section change is recommended in this situation and for concrete cylinder piles where a central axis gage location is not possible.

An embedded gage system consisting of a strain transducer and accelerometer being cast into a concrete pile is shown in Figure 10-3. The object on the right side sitting atop the casting bed houses the transmitter which will be cast flush mounted into the pile surface. The wire on the left side connects the transmitter to a second embedded gage set cast into the pile near the pile toe.

Figure 10-4 illustrates another embedded gage arrangement consisting of a sister bar mounted strain gage and accelerometer being cast into a concrete pile. Two diametrically opposite sister bar mounted strain gages are also shown in the figure. The additional diametrically opposite sister bar strain gages are not standard and were installed for measurement comparison with the center mounted gage.

Figure 10-3 Embedded strain gage and accelerometer unit being cast into concrete pile (courtesy Radise International).

Figure 10-4 Embedded resistance strain gage and accelerometer mounted on sister bar being cast into concrete pile (courtesy of Pile Dynamics, Inc.).

Embedded gages can also be cast into concrete piles at locations other than near the pile head. These additional embedded gages can be monitored during driving concurrently with gages located near the pile head for further insight into driving stresses, load transfer, or potential pile toe damage.

Depending on the dynamic testing system, data can be wirelessly sent from the pile to the processing unit using a reusable transmitter as shown in Figure 10-2(a), or using a transmitter cast into a concrete pile with the embedded gages.

The data acquisition system conditions and converts the measured 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*, using Equation 10-1.

$$F(t) = E\,A\,\varepsilon(t) \qquad \text{Eq. 10-1}$$

Where:

$F(t)$ = force computed at the gage location at time t (kips).
E = elastic modulus of pile material (ksi).
A = pile cross sectional area (in^2).
$\varepsilon(t)$ = measured strain at time t.

The velocity is obtained by integrating the measured acceleration record over time using Equation 10-2.

$$V(t) = \int a(t)dt \qquad \text{Eq. 10-2}$$

Where:

$V(t)$ = velocity computed at gage location at time t (ft/s).
$a(t)$ = measured acceleration at gage location at time t (ft/s^2).

In most dynamic testing systems, all components for processing, storing, and displaying dynamic test signals are combined into either a dedicated field processing unit such as the Pile Driving Analyzer (PDA) shown in Figure 10-5 or a laptop computer such as the Embedded Data Collector (EDC) shown in Figure 10-6.

During driving, these systems perform integrations and all other required computations to analyze the acquired dynamic records for transferred energy, driving stresses, structural integrity, and nominal geotechnical resistance. Numerical results for user selected dynamic quantities are also displayed with each blow in real time. Basic force and velocity records as well as other results can be viewed on the system screen during the test. Processed test records are digitally stored and then used for subsequent signal matching analysis performed on-site or in the office as well as for graphical and numeric output summaries.

Figure 10-5   Pile Driving Analyzer processing unit (courtesy of Pile Dynamics, Inc.).

Figure 10-6   Embedded Data Collector system (courtesy of Radise International).

10.5 BASIC WAVE MECHANICS

This section is intended to summarize basic wave mechanics principles applicable to pile driving. Through this general overview, an understanding of dynamic testing concepts and how dynamic 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 10-7(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 10-7(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 10-7(c) and 10-7(d). Since there are no resistance effects acting on the rod, a free end condition exists. A tensile wave reflection occurs at a free end 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.

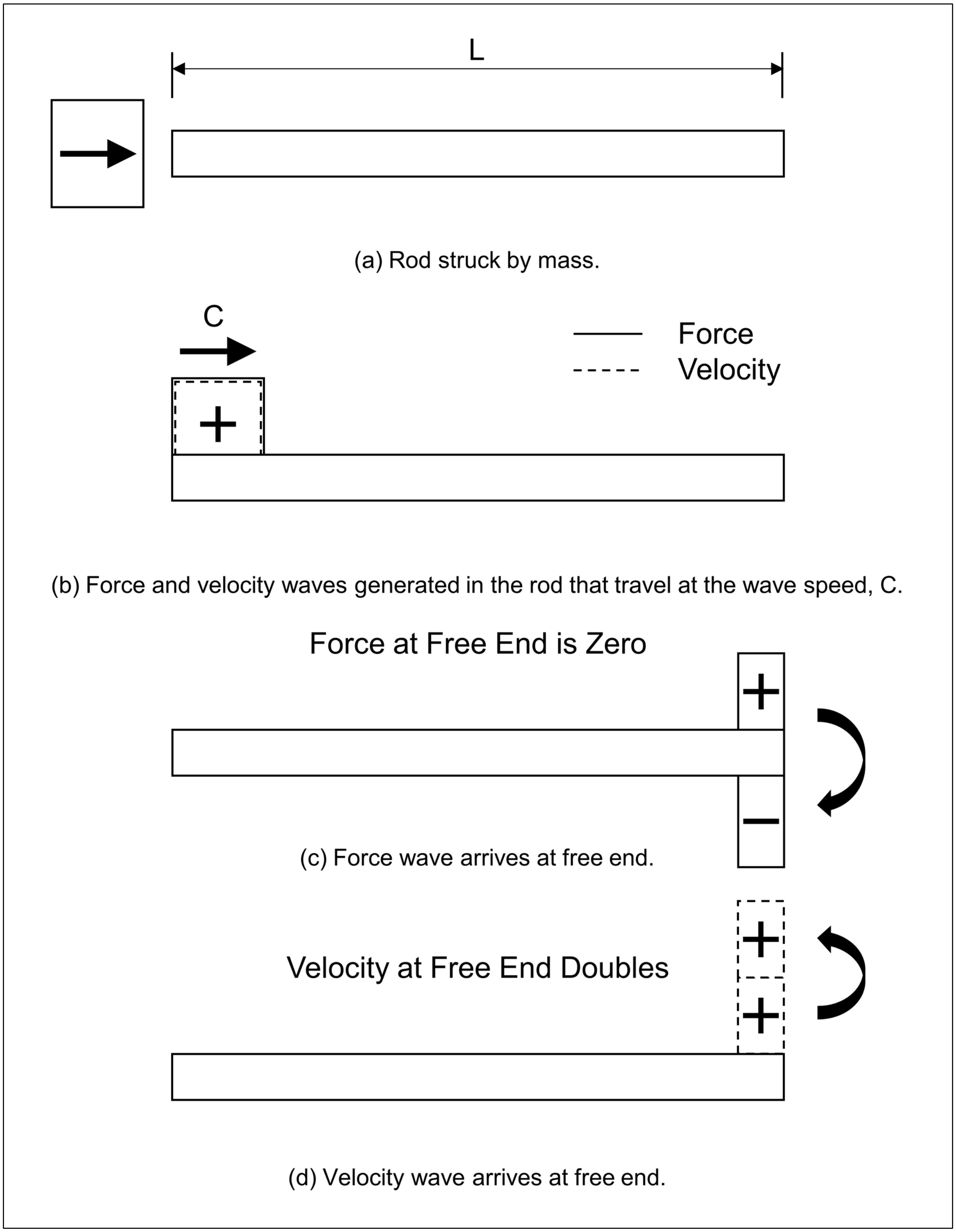


Figure 10-7 Free end wave mechanics.

Consider now that the rod is a pile with no resistance effects, and that force and velocity records are obtained from measurements made near the pile head. A typical force and velocity record versus time for this "free end" condition is presented in Figure 10-8.

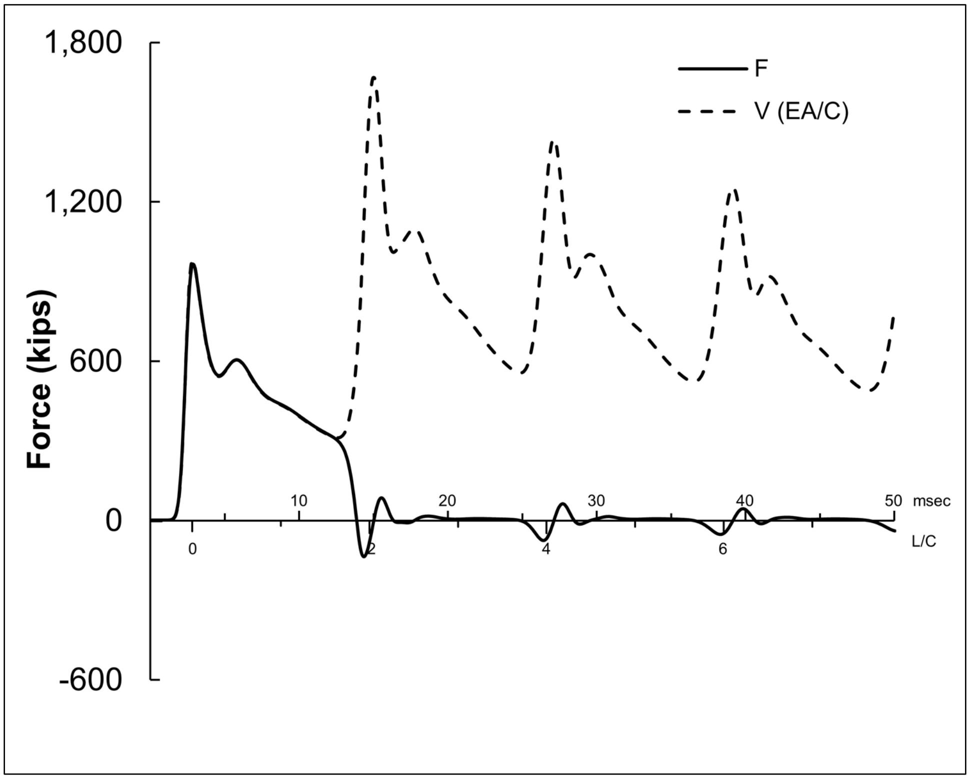


Figure 10-8 Force and velocity times (EA/C) records versus time for free end.

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 10-9(a) illustrates a uniform rod of length, *L*, 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 material wave speed, *C*. At time L/C, the waves will arrive at the end of the rod as shown in Figures 10-9(c) and 10-9(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 records are again obtained from measurements made near the pile head. The force and velocity records versus time for this condition are presented in Figure 10-10. 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.

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.

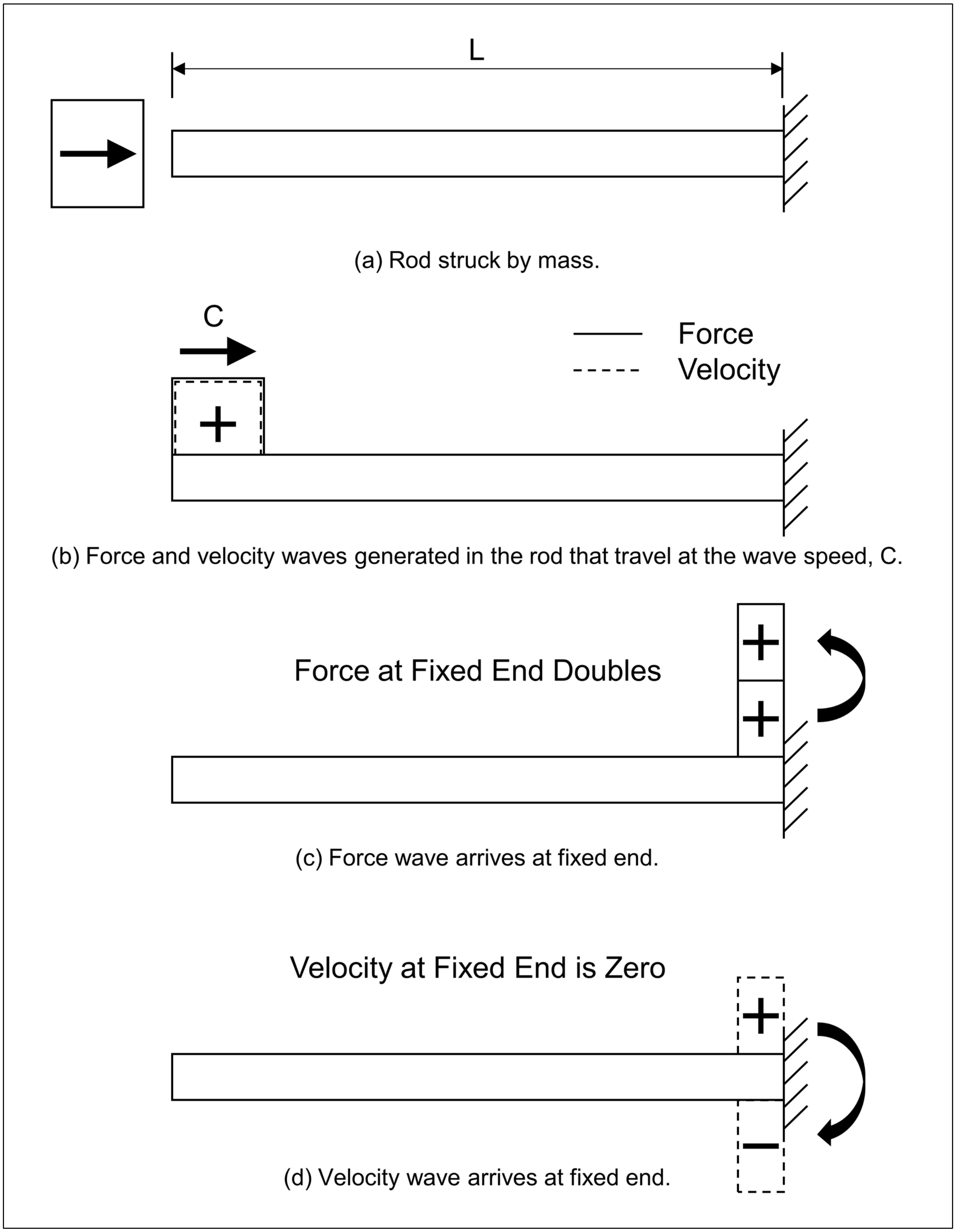


Figure 10-9 Fixed end wave mechanics.

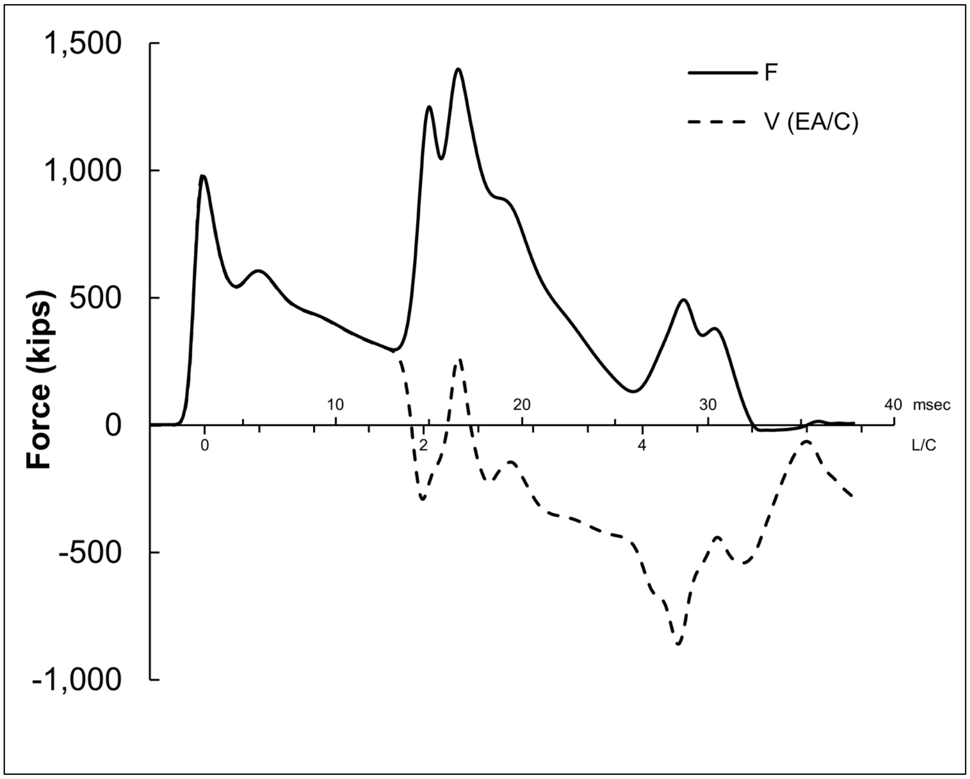


Figure 10-10 Force and velocity times (EA/C) records versus time for fixed end.

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 10-11. 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 soil 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 are presented in the lower portion of the figure. The onset of impact occurs when the force and velocity records rise together prior to time 0. The time interval from the onset of impact until the peak impact velocity is referred to as the rise time. The force and velocity records remain proportional or equal until one rise time before time 2A/C. At that time, the reflection from the small soil resistance effect begins to arrive at the measurement location. This soil resistance reflection causes the small increase in the force record and the small decrease in the velocity record at time 2A/C.

No additional soil resistance effects act on the pile between time 2A/C and time 2B/C. Therefore, the force and velocity records will remain parallel over this time interval with no additional separation. At one rise time prior to time 2B/C, the

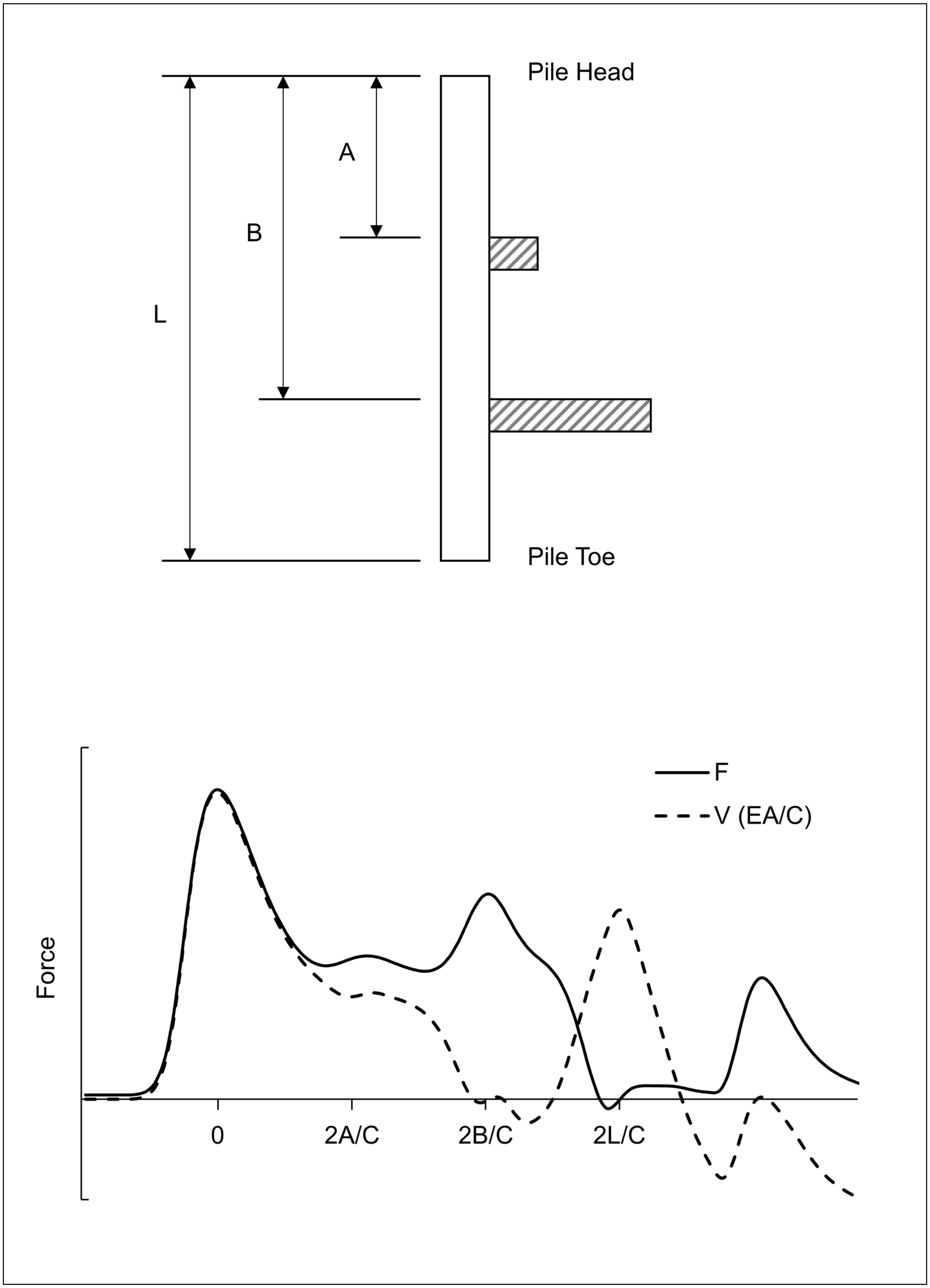


Figure 10-11 Soil resistance effects on force and velocity records
(after Hannigan 1990).

reflection from the large soil resistance effect will begin to arrive at the measurement location. This large soil resistance reflection then causes the large increase in the force record and the large decrease in the velocity record at time 2B/C. No additional soil resistance effects act on the pile between time 2B/C and time 2L/C. Therefore, the force and velocity records exhibit no additional separation over this time interval.

At one rise time before 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 is reflected. Hence, an increase in the velocity record and a decrease in the force record occurs.

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 10-12(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 minimal toe resistance on the pile.

In Figure 10-12(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 10-12(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.

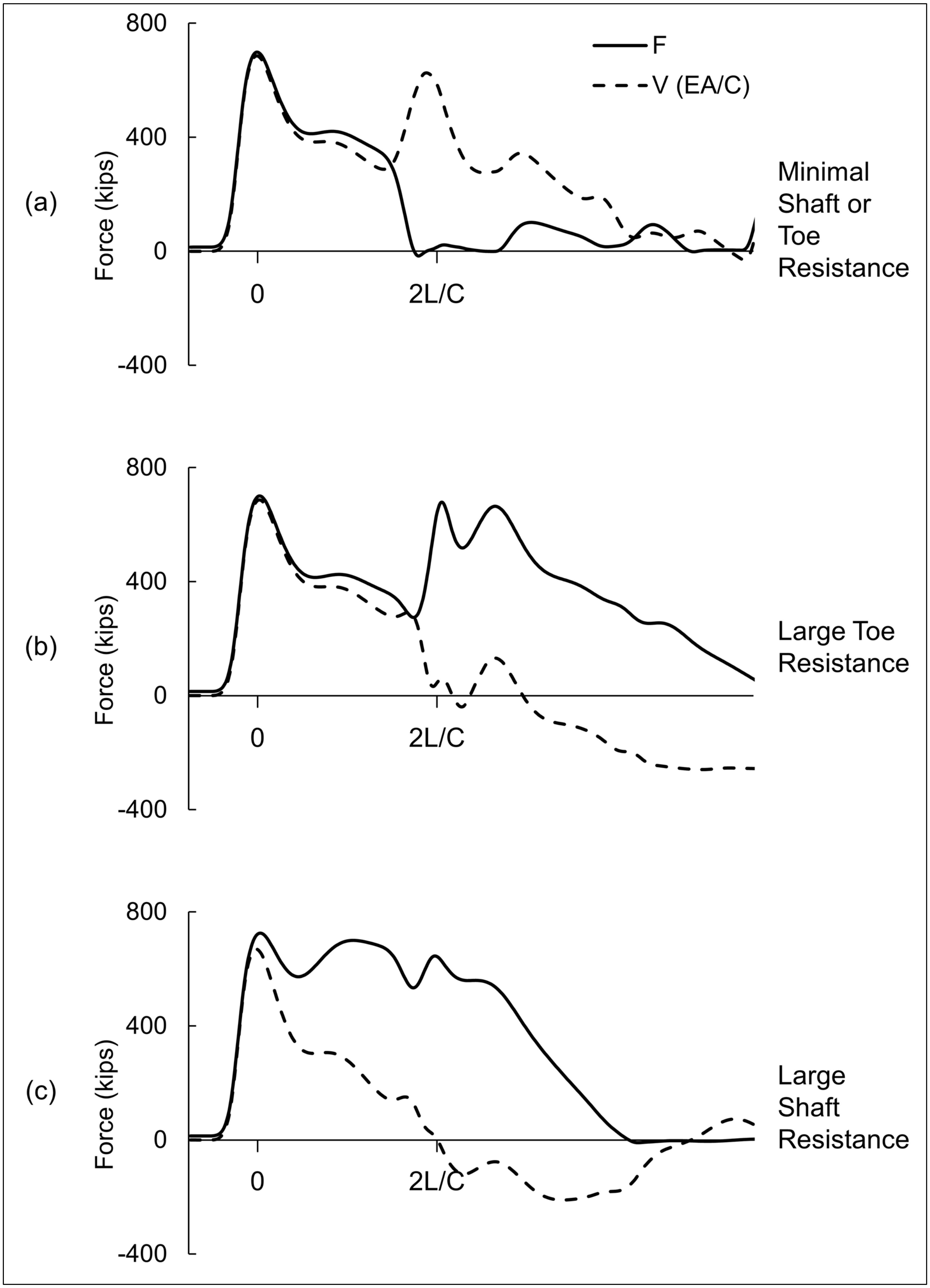


Figure 10-12 Typical force and velocity records for various soil resistance conditions.

10.6 DYNAMIC TESTING METHODOLOGY

As introduced in Section 10.1, simple field solutions such as the Case Method, and more rigorous numerical modeling or signal matching methods such as CAPWAP have been developed for analyzing dynamic measurement data. These dynamic test methods are sometimes confused with wave equation analysis described in Chapter 12, so it is useful to briefly review dynamic test and analysis methods as well as their application.

The wave equation is a computer program that is typically performed prior to field work. During the design stage, a wave equation drivability analysis is performed to check the suitability of potential pile types and sections. During the construction phase, the wave equation is used to check the suitability of the contractor's proposed equipment to satisfy nominal resistance and penetration requirements. In either application, the program inputs require the engineer to make assumptions on the hammer performance and static and dynamic soil response. Following test pile installation, a refined wave equation analysis may also be performed. In a refined analysis, the engineer uses hammer performance and soil response information from dynamic measurements and signal matching results to "calibrate" the wave equation to the field conditions. This process is described in Section 12.6.9. The wave equation provides a relationship between the nominal geotechnical resistance and the pile penetration resistance or blow count. It is therefore often used in establishing the driving criteria or in assessing the nominal geotechnical resistance of a pile based on its observed pile penetration resistance.

During test pile or production driving, dynamic measurements are made for estimates of the nominal geotechnical resistance. During driving, Case Method results for nominal resistance are calculated in real time from the measured force and velocity records obtained for each hammer blow. The Case Method equations for nominal resistance are described in detail in Section 10.6.1. While these simple field methods are useful for assessing the nominal resistance, AASHTO (2014) requires the nominal resistance be determined through signal matching analysis. Additional Case Method equations are used for calculation of driving stresses and pile integrity, as well as computation of transferred hammer energy. These additional Case Method equations are also described later in this chapter.

Signal matching is a more rigorous numerical analysis procedure that uses the measured force and velocity records from one hammer blow. Signal matching programs use the dynamic measurement data along with wave equation type pile and soil modeling to calculate the nominal geotechnical resistance, the relative soil resistance distribution, the dynamic soil properties of quake and damping, and compression and tension stresses throughout the pile. The nominal resistance determined by signal matching is a more accurate assessment of the nominal resistance than Case Method results. Signal matching determined soil information along with the dynamic test data on driving system performance are often used in the development of a refined wave equation analysis. This is the best use of all the three methods for driving criteria determination. Signal matching analysis is described in greater detail in Section 10.6.6.

10.6.1 Nominal Resistance Determination by Case Method

Research conducted at Case Western Reserve University in Cleveland, Ohio resulted in a method which uses electronic measurements taken during pile driving to predict nominal resistance. 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:

$$RTL = \frac{1}{2}[F(t_1) + F(t_2)] + \frac{1}{2}[V(t_1) - V(t_2)](\frac{EA}{C}) \qquad \text{Eq. 10-3}$$

Where:

RTL= total static and dynamic resistance on pile during driving (kips).
$F(t)$ = force measured at gage location at time t (kips).
$V(t)$ = velocity measured at gage location at time t (ft/s).
t_1 = time of first peak input.
t_2 = time of reflection of first peak input from pile toe $(t_1 + 2L/C)$.
L = pile length below gage location (feet).
E = elastic modulus of pile material (ksi).
A = pile cross sectional area (in^2).
C = wave speed of pile material (ft/s).

To obtain the static nominal resistance, 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 nominal resistance equation, *RSP*, expressed as follows:

$$RSP = RTL - J\left[V(t_1)\left(\frac{EA}{C}\right) + F(t_1) - RTL\right] \qquad \text{Eq. 10-4}$$

Where:

RSP= standard Case Method equation for static nominal resistance (kips).
RTL= total static and dynamic resistance on pile during driving (kips).
J = Case damping factor based on soil type near the pile toe (unit less).
$F(t)$ = force measured at gage location at time t (kips).
$V(t)$ = velocity measured at gage location at time t (ft/s).
t_1 = time of first peak input.
E = elastic modulus of pile material (ksi).
A = pile cross sectional area (in^2).
C = wave speed of pile material (ft/s).

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 nominal resistance 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. (2015), are presented in Table 10-1. While use of these values may provide good initial estimates of the nominal resistance, site specific damping correlations should be developed based upon signal matching analysis or static load test results. 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 12 for wave equation analysis.

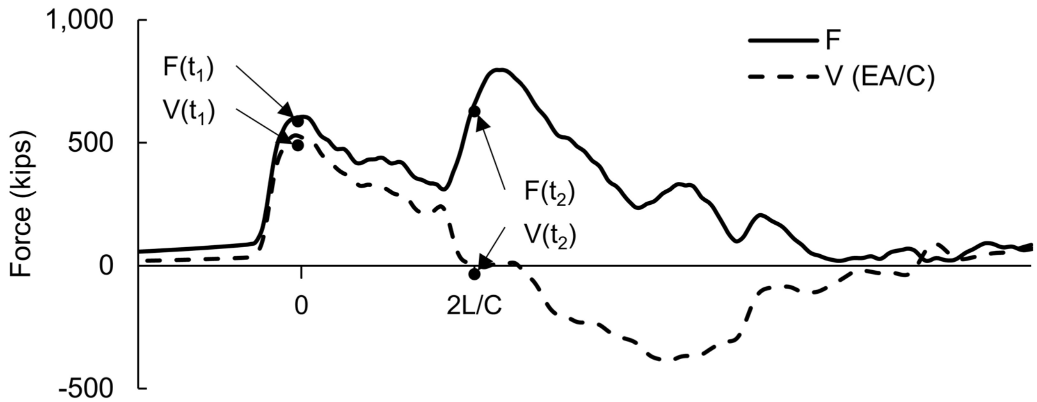
The RSP or standard Case Method equation is best used to evaluate the nominal resistance of low displacement piles, and piles with large shaft resistances. For displacement piles driven in soils with large toe quakes and for piles with large toe resistances, the maximum toe resistance is often delayed 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 nominal resistance 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 nominal resistance. An example of this technique is presented in Figure 10-13. 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 to 0.4 higher than that used for the standard Case Method equation for nominal resistance, RSP. Typical ranges in Case damping factor for the RMX equation are also presented in Table 10-1.

Table 10-1 Summary of Case Damping Factors for RSP and RMX Equations

Soil Type at Pile Toe	Original Case Damping Correlation Range Goble et al. (1975)	Recommended Range in Case Damping Constant for RSP Equation Pile Dynamics (2015)	Recommended Range in Case Damping Constant for RMX Equation Pile Dynamics (2015)
Clean Sands	0.05 to 0.20	0.10 to 0.15	0.40 to 0.50
Silty Sands	0.15 to 0.30	0.15 to 0.25	0.50 to 0.70
Silts	0.20 to 0.45	0.25 to 0.40	0.60 to 0.80
Silty Clays	0.40 to 0.70	0.40 to 0.70	0.70 to 0.90
Clay	0.60 to 1.10	0.70 or higher	0.90 or higher

The RMX and RSP Case Method equations are the two most commonly used solutions for field evaluation of pile nominal resistance. 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. These automatic methods in their appropriate condition are often helpful supplemental indicators of the nominal resistance with the more traditional maximum or standard Case Method equations.

While the above Case Method equations are valuable for a quick field assessment of the nominal resistance, AASHTO (2014) does not provide a resistance factor for these or any other simple direct methods. Signal matching analysis on the collected dynamic test data is required for all nominal resistance assessments to be in compliance with AASHTO specifications.

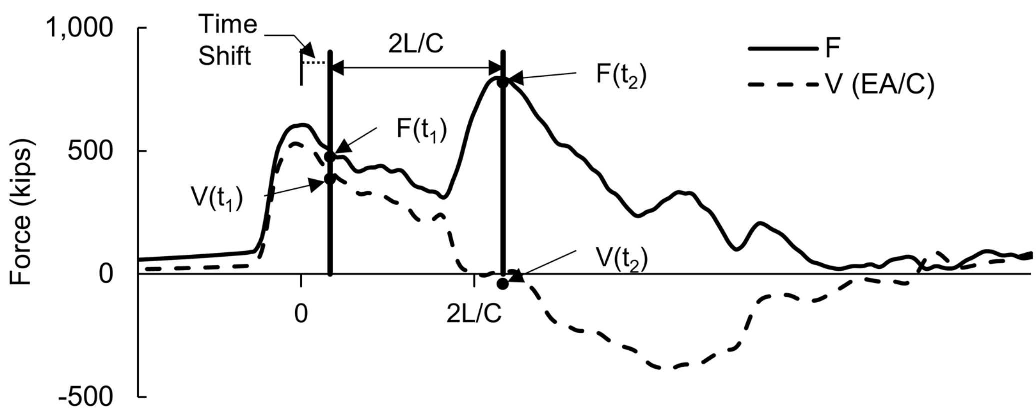


<u>TOTAL RESISTANCE</u>

RTL $= ½ [F(t_1) + F(t_2)] + ½ [V(t_1) – V(t_2)] (EA/C)$
$= ½ [599 + 587] + ½ [13.6 – 0.3] 38.9$
$= 593 + 259 = 852$ kips

<u>STATIC RESISTANCE</u>

RSP $= RTL – J [V(t_1)*(EA/C) + F(t_1) – RTL]$
$= 852 – 0.4 [13.6 * 38.9 + 599 – 852]$
$= 852 – 110 = 742$ kips



<u>TOTAL RESISTANCE</u>

RTL $= ½ [F(t_1) + F(t_2)] + ½ [V(t_1) – V(t_2)] (EA/C)$
$= ½ [529 + 795] + ½ [11.7 – 0.1] 38.9$
$= 662 + 226 = 888$ kips

<u>STATIC RESISTANCE</u>

RMX $= RTL – J [V(t_1)*(EA/C) + F(t_1) – RTL]$
$= 888 – 0.7 [11.7 * 38.9 + 529 – 888]$
$= 888 – 67 = 821$ kips

Figure 10-13 Examples of standard Case Method, RSP, and maximum Case Method, RMX, calculations for estimates of nominal resistance.

10.6.2 Soil Resistance Distributions

As noted in Section 10.6, soil resistance effects can be assessed from dynamic test records. The relative magnitude of the soil resistance can be evaluated from the difference between the force record and velocity times impedance records at a given time during the 2L/C time interval following impact. The depth of the soil resistance below the gage location can be determined from the reflection time, 2X/C where X is the depth and C is the pile wave speed. While relative soil resistance distribution effects can be evaluated in this manner, the magnitude of the soil resistance at a given depth should be determined from more rigorous signal matching analysis as described in Section 10.6.6.

For piles with externally mounted or embedded strain gages that are dynamically monitored during driving, the pile forces calculated by measurements at the monitored locations can be compared to the calculated forces at those depths in signal matching results. However, the presence of residual forces from pile casting and/or pile driving greatly complicate any simple closed form solution of the soil resistance distribution from externally mounted or embedded instrumentation.

10.6.3 Energy Transfer

The energy transferred to the pile head can be computed from the strain and acceleration measurements. As described in Section 10.3, 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:

$$E_p(t) \int_0^t F(t)\, V(t)\, dt \qquad \text{Eq. 10-5}$$

Where:

$E_p(t)$	=	energy computed at the gage location at time t (ft-kips).
$F(t)$	=	force measured at gage location at time t (kips).
$V(t)$	=	velocity measured at gage location at time t (ft/s).

This procedure is illustrated in Figure 10-14. The maximum energy transferred to the pile head corresponds to the maximum value of $E_p(t)$. The output quantity EMX is 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 10.7.

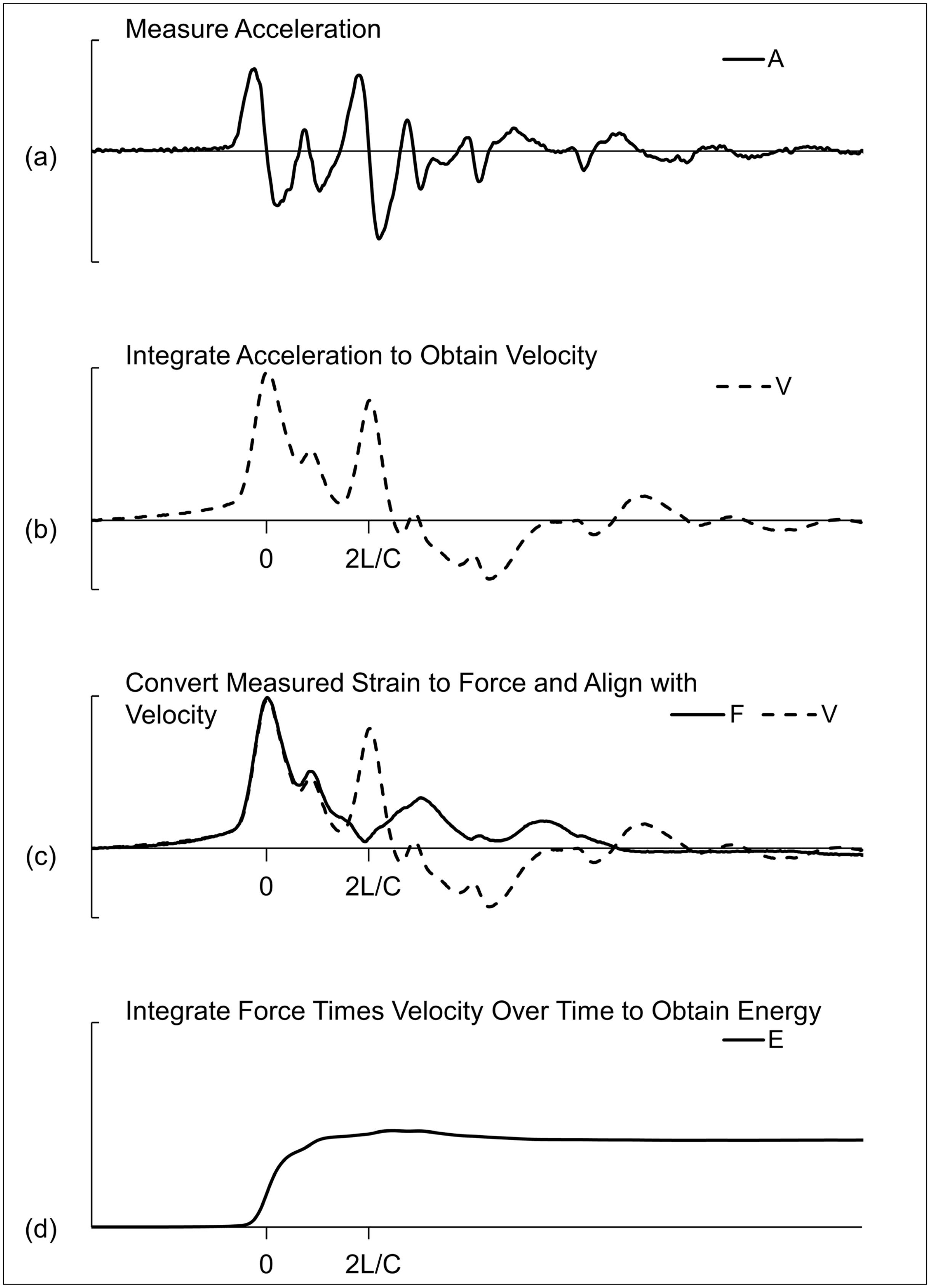


Figure 10-14 Energy transfer computation.

10.6.4 Driving Stresses

The compression stress at the gage location can be calculated 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, signal matching or wave equation analysis may be used to evaluate the maximum compression stress elsewhere in the pile. Figure 10-15 illustrates the computation process for compression and tension stresses. Force and velocity records for an 18 inch square prestressed concrete pile with a large toe resistance are presented in the top half of the figure. The penetration resistance associated with this record is 29 blows per inch. The vertical scale between the zero axis and the top of the window box is identified as 1500 kips. Point A identifies the maximum compression force at the gage location of 795 kips. The maximum compression stress at the gage location, CSX, is then this force of 795 kips divided by the pile cross section area of 324 in^2 or 2.45 ksi.

Computed tension stresses are based upon the superposition of the upward and downward traveling waves. The downward traveling wave, wave down identified as WD, and the upward traveling wave, wave up identified as WU, are presented in the lower half of Figure 10-15. The vertical scale between the zero axis and the top of the lower box is once again 1500 kips. The value of wave down, WD, at time (t) is computed from the measured force and velocity records according to:

$$WD\ (t) = \frac{1}{2}\left[F(t) + V(t)(\frac{EA}{C})\ \right] \qquad \text{Eq. 10-6}$$

Where:

$WD(t)$	=	downward traveling wave, wave down, at time t (kips).
$F(t)$	=	force measured at gage location at time t (kips).
$V(t)$	=	velocity measured at gage location at time t (ft/s).
E	=	elastic modulus of pile material (ksi).
A	=	pile cross sectional area (in^2).
C	=	wave speed of pile material (ft/s).

The value of wave up, WU, at time (t) is computed from the measured force and velocity records according to:

$$WU\ (t) = \frac{1}{2}\left[F(t) - V(t)(\frac{EA}{C})\ \right] \qquad \text{Eq. 10-7}$$

Where:

$WU(t)$ = upward traveling wave, wave up, at time t (kips).
$F(t)$ = force measured at gage location at time t (kips).
$V(t)$ = velocity measured at gage location at time t (ft/s).
E = elastic modulus of pile material (ksi).
A = pile cross sectional area (in^2).
C = wave speed of pile material (ft/s).

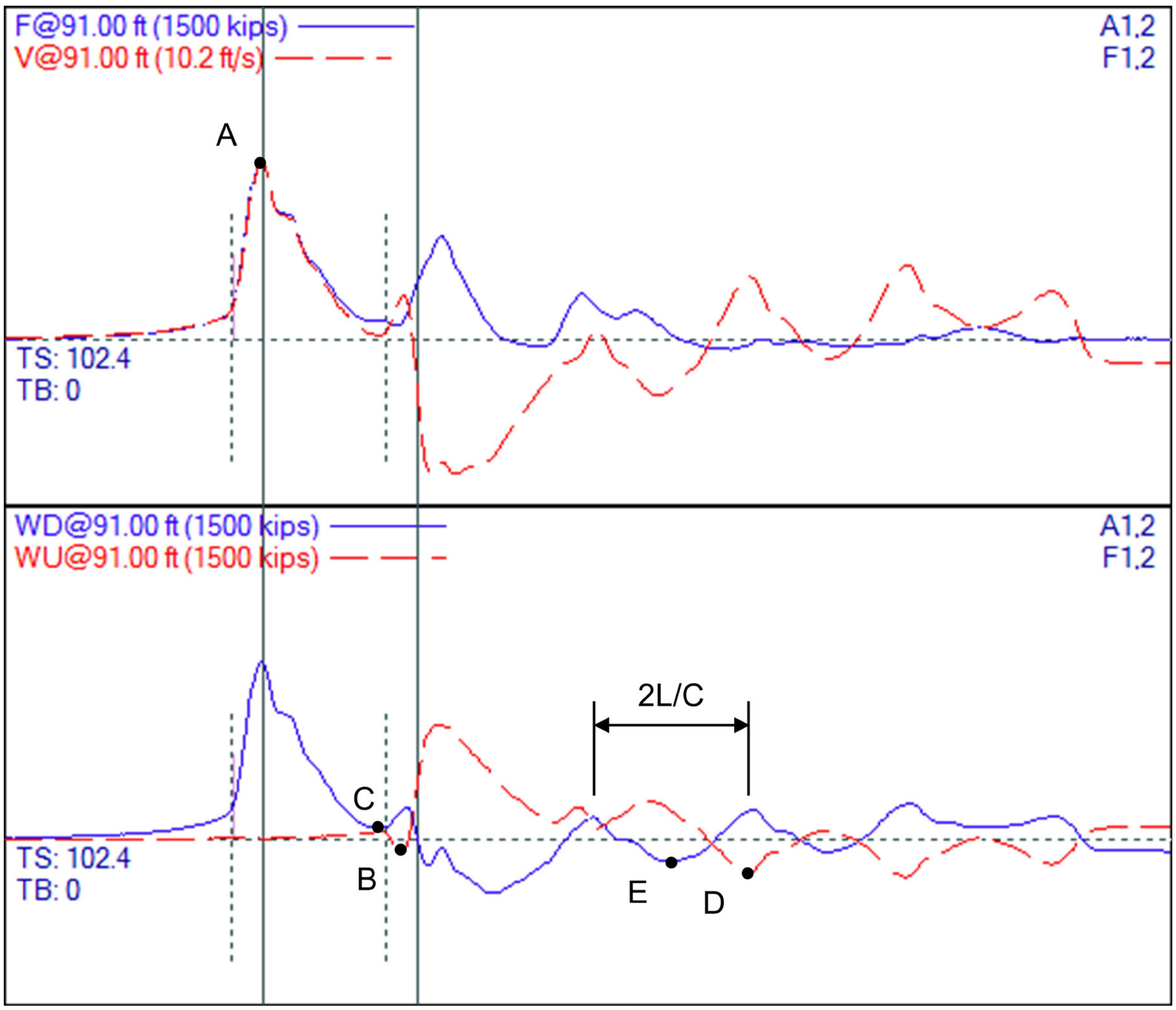


Figure 10-15 Example compression and tension stress calculation.

In Figure 10-15, the tension in the pile is computed from the superposition of the maximum upward tension force in wave up at time 2L/C +/- 20% identified by point B, and the minimum downward compression force in wave down between time 0 and time *2L/C* identified by point *C*. For the example presented, these values are -62 kips for point B and 45 kips for point *C*. The computed net tension force, CTN, is -17 kips. This corresponds to a computed tension stress maxima within the first

2L/C, TSN, of 0.05 ksi. This low tension stress in the upward traveling wave agrees with the hard driving conditions depicted by the force and velocity records and reported blow count.

The maximum tension stress can also occur later in the blow. Therefore, the full record length is searched for the minimum net tension force occurring from the upward travelling tension wave and the minimum value of the downward travelling wave in the previous 2L/C interval. In the example given, the minimum tension force occurs as a result of the tension in wave up of -154 kips identified by point D and the minimum force in the wave down in the preceding 2L/C interval of -105 kips identified by point E. The computed tension force of -259 kips corresponds to a computed tension stress, TSX, of 0.80 ksi. Hence, high tension stresses can occur in hard driving cases due to the downward traveling wave. This occurs when the reflected compression wave from a fixed toe condition reaches the free end at the pile head and reflects down the pile as a tension wave.

10.6.5 Pile Integrity

The basic concepts of wave mechanics were presented in Section 10.4. Convergence between the force and velocity records prior to the rise time before the toe response at time 2L/C indicates a reduction in pile impedance, *EA/C*. For uniform cross section piles, an impedance reduction is therefore pile damage.

The Beta Method, developed by Rausche and Goble (1979), is used to assess the relative severity of any damage along the pile shaft based on the convergence between the force and velocity records. If damage is detected, the relative severity of the damage is quantified and assigned a BTA value indicating the approximate reduction in pile impedance at the damage location. Rausche and Goble (1979) proposed the guidelines in Table 10-2 as in indication of the severity of pile damage. Piles with BTA values below 80% correspond to damaged or broken piles.

Table 10-2 Pile Damage Guidelines (after 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

A method to assess pile toe damage solely for concrete piles driven using the EDC system with an embedded strain gage and accelerometer at both the pile head and pile toe was proposed by Verbeek and Middendorp (2011). This method uses a change in the prestress strain level in the top or bottom strain gage to assess damage in that portion of the pile. A change in the prestress level by more than 50 microstrains over 10 consecutive blows was recommended as identifying pile damage. This method will detect damage occurring due to a change in the prestress level but not damage due to concrete tension cracks.

One limitation of the original Beta method was its ability to detect damage near the pile toe when high toe resistances and/or stress wave reflections occur. Likins and Rausche (2014) revisited the original Beta method based on improvements in signal processing since 1979. For concrete piles, the wave speed must be determined from an early blow. They proposed near toe damage can then be assessed by looking for a reflection occurring too early for the correct 2L/C time. The PDA performs this computation and automatically identifies an early toe reflection by a BTT value that changes from 100% to 1% thereby highlighting the early toe reflection for further study and evaluation by the test engineer. The BTA and BTT methods can be used for pile damage assessments on all pile types.

10.6.6 Signal Matching

Once dynamic test data is acquired, it is routinely analyzed with a signal matching software program at select events such as the end of initial driving and beginning of restrike, or at key pile penetration depths such as the estimated pile toe elevation. Signal matching software programs include CAPWAP, iCAP, TNOWAVE, and All-Wave DLT. The pile and soil models used in these programs vary. Hence, both the analyst and the end user should fully understand the models contained within a specific program as well as that programs performance and limitations under a given set of conditions.

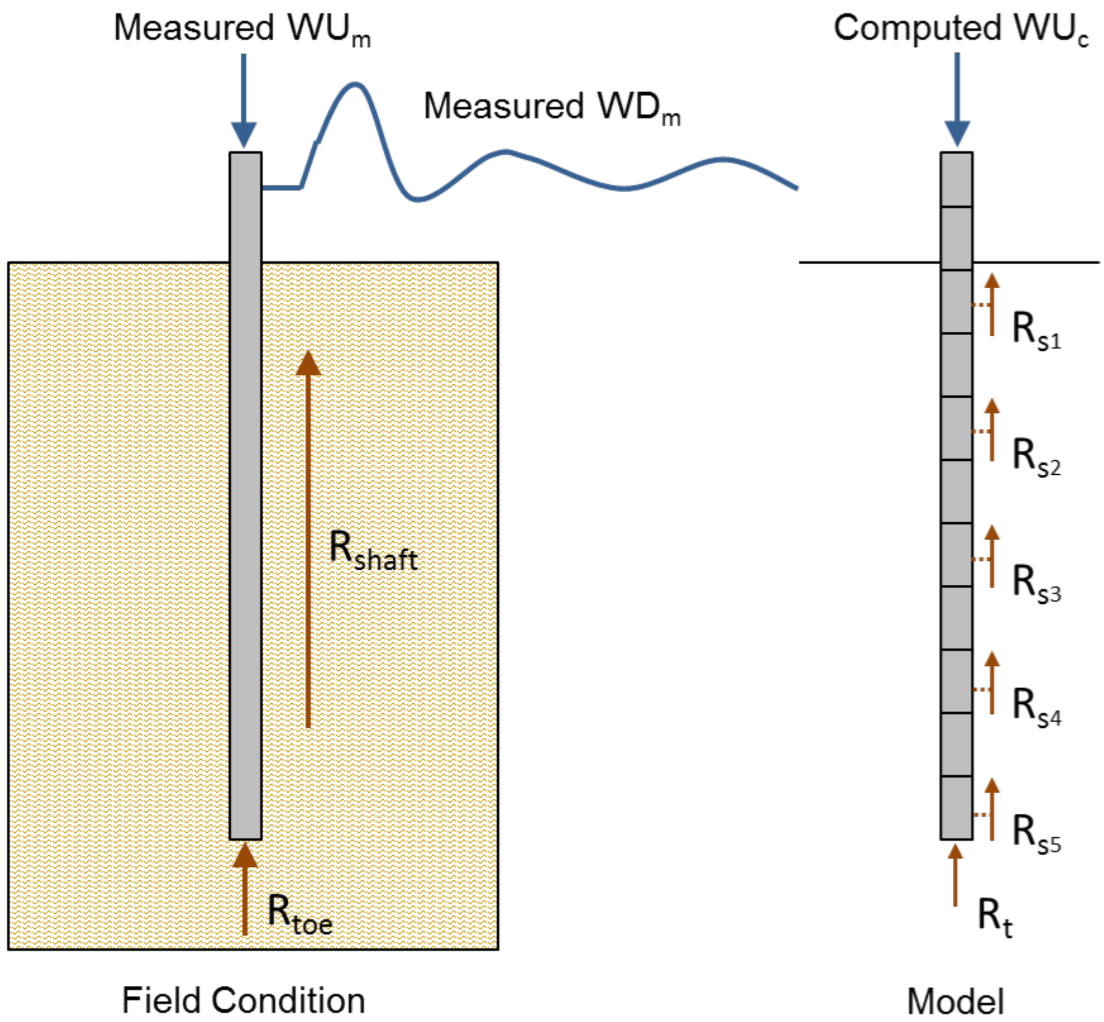
Signal matching programs allow a more rigorous evaluation of nominal geotechnical resistance, the relative resistance distribution, as well as insight into the soil quake and soil damping. Signal matching analyses are typically performed on an individual hammer blow selected from the end of driving or beginning of restrike. As such, a signal matching analysis refines the field dynamic test results at a particular penetration depth or time.

The AASHTO resistance factor for a dynamic test with signal matching was based on restrike dynamic test data analyzed with the CAPWAP program. Therefore, the discussion and examples presented in the remainder of this signal matching section will use the analysis procedures and output from that program for illustrative purposes.

The CAPWAP signal matching program uses wave equation type pile and soil models with the measured force and velocity records from the dynamic test replacing the hammer model. In this method, depicted in Figure 10-16, the pile is modeled by a series of continuous pile segments and static and dynamic soil resistances are modeled by elasto-plastic springs and dashpots, respectively. The dynamic test data is 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, soil quakes, and soil damping parameters are made. Then, the measured wave down is used to set the pile model in motion. The program then computes the equilibrium wave up, which can be compared to the measured wave up. Initially, the computed and measured wave up will not agree with each other. Adjustments are made to the soil model and the calculation process repeated.

The ability to match the measured and computed waves at various times is controlled by different factors. Figure 10-17 presents an initial trial in the signal matching process where the measured and computed records do not agree. This figure identifies the factors that most influence the signal match quality over 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 10-17.



1. Set up pile and soil model. Assume R_{shaft}, R_{toe}, and $R_{nominal}$.
2. Apply measured wave down, WD_m, to model at pile head and compute wave up, WU_c.
3. Compare computed wave up, WU_c, with measured wave up, WU_m.
4. Adjust R_{shaft}, R_{toe}, $R_{nominal}$, quake, and damping.
5. Return to step 2 and iterate until satisfactory match is obtained.

Figure 10-16 Schematic of CAPWAP signal matching method.

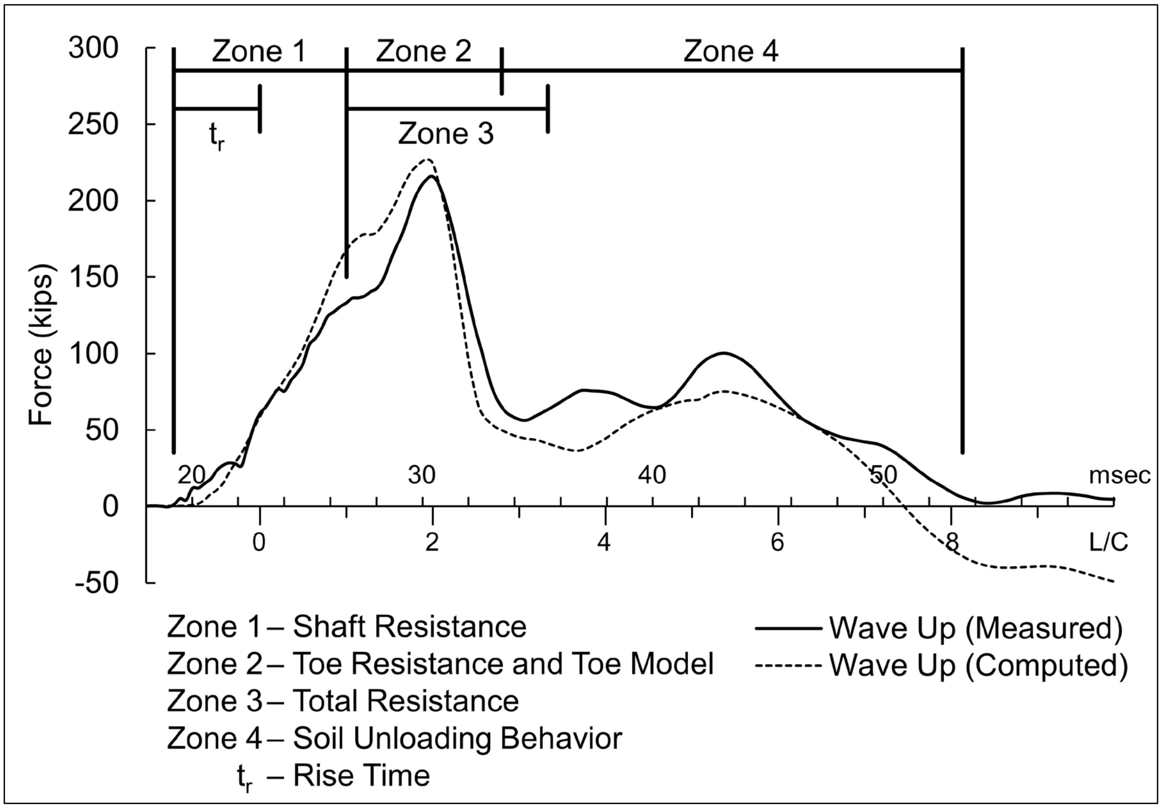


Figure 10-17 Factors most influencing signal matching analysis.

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 nominal resistance controls the match quality. A good wave match in Zone 3 is essential for accurate nominal resistance assessments. Zone 4 begins at the end of Zone 2 and continues for a duration of 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 iteration process is presented in Figure 10-18.

Throughout the iteration process, adjustments are made to the soil model until no further improvement can be obtained between the measured and computed wave up. The resulting soil model from signal matching is then considered the best estimate of the nominal resistance, including the soil resistance distribution, the soil quakes, and the soil damping characteristics.

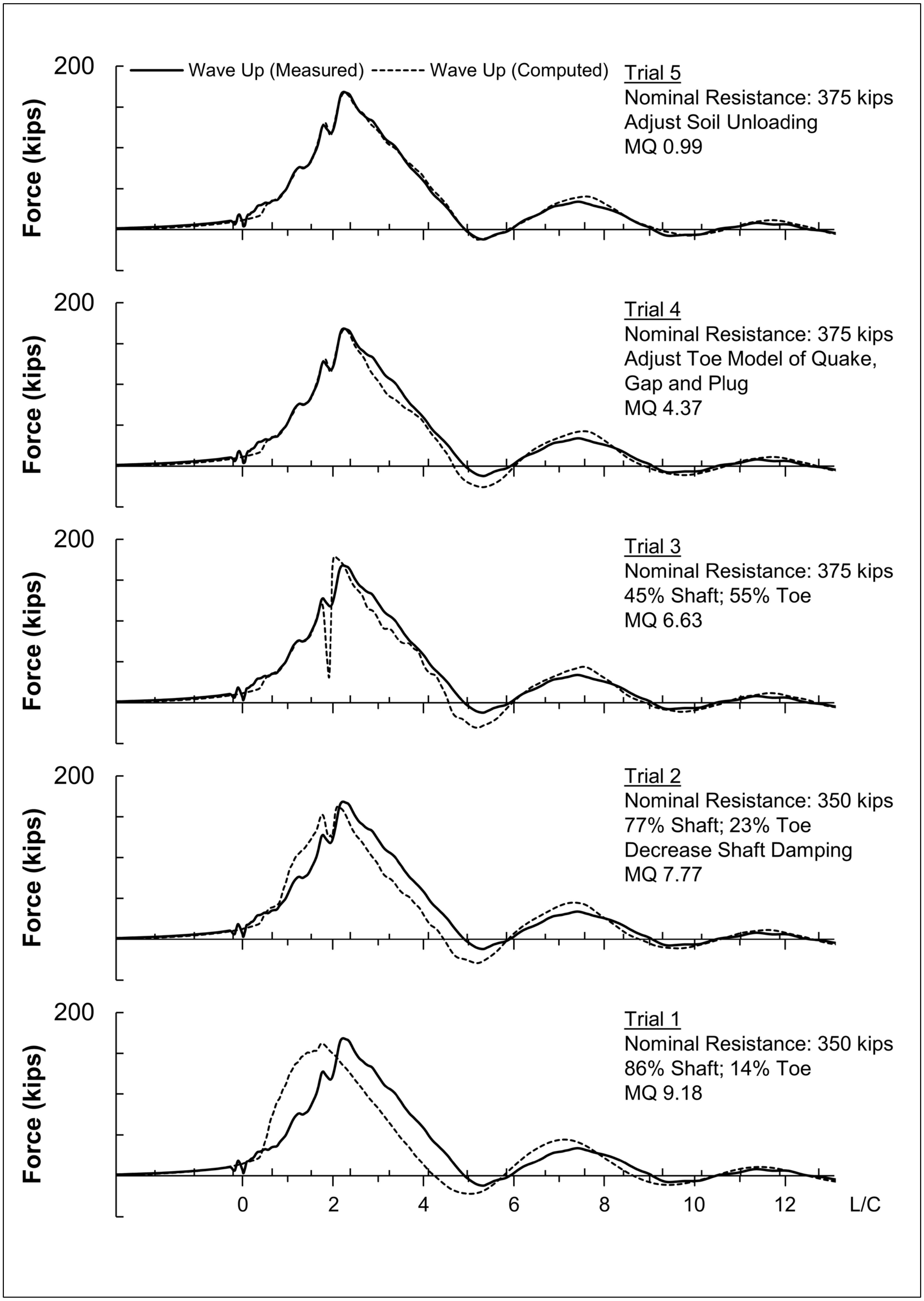


Figure 10-18 Example of signal matching iteration matching process.

The final graphical results from a signal matching analysis are presented in Figure 10-19. Four plots are included in the graphical output. While wave up matching was performed for the analysis, the “final match” plot in the upper left hand corners is typically presented in terms of the measured and computed force waves versus time. The force and velocity record versus time for the analyzed hammer blow are presented in the upper right hand plot. In the lower right hand corner, the shaft resistance magnitude on each soil segment in kips/ft is plotted above the load transfer diagram in kips. Important numerical results are presented immediately to the right of these plots including the analyzed pile properties, the final match quality number, maximum compression and tension stresses, as well as dynamic soil properties. The final plot, in the lower left hand corner, includes simulated static load test result. The pile model along with the soil resistance and quake values are used to develop these pile top and pile toe load versus displacement plots.

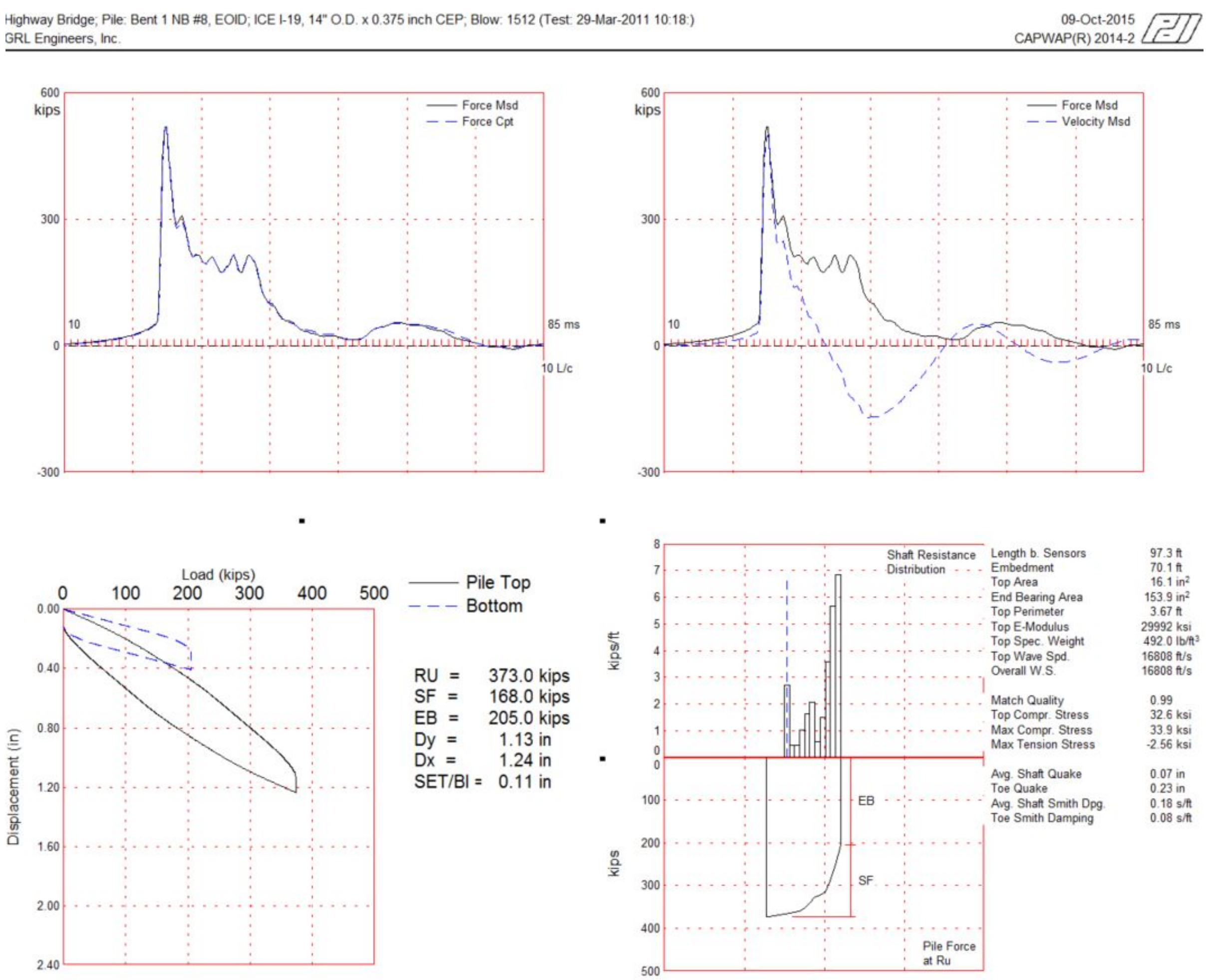


Figure 10-19 Example of signal matching graphical output.

An example of a signal matching final result summary is presented in Figure 10-20. For each soil segment, this table lists the depth below grade and the corresponding static soil resistance, Ru. Unit shaft resistance values are also provided in the far right hand column and can be compared to expected values from static analyses. The shaft and toe quake and damping values are summarized in the “Soil Model Parameters / Extensions” table beneath the soil resistance output. The match quality number, pile penetration resistance, compression and tension stresses, and transferred energy are summarized in the bottom section.

The “EXTREMA TABLE”, presented in Figure 10-21, summarizes the stress distribution throughout the pile. This table is important because it indicates if higher compression stresses are present elsewhere in the pile below the gage location at pile segment number 1. In the example provided, the maximum compression stresses is 33.9 ksi and it occurs at a distance of 30.2 feet below the gage location. Similarly, the maximum tension stress of 2.56 ksi occurs at a location 77.2 feet below the gage location.

Figure 10-22 presents the “CASE METHOD” summary table. This output table can be used to determine which Case Method nominal resistance equation and damping factor correlates best with the nominal resistance from the more rigorous signal matching analysis. Hence, this table helps determine which Case Method equation and what damping factor should be used for any similar piles that will not be analyzed by signal matching. For the results summarized in Figure 10-22, the RMX Case Method equation with a damping factor of 1.45 would likely be selected based on correlation with the nominal resistance from the signal matching analysis.

The final output table from signal matching analysis is the “PILE PROFILE AND PILE MODEL” table presented in Figure 10-23. For the uniform closed-end pipe pile in the analysis example, no significant changes in the pile cross section area, elastic modulus, unit weight, or perimeter were entered. For non-uniform pile types of differing materials, tapered piles, or open end piles with added impedance due to a soil plug, a more complex pile profile would be reported.

Highway Bridge; Pile: Bent 1 NB #8, EOID — Test: 29-Mar-2011 10:18
ICE I-19, 14" O.D. x 0.375 inch CEP; Blow: 1512 — CAPWAP(R) 2014-2
GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 373.0; along Shaft 168.0; at Toe 205.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf
				373.0			
1	30.2	3.0	8.0	365.0	8.0	2.70	0.74
2	36.9	9.7	3.0	362.0	11.0	0.45	0.12
3	43.6	16.4	3.0	359.0	14.0	0.45	0.12
4	50.3	23.1	7.0	352.0	21.0	1.04	0.28
5	57.0	29.8	11.0	341.0	32.0	1.64	0.45
6	63.7	36.5	14.0	327.0	46.0	2.09	0.57
7	70.5	43.2	4.0	323.0	50.0	0.60	0.16
8	77.2	49.9	10.0	313.0	60.0	1.49	0.41
9	83.9	56.6	24.0	289.0	84.0	3.58	0.98
10	90.6	63.4	38.0	251.0	122.0	5.66	1.55
11	97.3	70.1	46.0	205.0	168.0	6.86	1.87
Avg. Shaft			15.3			2.40	0.65
Toe			205.0				191.77

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.18	0.08
Quake	(in)	0.07	0.23
Case Damping Factor		1.06	0.57
Damping Type		Viscous	Viscous
Unloading Quake	(% of loading quake)	52	49
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	9	
Resistance Gap (included in Toe Quake) (in)			0.07
Soil Plug Weight	(kips)		0.027

CAPWAP match quality = 0.99 (Wave Up Match) ; RSA = 0
Observed: Final Set = 0.11 in; Blow Count = 108 b/ft
Computed: Final Set = 0.10 in; Blow Count = 118 b/ft

max. Top Comp. Stress = 32.6 ksi (T= 26.3 ms, max= 1.042 x Top)
max. Comp. Stress = 33.9 ksi (Z= 30.2 ft, T= 27.9 ms)
max. Tens. Stress = -2.56 ksi (Z= 77.2 ft, T= 51.7 ms)
max. Energy (EMX) = 22.4 kip-ft; max. Measured Top Displ. (DMX)= 0.86 in

Page 2 — Analysis: 05-Oct-2015

Figure 10-20 Example signal matching output “Summary Results” table.

Highway Bridge; Pile: Bent 1 NB #8, EOID Test: 29-Mar-2011 10:18
ICE I-19, 14" O.D. x 0.375 inch CEP; Blow: 1512 CAPWAP(R) 2014-2
GRL Engineers, Inc.

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	523.2	-11.2	32.6	-0.70	22.4	17.4	0.87
2	6.7	524.0	-11.8	32.6	-0.74	22.2	17.4	0.85
4	13.4	525.8	-13.0	32.7	-0.81	21.9	17.3	0.82
6	20.1	528.7	-14.3	32.9	-0.89	21.6	17.2	0.78
8	26.8	541.2	-15.3	33.7	-0.95	21.1	16.8	0.74
10	33.6	516.4	-18.4	32.2	-1.15	19.5	16.5	0.70
12	40.3	511.7	-25.2	31.9	-1.57	18.6	16.3	0.66
14	47.0	512.8	-31.6	31.9	-1.97	17.7	15.8	0.61
16	53.7	504.5	-35.9	31.4	-2.24	16.4	15.2	0.57
17	57.0	514.7	-39.0	32.1	-2.43	16.1	14.8	0.54
18	60.4	485.5	-38.7	30.2	-2.41	14.6	14.4	0.52
19	63.7	490.2	-40.6	30.5	-2.53	14.3	14.2	0.49
20	67.1	445.4	-38.9	27.7	-2.42	12.6	14.1	0.47
21	70.5	452.4	-40.2	28.2	-2.51	12.3	13.8	0.45
22	73.8	448.0	-40.2	27.9	-2.50	11.6	13.5	0.42
23	77.2	461.9	-41.1	28.8	-2.56	11.3	13.0	0.40
24	80.5	446.7	-38.9	27.8	-2.43	10.2	12.4	0.37
25	83.9	466.9	-39.0	29.1	-2.43	9.9	11.7	0.34
26	87.2	415.9	-33.5	25.9	-2.09	8.2	10.9	0.32
27	90.6	422.7	-33.6	26.3	-2.09	7.9	10.7	0.30
28	93.9	309.3	-24.8	19.3	-1.55	5.8	10.9	0.28
29	97.3	334.4	-24.7	20.8	-1.54	4.0	10.4	0.25
Absolute	30.2			33.9			(T =	27.9 ms)
	77.2				-2.56		(T =	51.7 ms)

Figure 10-21 Example signal matching output “Extrema” table.

Highway Bridge; Pile: Bent 1 NB #8, EOID Test: 29-Mar-2011 10:18
ICE I-19, 14" O.D. x 0.375 inch CEP; Blow: 1512 CAPWAP(R) 2014-2
GRL Engineers, Inc.

CASE METHOD

J =	0.0	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8
RP	630.3	593.0	555.7	518.4	481.1	443.8	406.4	369.1	331.8	294.5
RX	630.3	593.0	555.7	518.4	481.1	443.8	406.4	376.7	362.8	351.3
RU	660.0	587.6	515.1	442.7	370.3	297.8	225.4	152.9	80.5	8.1

RAU = 124.9 (kips); RA2 = 340.0 (kips)

Current CAPWAP Ru = 373.0 (kips); Corresponding J(RP)= 1.38; J(RX) = 1.45

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips	KEB kips/in
17.5	26.15	501.8	520.4	520.6	0.86	0.11	0.11	22.4	552.7	1296

Figure 10-22 Example signal matching output “Case Method” table.

Highway Bridge; Pile: Bent 1 NB #8, EOID — Test: 29-Mar-2011 10:18
ICE I-19, 14" O.D. x 0.375 inch CEP; Blow: 1512 — CAPWAP(R) 2014-2
GRL Engineers, Inc.

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.0	16.1	29992.4	492.000	3.67
97.3	16.1	29992.4	492.000	3.67

Toe Area 153.9 in^2

Top Segment Length 3.36 ft, Top Impedance 29 kips/ft/s

Wave Speed: Pile Top 16807.9, Elastic 16807.9, Overall 16807.9 ft/s
Pile Damping 1.00 %, Time Incr 0.200 ms, 2L/c 11.6 ms
Total volume: 10.846 ft^3; Volume ratio considering added impedance: 1.000

Figure 10-23 Example signal matching output “Pile Profile and Pile Model” table.

10.7 CONSIDERATIONS IN TEST SPECIFICATION AND IMPLEMENTATION

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, dynamic tests on a specified percentage of the production pile quantity per site condition may be performed. Production pile testing is usually performed for quality assurance checks on nominal resistance, hammer performance, driving stress compliance, pile integrity, and for site variability considerations. Lastly, dynamic testing can be used where it was not specified to troubleshoot problems that arise during construction such as differing penetration depths or unusual driving records.

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. For example, it may be desirable to test a higher percentage of piles 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 nominal resistance.

On smaller projects, AASHTO specifications suggest a minimum of two dynamic tests per site condition, but no less than 2% of the production piles. On larger projects and small projects with anticipated installation difficulties or significant time dependent nominal resistance 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, the design or construction engineer should be able to adjust the number and locations of dynamically tested piles based on design or construction issues that arise. For example, a change to 100% dynamic testing and the resulting higher resistance factor may benefit a project having pile length overruns if the cost of testing and associated construction time is less than the cost of the pile length overrun.

Restrike dynamic tests should be performed whenever pile nominal resistance is being evaluated by dynamic test methods. Restrikes are often specified 24 hours after initial driving. However, in fine grained soils, longer time periods are generally required for the full time dependent nominal resistance 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. 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 for any restrike test.

In soils that exhibit a large increase in resistance from soil setup, it may not be possible to activate all the resistance during restrike with the pile hammer used for the pile installation. In this situation, a drop hammer system such as the one shown in Figure 10-24 can be used to determine the nominal resistances during restrike. Typically, a ram weight of approximately 2% of the desired nominal geotechnical resistance is required for resistance mobilization in a high strain dynamic test.

When dynamic testing is performed by a consultant, the requirements for signal matching analysis should be clearly addressed in the dynamic testing specification including the signal matching method. The AASHTO resistance factor of 0.65 is based on signal matching on restrike dynamic test data with the CAPWAP analysis method. A modified or locally calibrated resistance factor may be appropriate for other signal matching methods. This includes the simplified automatic signal matching program, iCAP, that shares some but not all of the soil and pile modeling and analysis capabilities of the CAPWAP program.

On design-build projects, it is common for the dynamic testing to be performed under the design-build team with independent verification tests or test reviews performed by the owner. On conventional design-bid-build projects, it is also sometimes contractually convenient to specify that the general contractor retain the services of

Figure 10-24 APPLE drop weight system.

the dynamic testing firm. However, this can create potential problems since the contractor is then responsible for the owner's quality assurance program. Some agencies have contracted directly with a dynamic testing firm to avoid this potential conflict. Other large public owners have acquired dynamic test equipment and perform these tests with their in-house staff.

Knowledgeable dynamic testing personnel who properly acquire and interpret dynamic test records are important for correct implementation of dynamic test results on a project. Therefore, specifications should require that dynamic testing personnel attain an appropriate level of expertise on the Pile Driving Contractors Association (PDCA) sponsored "Dynamic Measurement and Analysis Proficiency Test" for providers of high strain dynamic testing services. The test was designed to reflect the knowledge and ability of dynamic test providers which is then indicated in a "Certificate of Proficiency." The exam results categorize the six levels of proficiency as provisional, basic, intermediate, advanced, master, or expert. This allows agencies to specify the level of expertise required for their standard practice or for a given project. Additional details on the exam can be found on the PDCA website, www.piledrivers.org.

10.8 PRESENTATION AND INTERPRETATION OF DYNAMIC TEST 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 7, numerous factors are considered in a pile foundation design. Therefore, the foundation designer should review the test results since many considerations; (downdrag, scour, uplift, lateral loads, settlement, etc.) may affect the overall design and construction requirements.

10.8.1 Field Results

Construction personnel are often presented with dynamic testing results with minimal guidance on how to use or interpret the information. Therefore, it may be helpful to both construction personnel and foundation designers to familiarize themselves with a typical screen display and the information available in the field during a dynamic test. Figure 10-25 presents a typical dynamic test screen display from the Pile Driving Analyzer 8G system.

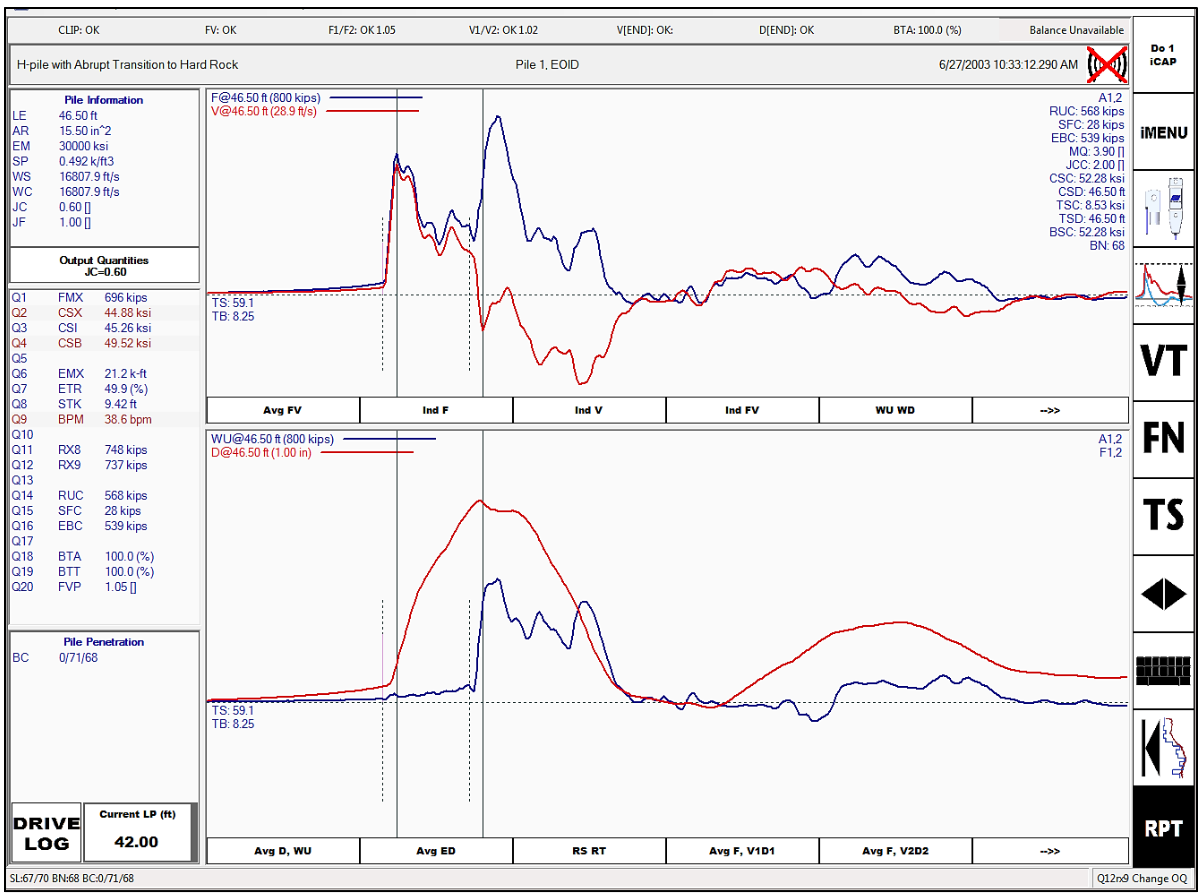


Figure 10-25 Typical dynamic test display from Pile Driving Analyzer.

The main pile information input quantities are displayed near 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 graphic portion of the screen is typically divided into two displays as illustrated above in Figure 10-25. The upper graphic display presents the force and velocity records versus time plotted proportionally. The lower display presents the upward travelling wave, WU, and the displacement at the gage location versus time. Both displays will change for each hammer blow. The first full height solid vertical line represents time t_1 in the Case Method calculations and corresponds to the time of impact (i.e., the first input peak, as the waves pass the gage location near the pile head). The second full height solid 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 at time 2L/C. The partial height, dotted, vertical lines preceding the full height lines note the start of the impact event or start of the toe reflection, respectively.

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 total soil resistance above that depth. Toe resistance is indicated by the separation between these records beginning at the rise time marker prior to time t_2.

A search for convergence between the force and velocity records is performed 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. As discussed in Section 10.6.5, an early reflection prior to 2L/C is also evaluated and, if noted, a BTT value of 1% will highlight the need for record review for toe damage by the engineer.

The results of Case Method numerical computations are identified by three letter codes displayed on the left hand side of the screen. A summary of the most commonly computed quantities and their corresponding three letter code is presented in Table 10-3.

Table 10-3 Description of Typical Dynamic Test Output Codes

PDA Output Code	Output Quantity Description
CSX	Maximum compression stress at the gage location.
CSI	Maximum compression stress from an individual strain transducer.
CSB	Maximum computed compression stress at pile toe.
TSX	Maximum computed tension stress.
BTA	Pile integrity factor.
BTT	Integrity indicator for early toe reflection.
LTD	Length to pile damage.
EMX	Maximum energy transferred to the gage location.
ETR	Energy transfer ratio (EMX / E rated).
STK	Computed hammer stroke.
BPM	Hammer operating rate.
RMX	Maximum Case Method (requires damping factor, J).
RSP	Standard Case Method (requires damping factor, J).
RSU	Case Method with unloading correction (requires damping factor, J).
RAU	Automatic Case Method - toe bearing. No shaft resistance.
RA2	Automatic Case Method - Moderate shaft resistance.
RUC*	iCAP nominal resistance.
SFC*	iCAP shaft resistance.
EBC*	iCAP toe resistance.
CSC*	iCAP computed maximum compression stress.
CBC*	iCAP computed compression stress at pile toe.
TSC*	iCAP computed tension stress.

* - requires additional iCAP automated signal matching software.

In the example given in Figure 10-25, the first four output quantities Q1, Q2, Q3, and Q4 provide information on the maximum force and associated driving stresses. The maximum average force at the pile head, FMX, is 696 kips. This corresponds to an average compression stress at the gage location, CSX, of 44.9 ksi. The maximum compression stress from an individual strain transducer, CSI, is 45.3 ksi, and the maximum computed compression stress at the pile toe, CSB, is 49.5 ksi. Hence, driving stress levels are quite high and are above AASHTO recommended limits.

As noted earlier, the compression stress levels exceed the driving stress limits as well as the guaranteed minimum yield strength for H-piles made from A572, Grade 50 steel. Hence, the potential for pile toe damage is high. The pile integrity, BTA, is calculated as 100%, indicating that no detected damage. The integrity near the pile toe is also assessed and a BTT of 100% is calculated despite the very high driving stresses resulting from the abrupt refusal driving on hard rock.

An assessment of the performance of the single acting diesel hammer is made from output quantities Q5 through Q9. The average energy transferred to the gage location, EMX, is 21.2 ft-kips. As indicated by Q6, this corresponds to an energy transfer ratio, ETR, of 49.9% (EMX / manufacturer's rated hammer energy of 42.4 ft-kips). The hammer stroke, STK is 9.42 feet and the hammer operating rate, BPM, is 38.6 blows per minute.

Output quantities Q11 and Q12 display the nominal resistance as evaluated by the maximum Case Method, RMX, with a damping factor of 0.80 and 0.90. These nominal resistance computations are 748 and 737 kips, respectively and are reported as RX8 and RX9.

Output quantities Q14 to Q16 present the results of automated signal matching analysis using the iCAP method. The iCAP results indicate a nominal resistance, RUC, of 568 kips with 28 kips of shaft resistance, SFC, and 539 kips of toe resistance, EBC. Complete automated signal matching results are presented in the upper right corner of the graphical screen and include the match quality, MQ, of 3.90 along with the maximum computed compression stress, CSC, of 52.3 ksi which occurs at the pile toe. It should be emphasized that iCAP automated signal matching is only applicable to uniform piles. It cannot accurately analyze non-uniform piles, piles with splice gaps, damaged piles, concrete piles with minor cracking, or piles with uncertain properties. iCAP should also not be used for large diameter open-end pipe piles (due to internal plug movements) or on piles in unusual soil conditions.

Construction personnel should review the dynamic test results and 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. Nominal resistance estimates should be compared with the required nominal resistance. In soils with time dependent soil strength changes, this comparison of nominal resistance should be based on restrike tests and not end of initial driving results.

10.8.2 Evaluation of Hammer and Drive System Performance

The performance of a hammer and driving system can be evaluated from a driving system's energy transfer ratio, which is defined as the energy transferred to the pile head divided by the manufacturer's rated hammer energy. The energy transfer ratio should not be misinterpreted as the hammer efficiency as the energy transfer ratio includes all energy losses in the driving system. Numerous factors affect the transferred energy and hence the energy transfer ratio. These include the hammer stroke, fuel setting, helmet weight, hammer and pile cushions, pile impedance, pile length, soil resistance, dynamic soil properties, as well as the hammer efficiency.

Figure 10-26 presents energy transfer ratios 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 50th percentile. Depending upon the hammer-pile combination, average transferred energies as a percentage of the rated energy range from about 26% for a diesel hammer on a concrete pile to 69% for a hydraulic hammer on a steel pile.

Histograms of the energy transfer ratios for all diesel, single acting (SA) air/steam, and single acting (SA) hydraulic hammer on steel or concrete and timber piles are presented in Figures 10-27, 10-28, and 10-29, respectively. 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.

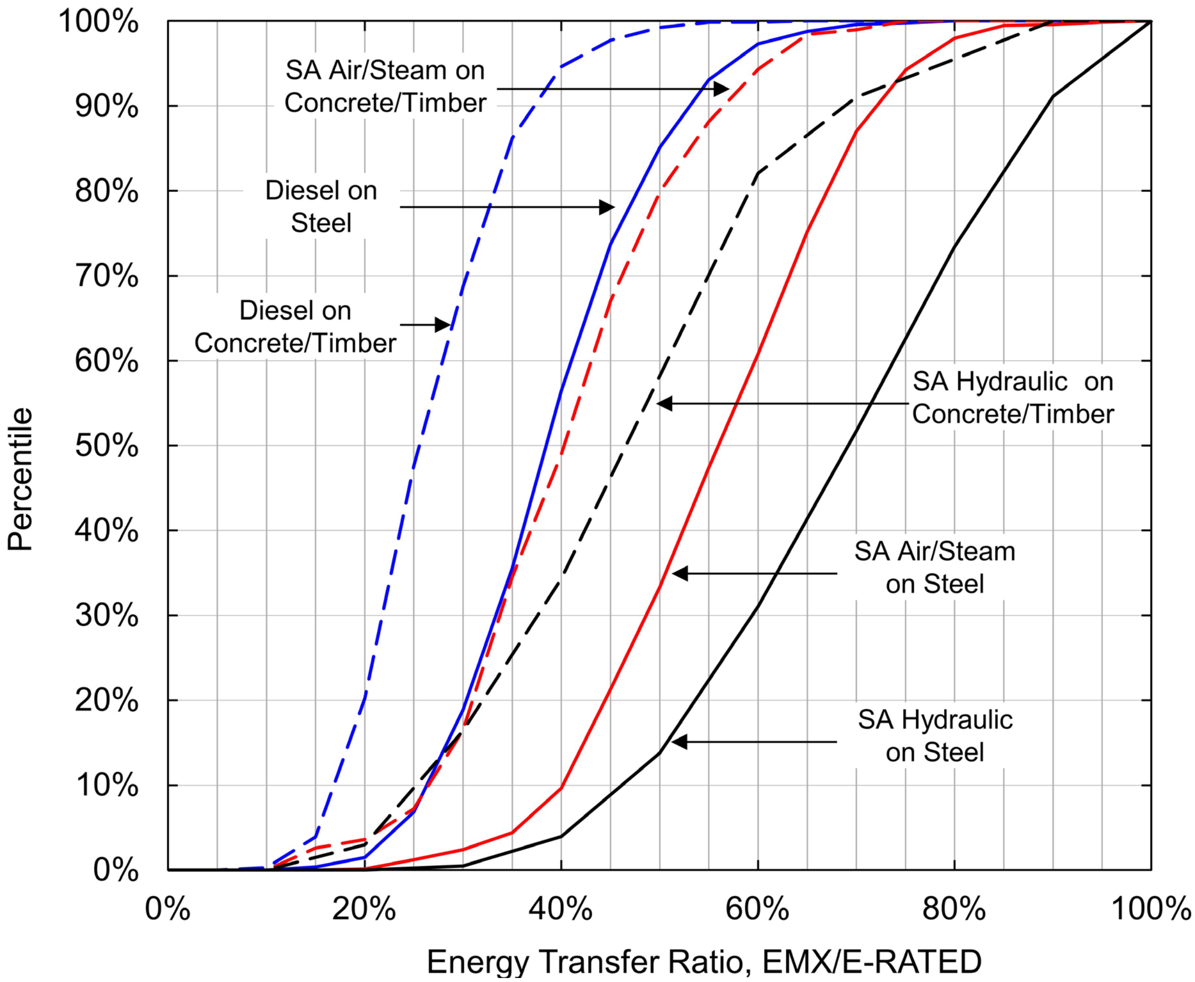


Figure 10-26 Energy transfer ratios for select hammer and pile combinations.

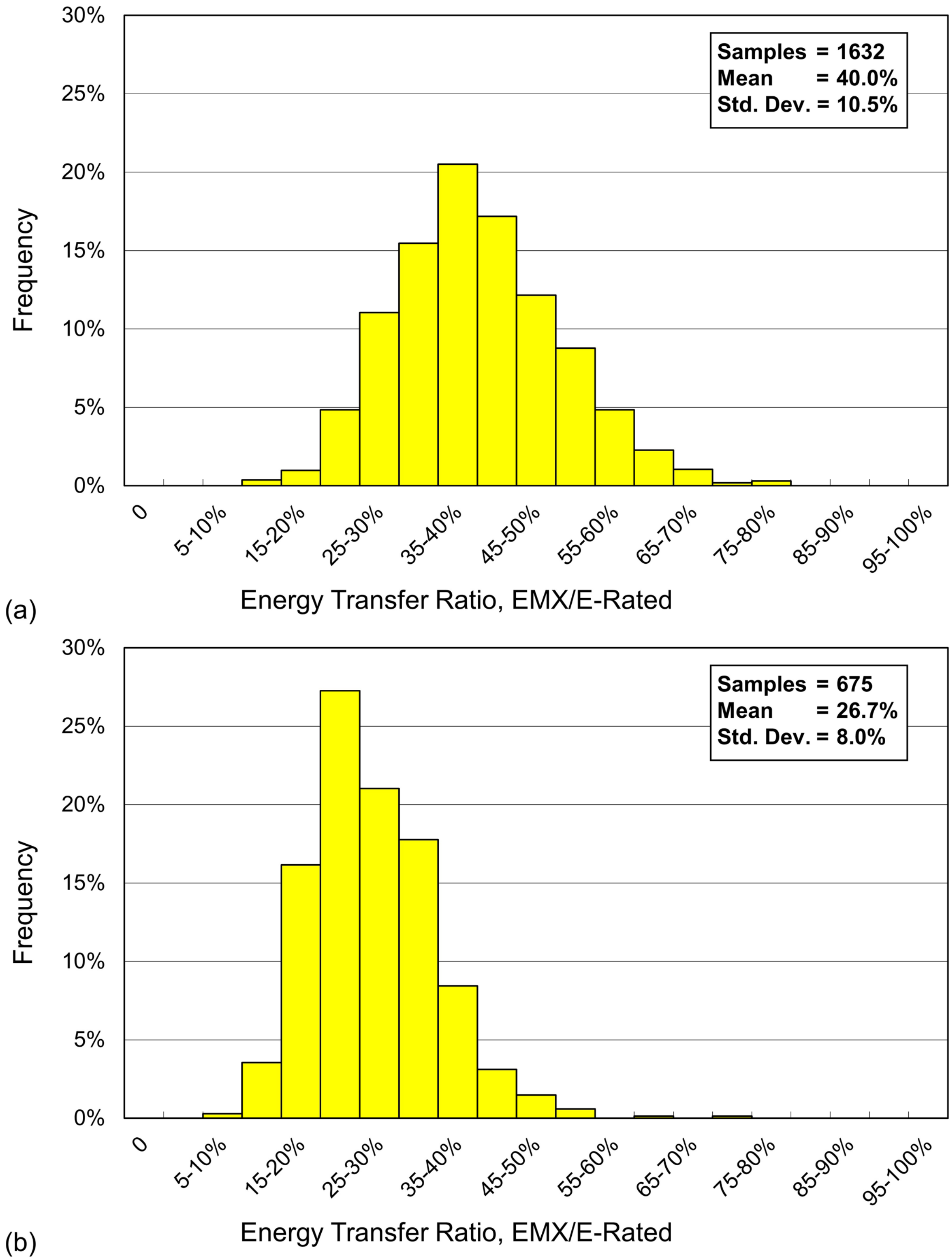
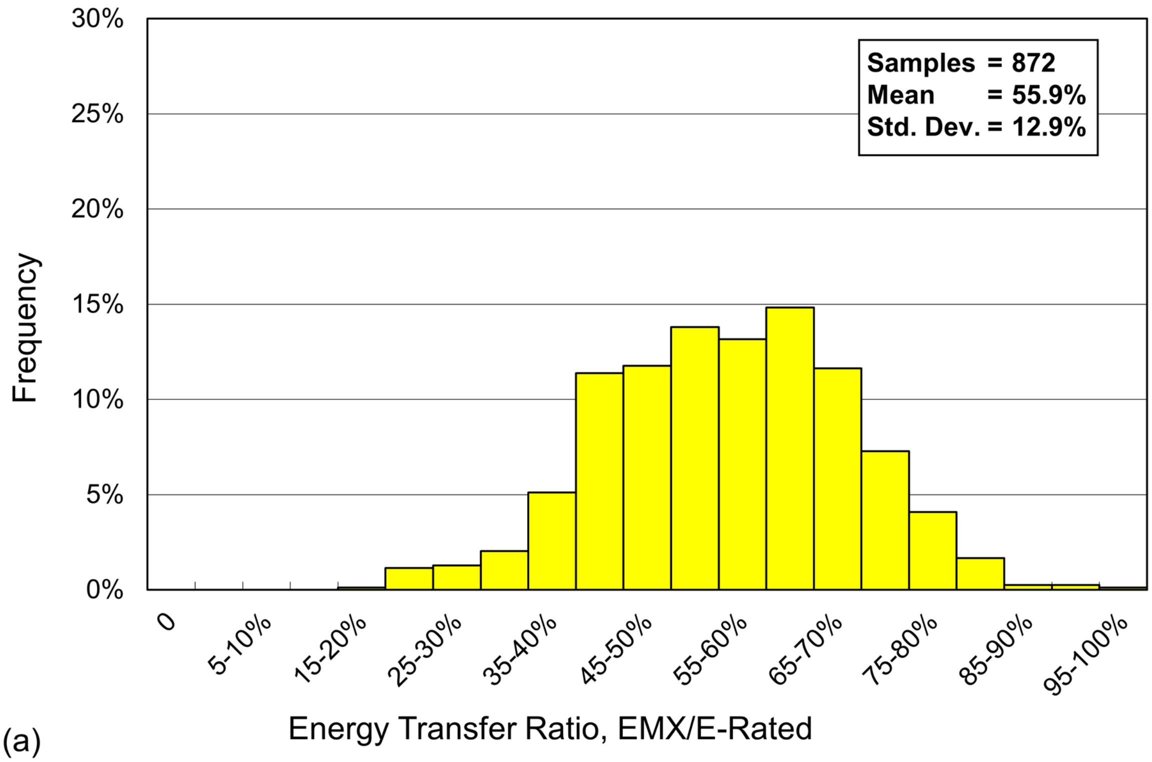
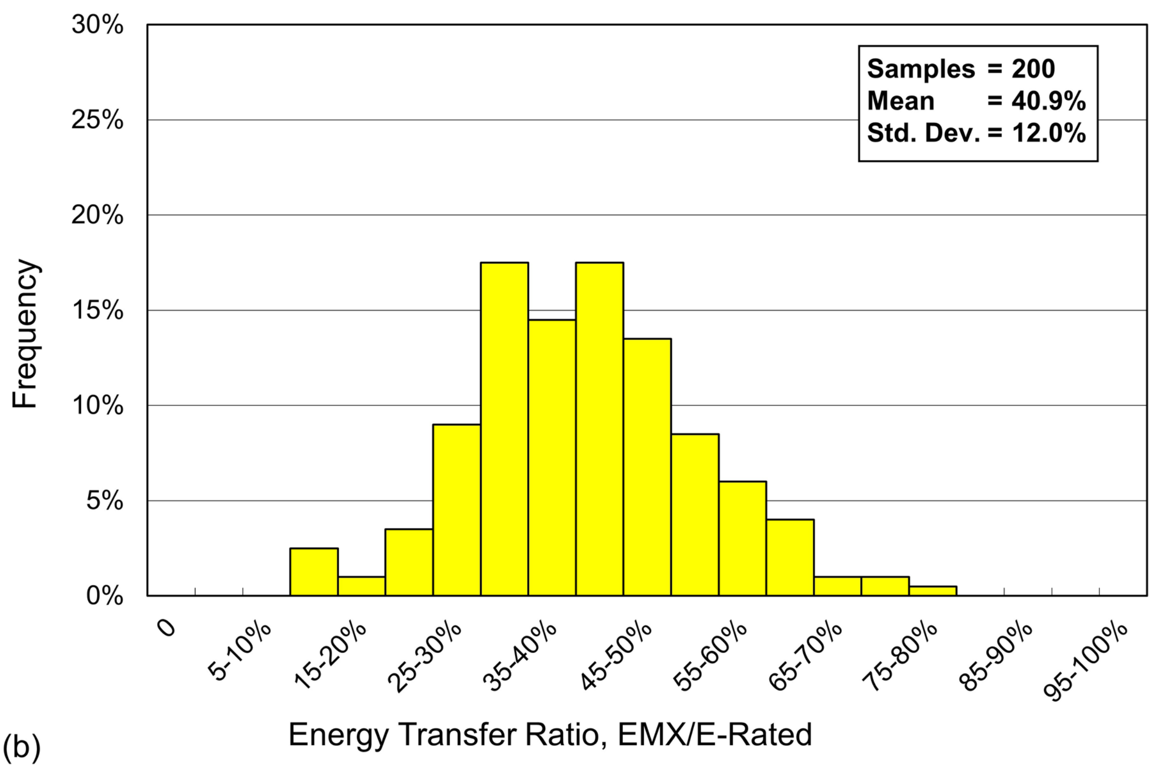


Figure 10-27 Histograms of energy transfer ratio for diesel hammers on (a) steel piles and (b) concrete/timber piles.



(a)



(b)

Figure 10-28 Histograms of energy transfer ratio for single acting air/steam hammers on (a) steel piles and (b) concrete/timber piles.

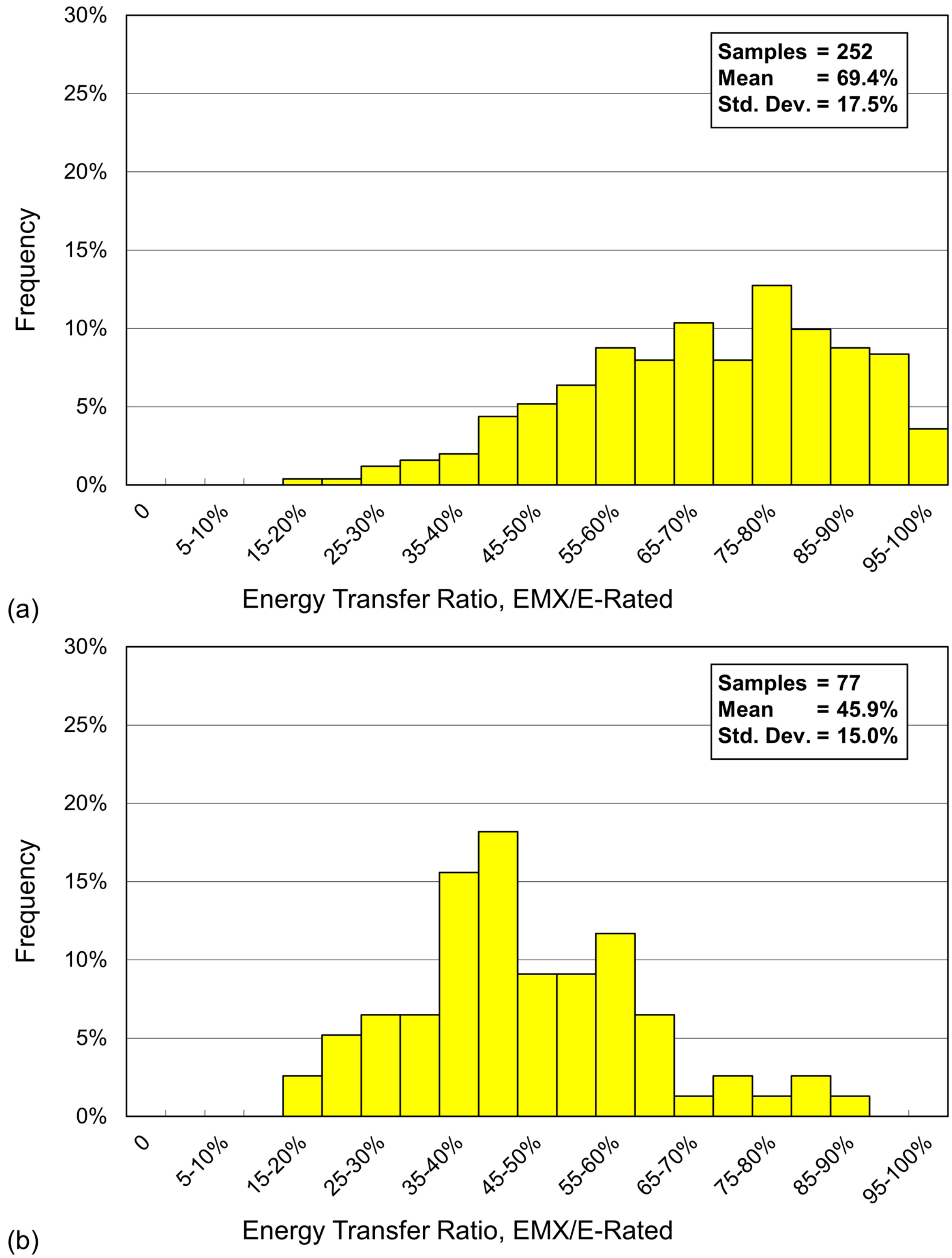


Figure 10-29 Histograms of energy transfer ratio for single acting hydraulic hammers on (a) steel piles and (b) concrete/timber piles.

10.8.3 Test Record Illustrating Problematic Hammer Performance

Records for a single acting diesel hammer exhibiting diesel fuel pre-ignition are presented in Figure 10-30. On this project, the diesel hammer had been operated relatively continuously all day with minimal down time. Note the magnitude of the force record during the pre-compression phase prior to impact, and compare that to a more typical diesel hammer record in Figure 10-31. In the presented pre-ignition data for Blow 986, the hammer stroke is 7.49 feet, the impact force is 383 kips, the transferred energy is 20.6 ft-kips, and the energy transfer ratio is 29.4% which is 10% less than the mean value for a diesel hammer on a steel pile. In Figure 10-26, an energy transfer ratio of 29.4% for a single acting diesel hammer on a steel pile falls near the 18th percentile which is also clearly indicative of a problem. For comparison, Blow 2 of the same drive sequence had a similar hammer stroke of 7.53 feet when the diesel fuel was not pre-igniting. However, in Blow 2, the hammer had an impact force of 481 kips, a transferred energy of 34.2 ft-kips, and an energy transfer ratio of 48.8%.

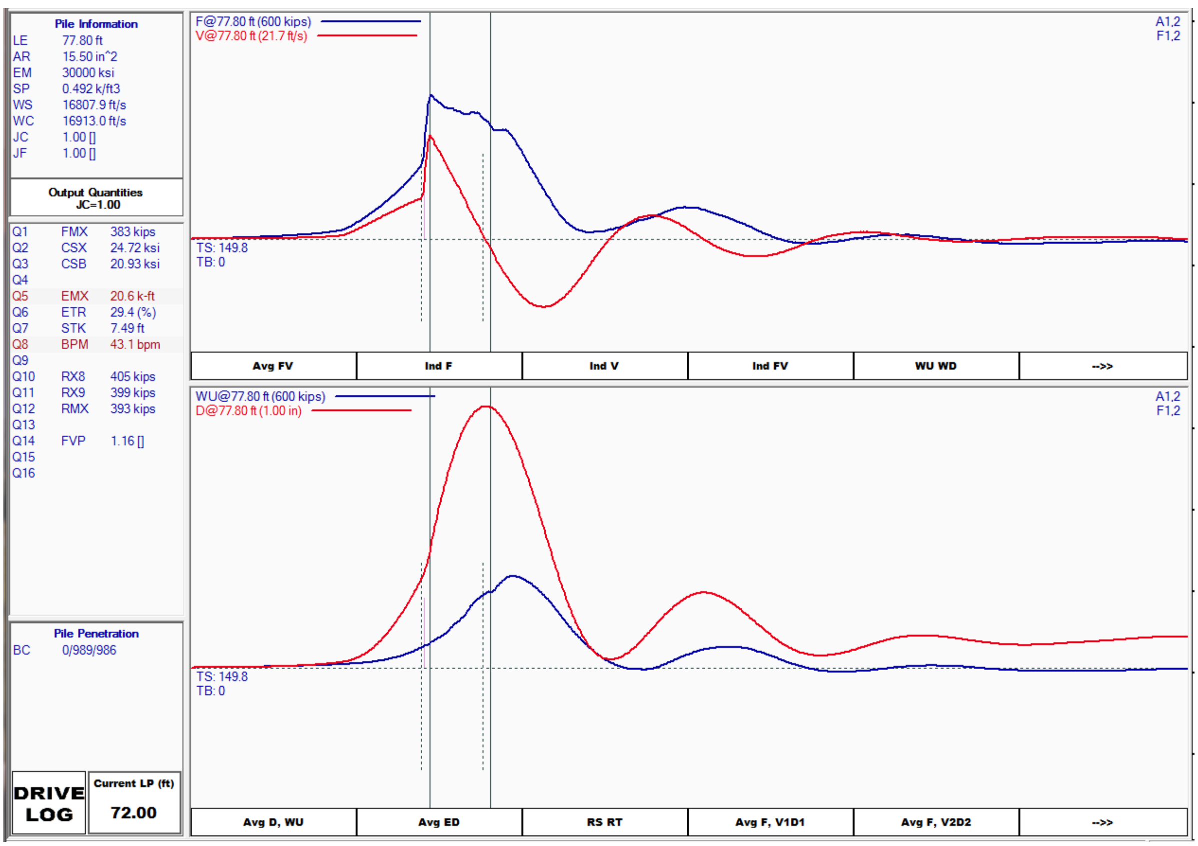


Figure 10-30 Example dynamic test records on pre-igniting diesel hammer.

Note the magnitude of the force record in Figure 10-30 during the pre-compression phase prior to impact. The pre-compression force on the pile is almost 50% of the impact force. This reduces the effectiveness of the hammer blow on the pile because it takes more energy to compress the combustion gases in the combustion chamber prior to the ram impacting the impact block. The reduced transferred energy to the pile results in a greater blow count occurring at a shallower pile penetration depth. Hence, if pre-ignition was not detected, pile driving would be terminated prematurely at a nominal resistance less than the required resistance.

10.8.4 Test Record Illustrating Pile Damage

Force and velocity records are presented in the upper graph of Figure 10-31 for a HP 14x117 H-pile driven with a single acting diesel hammer. The lower graph presents the wave up and displacement records. The pile information section of the screen in the left hand corner indicates the H-pile has a length of 88.3 feet below the gages. A visual interpretation of the force and velocity record suggests the pile has developed moderate shaft resistance over the lower portion of the pile with a significant amount of the nominal resistance due to toe resistance. Note that a full height, dash and dotted vertical line has also appeared between the two solid 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 dash and dotted line, indicates a pile impedance reduction or damage.

The BTA warning box near the top of the screen has also turned black and indicates a calculated BTA value of 82%. For the example shown, damage was occurring at a depth of 83.2 feet below gages due to the H-pile buckling and bending at this location. The integrity near the pile toe has been assessed and a BTT of 1% is reported highlighting the occurrence of near toe damage. The pile was extracted to confirm the indicated damage 5 feet above the pile toe. Photographs of the extracted pile are presented in Figure 10-32.

10.8.5 Test Records Illustrating Soil Setup

Test records for a 14 inch O.D. x 0.50 inch wall closed end pipe pile driven with a single acting diesel hammer are presented in Figures 10-33 and 10-34. Dynamic testing on this test pile was performed during initial driving and again during restrike 3 days after initial driving. The required nominal resistance was 765 kips based on resistance verification using dynamic testing with signal matching. At this test pile location, a minimum penetration depth of 90 feet was also specified to satisfy lateral loading requirements. Soil conditions consist of predominantly stiff to very stiff silty clays with interbedded medium dense to dense silty sand and sandy silt layers.

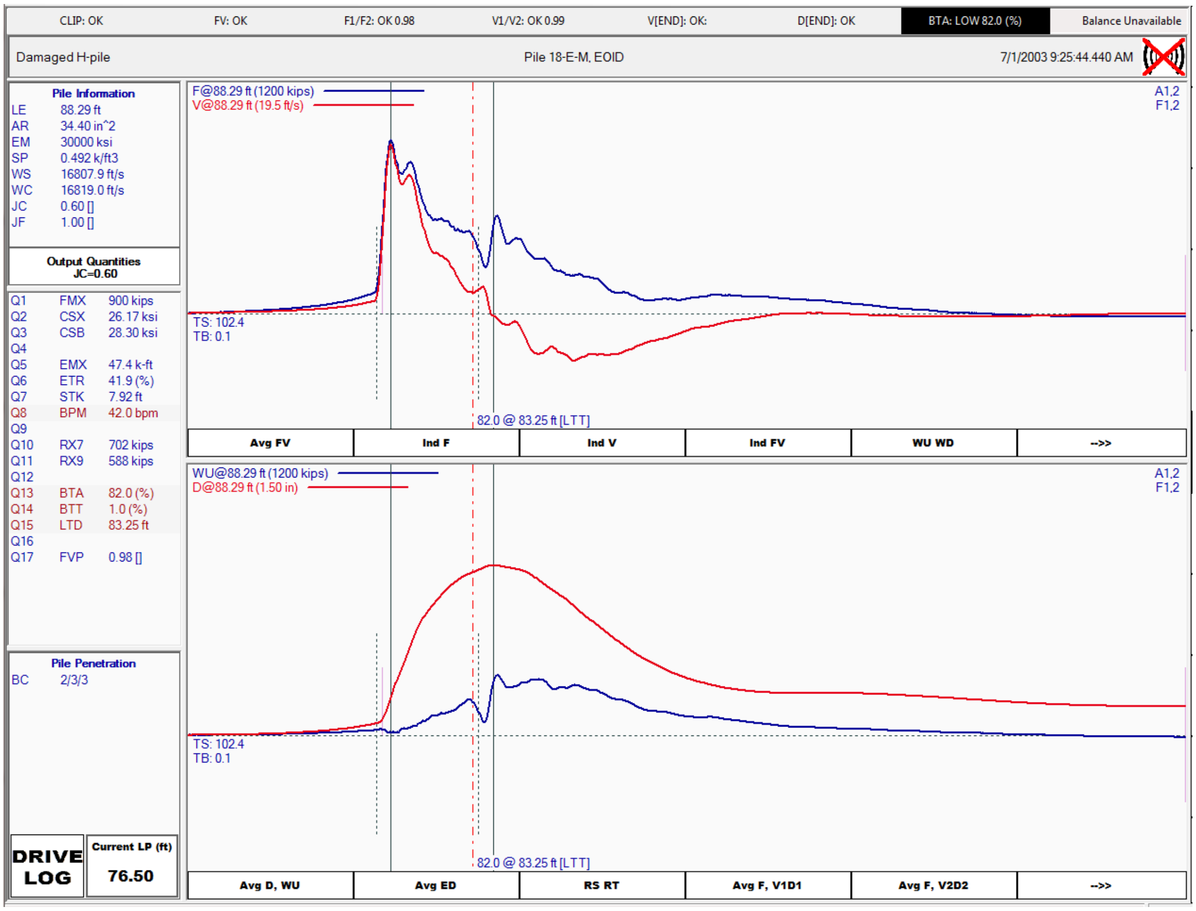


Figure 10-31 Example dynamic test records indicating pile damage.

Figure 10-32 Photographs of extracted pile showing damage.

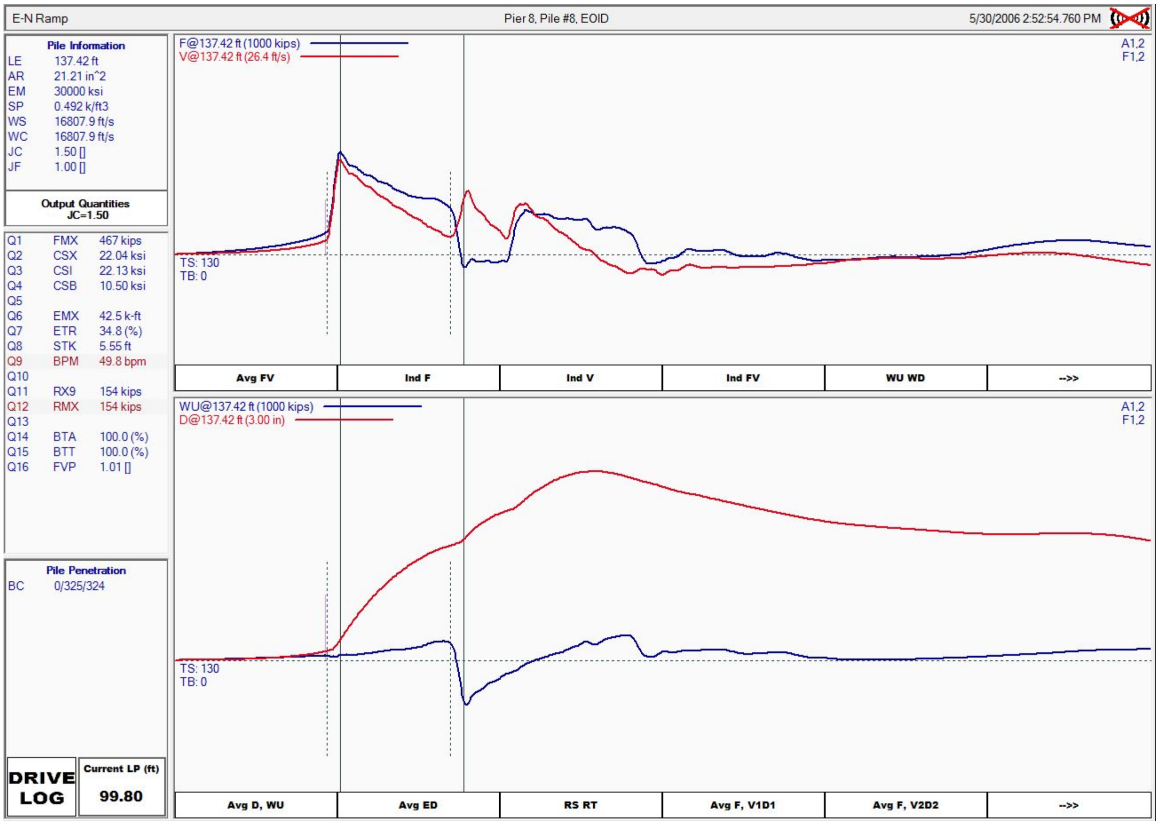


Figure 10-33 Example test record illustrating soil setup - end of initial driving data.

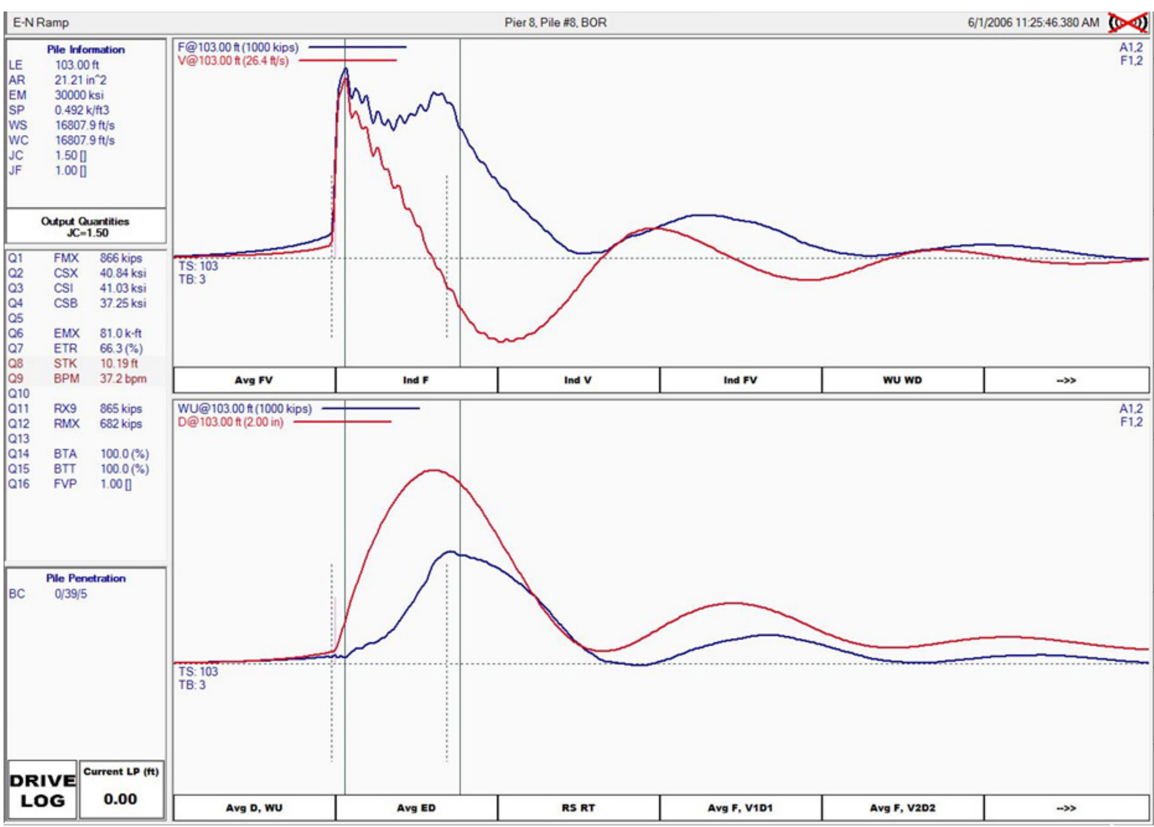


Figure 10-34 Example test record illustrating soil setup - beginning of restrike data.

Figure 10-33 presents a representative dynamic test record obtained near final driving. The pile penetration resistance or blow count at the end of initial driving was 9 blows per foot at an average hammer stroke of 5.8 feet. For the presented hammer blow, the impact force was 467 kips, the transferred energy was 42.5 ft-kips, and the energy transfer ratio was 34.8%. The nominal resistance based on the Case Method solution was 154 kips which was well below the 765 kips required. This was substantiated by signal matching analysis which indicated a nominal resistance of 129 kips, with 75 kips of the resistance carried by shaft resistance and 54 kips through toe resistance.

Figure 10-34 presents a representative dynamic test record obtained near the beginning of restrike 3 days later. The blow count at the beginning of restrike was 8 blows per inch at an average hammer stroke of 10.1 feet. Hence, both the restrike blow count and hammer stroke were significantly higher. For the blow presented, the impact force was 866 kips, the transferred energy was 81.0 ft-kips, and the energy transfer ratio is 66.5%. The nominal resistance based on the Case Method solution was 682 kips, or roughly 4.4 times the nominal resistance at the end of driving. A nominal resistance of 737 kips with 472 kips of shaft resistance and 265 kips of toe resistance was determined by signal matching. Long term restrike tests showed that, with additional time, the 765 kip nominal resistance would be obtained.

The increased nominal resistance during restrike is visually apparent when comparing the end of initial driving and beginning of restrike test records. The separation between the force and velocity records between time 0 and time 2L/C is substantially greater in the restrike test data compared to the end of initial driving data. This is also apparent in the restrike wave up record which has a much greater magnitude as well as significantly steeper slope compared to the end of driving wave up record.

The magnitude of the separation between the force and velocity records in the restrike data substantially increases beginning near the midpoint between time 0 and time 2L/C. This is also apparent in the wave up graph which has a significant change in slope occurring at the same time. These records indicate that the much larger shaft resistance present during restrike developed primarily over the lower half of the pile.

A dramatic change in the test records also occurs at, and immediately, after 2L/C. This indicates a change in the toe resistance and dynamic toe response. Signal matching analysis indicated an increase in the toe resistance, an increase in both the shaft and toe damping, and a reduction in the soil quake at the pile toe.

10.8.6 Test Records Illustrating Relaxation

Test records for HP 12 x 74 H-pile was driven into very dense, clayey silt with a single acting diesel hammer are presented in Figures 10-35 and10-36. SPT N values in the clayey silt deposit ranged from 37 to 100 blows per foot with a SPT N value of 87 blows per foot closest to the pile toe elevation. Dynamic testing was performed on the pile during initial driving and 3 days later during restrike. A nominal resistance of 408 kips was required based on resistance verification using dynamic testing with signal matching.

Figure 10-35 presents a representative dynamic test record obtained near final driving. The pile penetration resistance or blow count at the end of initial driving was 26 blows per foot at an average hammer stroke of 9.1 feet. For the presented hammer blow, the impact force was 738 kips, the transferred energy was 38.6 ft-kips, and the energy transfer ratio was 55.0%. The nominal resistance based on the Case Method solution was 456 kips, exceeding the 408 kips required. This was substantiated by signal matching analysis which indicated a nominal resistance of 471 kips, with 216 kips of the resistance carried by shaft resistance and 255 kips through toe resistance.

Figure 10-36 presents a representative dynamic test record obtained near the beginning of restrike. The blow count at the beginning of restrike was 3 blows per inch at an average hammer stroke of 8.3 feet. Hence, the restrike blow count was slightly higher, but the hammer stroke was reduced. For the blow presented, the impact force was 646 kips, the transferred energy was 27.0 ft-kips, and the energy transfer ratio was 38.5%. The nominal resistance based on the Case Method solution was 351 kips, or roughly 100 kips less than at the end of driving. Signal matching analysis indicated a nominal resistance at the beginning of restrike of 330 kips with 235 kips of shaft resistance and 95 kips of toe resistance.

The reduced nominal resistance during restrike is evident when comparing the end of initial driving and beginning of restrike records. The force and velocity records in the top portion of both figures exhibit less separation between force at velocity records at, and immediately after, the 2L/C marker. This is also apparent in the wave up graphs in the lower portion of both figures. The restrike record has a lower magnitude resistance occurring after the 2L/C marker as well as having a flatter slope.

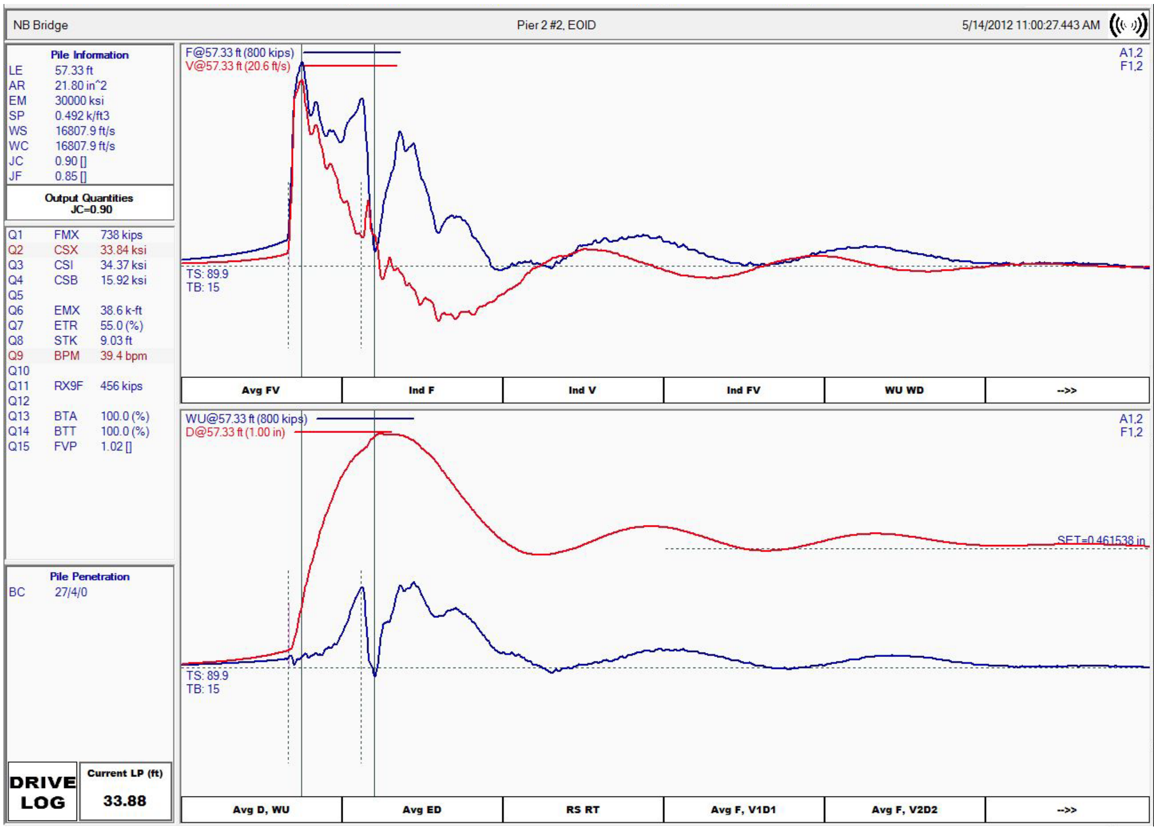


Figure 10-35 Example test record illustrating relaxation - end of initial driving data.

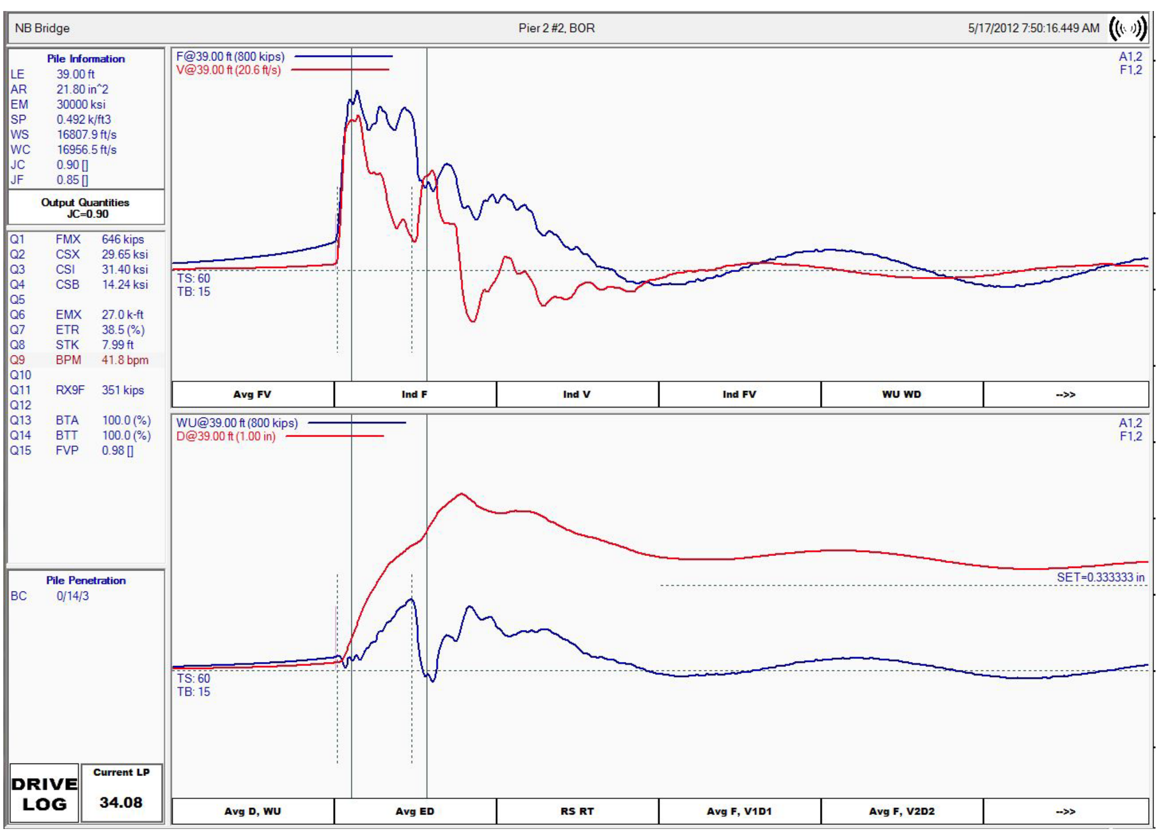


Figure 10-36 Example test record illustrating relaxation - beginning of restrike data.

10.8.7 Reporting of Dynamic Test Results

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 blow number to each hammer blow. By comparing the pile driving log with these blow numbers, numerical and graphical summaries of the dynamic testing results versus pile penetration depth and pile penetration resistance can be prepared.

An example of a numerical summary of dynamic testing results versus depth for a 24 inch square prestressed concrete pile driven with an APE D62-22 diesel hammer is presented in Figure 10-37. The test pile has a required nominal resistance of 1000 kips. Compression and tension stresses are limited to 3.80 ksi and 1.50 ksi, respectively. The accompanying graphical results are presented in Figure 10-38. Specification should require that the dynamic test data for each pile tested be processed versus pile penetration depth with the corresponding blow count in a similar manner. These numerical and graphical results can easily be compared to project requirements by construction personnel.

The effects of fuel setting adjustments as well as pile cushion changes are readily apparent in these graphical and numerical results. Near 43 feet, the diesel hammer fuel setting was increased from fuel setting 1 to 2. Dynamic test results at this depth illustrate an increase in both the compression and tension driving stresses, an increase in the hammer stroke, and an increase in transferred energy. A decrease in the pile penetration resistance also occurs as a result of the fuel setting adjustment even though a small increase in the nominal resistance occurs. Near 68 feet, the fuel setting is once again increased, this time from fuel setting 2 to 3. At that depth and time, the pile cushion is also replaced. When driving is resumed, both compression and tension driving stresses increase, the hammer stroke increases, and the transferred energy increases. The pile penetration resistance decreases as a result of the fuel setting adjustment even though a small increase in the nominal resistance once again occurs. Driving is temporarily stopped at the estimated pile penetration depth of 76 feet and a one hour restrike performed. Unfortunately, the nominal resistance slightly decreases during restrike requiring the test pile to subsequently be driven deeper for the required nominal resistance. The compression and tension driving stresses were maintained within specification limits throughout the test pile installation process.

GRL Engineers, Inc. Page 1
Case Method & iCAP® Results PDIPLOT2 2015.1.51.2 - Printed 30-September-2015

PDA Plot Example - 24" Prestressed Concrete Pile APE D62-22
OP: MLB Date: 26-June-2013

AR: 576.00 in^2 SP: 0.150 k/ft^3
LE: 118.00 ft EM: 6,346 ksi
WS: 14,000.0 f/s JC: 0.70 []

CSX: Max Measured Compr. Stress STK: O.E. Diesel Hammer Stroke
CSB: Compression Stress at Bottom EMX: Max Transferred Energy
TSX: Tension Stress Maximum RMX: Max Case Method Capacity

BL#	Depth	BLC	TYPE	CSX	CSB	TSX	STK	EMX	RMX
	ft	blows/ft		ksi	ksi	ksi	ft	k-ft	kips
8	21	8	AV5	1.93	0.54	1.37	6.16	28.1	218
18	22	10	AV10	1.93	0.58	1.36	5.94	26.9	231
25	23	7	AV7	1.97	0.60	1.38	6.01	27.9	235
34	24	9	AV9	1.96	0.61	1.37	5.95	27.0	248
44	25	10	AV9	1.99	0.62	1.40	6.00	27.5	259
57	26	13	AV13	2.00	0.68	1.36	6.04	27.2	287
68	27	11	AV11	2.03	0.69	1.41	6.10	28.0	287
81	28	13	AV10	2.04	0.75	1.37	6.10	27.7	300
94	29	13	AV13	2.01	0.73	1.36	6.05	27.0	297
104	30	10	AV8	2.02	0.73	1.39	6.04	27.3	287
115	31	11	AV11	1.94	0.62	1.39	5.86	26.2	268
124	32	9	AV9	2.01	0.67	1.39	6.01	27.3	264
134	33	10	AV10	2.03	0.64	1.42	6.03	27.9	244
142	34	8	AV8	2.07	0.64	1.46	6.15	29.2	227
150	35	8	AV8	1.96	0.55	1.42	5.88	27.2	208
159	36	9	AV9	1.92	0.54	1.38	5.84	26.1	207
171	37	12	AV11	2.07	0.66	1.40	6.18	28.5	241
189	38	18	AV18	2.11	0.79	1.32	6.32	28.4	339
218	39	29	AV29	2.16	1.00	1.19	6.50	27.7	447
255	40	37	AV37	2.23	1.16	1.10	6.67	28.7	526
304	41	49	AV49	2.25	1.23	1.03	6.68	29.0	566
356	42	52	AV52	2.24	1.24	0.95	6.63	28.7	572
412	43	56	AV56	2.20	1.25	0.87	6.51	27.4	564
457	44	45	AV45	2.44	1.36	1.02	7.26	33.4	585
497	45	40	AV40	2.64	1.44	1.14	7.85	38.5	601
536	46	39	AV39	2.70	1.44	1.17	8.02	40.0	595
571	47	35	AV35	2.77	1.45	1.20	8.29	42.4	592
605	48	34	AV34	2.85	1.50	1.25	8.46	44.5	599
642	49	37	AV37	2.86	1.52	1.22	8.43	44.3	606
679	50	37	AV37	2.87	1.55	1.20	8.39	44.7	611
719	51	40	AV40	2.88	1.56	1.20	8.38	45.0	613
754	52	35	AV35	2.90	1.57	1.19	8.36	45.3	603
791	53	37	AV37	2.92	1.57	1.22	8.34	45.5	595
830	54	39	AV38	2.99	1.60	1.21	8.48	47.1	602
864	55	34	AV34	3.04	1.66	1.23	8.54	48.4	616
907	56	43	AV43	3.06	1.74	1.22	8.58	48.9	647
954	57	47	AV47	3.07	1.83	1.16	8.58	49.0	664
1004	58	50	AV50	3.13	1.90	1.14	8.71	50.5	675
1053	59	49	AV44	3.11	1.96	1.08	8.58	49.2	680
1106	60	53	AV53	3.13	1.99	1.07	8.61	49.5	684
1155	61	49	AV49	3.11	2.02	1.02	8.57	48.7	669
1208	62	53	AV53	3.11	2.02	1.02	8.55	48.5	658
1258	63	50	AV50	3.05	1.98	1.00	8.41	47.3	639
1308	64	50	AV50	2.94	1.92	0.96	8.21	44.7	602
1354	65	46	AV46	2.88	1.91	0.91	8.19	43.9	570
1404	66	50	AV50	2.82	1.91	0.83	8.14	43.0	549
1452	67	48	AV48	2.80	1.93	0.77	8.17	42.6	545
1501	68	49	AV49	2.80	1.94	0.74	8.26	42.6	542
1549	69	48	AV41	2.90	2.02	0.75	8.73	45.3	600
1593	70	44	AV44	3.18	2.15	0.88	8.83	49.0	594
1638	71	45	AV45	3.16	2.18	0.82	8.80	48.4	600
1687	72	49	AV49	3.13	2.20	0.76	8.86	47.9	627
1740	73	53	AV53	3.09	2.21	0.70	8.99	48.3	665
1798	74	58	AV52	3.02	2.22	0.63	8.91	47.2	706
1855	75	57	AV57	2.99	2.23	0.59	9.09	47.8	743
1904	76	49	AV47	3.09	2.20	0.78	9.02	49.6	735
1955	77	51	AV51	3.25	2.22	0.88	8.95	52.9	711
1995	78	48	AV40	3.15	2.18	0.80	8.96	50.7	694
2018	79	48	AV23	3.24	2.22	0.86	8.93	51.8	710
2065	80	47	AV47	3.34	2.34	0.83	8.96	54.3	761
2119	81	54	AV54	3.31	2.40	0.74	9.06	53.8	847
2182	82	63	AV63	3.24	2.47	0.60	9.10	51.9	938
2258	83	76	AV76	3.13	2.55	0.47	9.08	49.6	1,011
2367	84	109	AV109	3.15	2.57	0.50	9.21	50.7	1,028
2462	85	95	AV95	3.20	2.45	0.58	9.32	51.7	1,006

Figure 10-37 Typical tabular presentation of dynamic test results.

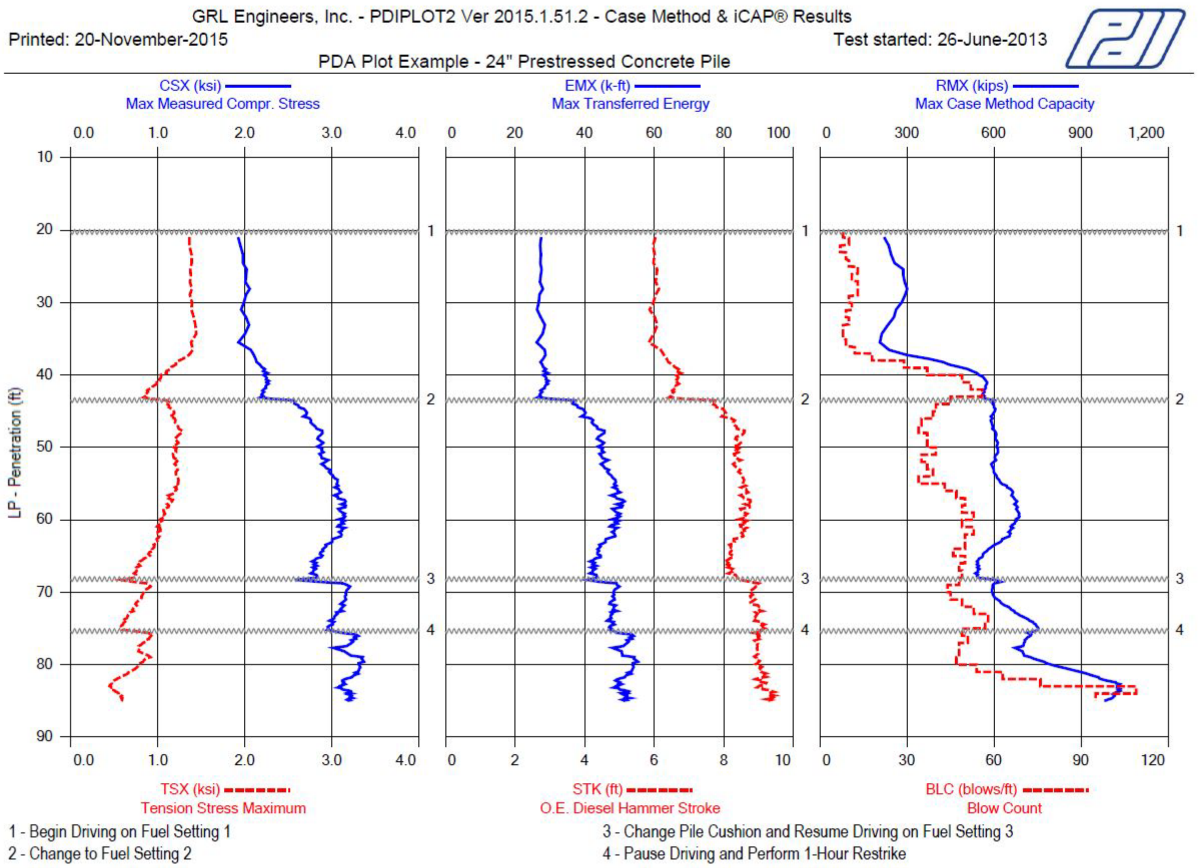


Figure 10-38 Typical graphical presentation of dynamic test results versus depth.

10.9 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 98 foot thick deposit of moderately clean, medium dense to dense sands with SPT N values ranging from 10 to 50. Based upon these conditions, the foundation report recommended 12.75 inch O.D. closed end pipe piles be used for the bridge abutment foundations. The pipe piles had an estimated length of 39 feet for a nominal resistance of 326 kips. 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 nominal resistance versus penetration depth during the test pile driving. More rigorous signal matching 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 12.75 inch pipe pile drove beyond the estimated pile penetration depth without developing the required nominal resistance. The pile was driven to a depth of 75 feet and had an end of drive nominal resistance of 235 kips. A restrike dynamic test performed one day after initial driving indicated the nominal resistance increased slightly to 245 kips.

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 16 inch O.D. closed end pipe pile was performed to determine if a larger diameter pipe pile could develop the required nominal resistance and, if so what pile penetration depth was necessary. The 16 inch pile was driven to a depth of 89 feet and had an end of drive nominal resistance of 222 kips. A one day restrike test on this pile indicated a nominal resistance of 280 kips. The 16 inch pile was driven deeper following the restrike test to a final penetration depth of 112 feet. With the additional driving, the nominal resistance at the end of redrive decreased to 240 kips.

Approximately two weeks later, a 12.75 O.D. closed end pipe pile and a 14 inch diameter Monotube pile with a 25 foot tapered lower section were driven at Abutment 2. The 12.75 inch pipe pile was driven to a penetration depth of 95 feet with an end of drive nominal resistance of 105 kips. The Monotube pile was driven to a depth of 43 feet and had an end of drive nominal resistance of 190 kips. One day restrike tests on both piles indicated a slight increase in nominal resistance to 180 kips and 205 kips, respectively. During this same site visit, a 16 day restrike test was performed on the 16 inch pipe pile. The long term restrike nominal resistance for the 16 inch pipe pile was 400 kips.

The dynamic testing results from both abutments indicated that the required nominal resistance could not be obtained at or near the estimated pile penetration depth with the 12.75 inch pipe piles. However, two foundation solutions were indicated by the dynamic testing results. If a reduced nominal resistance were chosen, the test results indicated a Monotube pile driven to a significantly shorter penetration depth could develop about the same nominal resistance as could be developed by the 12.75 inch pipe piles. Alternatively, if the original nominal resistance was desired, 16 inch pipe piles could be driven on the order of 92 feet below grade.

Although not originally planned, two static load tests were performed to confirm the nominal resistance that could be achieved at the site. The 12.75 inch pipe and the 14 inch Monotube piles at Abutment 2 were selected for testing. The static load test results indicated the 12.75 inch pipe pile with a pile penetration depth of 95 feet had a nominal resistance of 230 kips and the Monotube pile with a pile penetration depth of 43 feet had a nominal resistance of 220 kips. The dynamic test nominal resistances determined during restrike 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 the nominal resistances 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.

10.10 ADVANTAGES, DISADVANTAGES, AND LIMITATIONS

An advantage of dynamic testing over other methods of nominal resistance verification is the additional information gained on the pile installation process. In addition to providing estimates of nominal resistance during driving and during restrike, dynamic test data can be used to check hammer and drive system performance, to monitor driving stresses, and to assess pile structural integrity.

Many piles can also be dynamically 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 nominal resistance versus depth, nominal resistance variations between locations, and nominal resistance 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 signal matching analysis can be used to develop pile driving criterion. A procedure describing the use of dynamic test results to refine wave equation input parameters and wave equation analysis results is described in Section 12.6.9.

A disadvantage with dynamic testing for determining the nominal resistance can be the pile driving system. The pile hammer must be capable of mobilizing 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 pile penetration resistances approach 10 blows per inch, the soil resistance may not be fully mobilized at and near the pile toe. In these circumstances, dynamic test capacities tend to produce lower bound estimates of the nominal resistance. If available, a larger pile hammer or higher hammer stroke can be used to increase the net pile penetration per blow and thereby mobilize more resistance, if present.

Dynamic testing estimates of nominal resistance also indicate the nominal resistance at the time of testing. Since increases and decreases in the pile nominal resistance 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 nominal resistance. This may require an additional move of the pile driving rig for restrike testing.

A limitation of dynamic testing can be the geotechnical failure mechanism. Large 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 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 nominal resistance under static loading conditions. Interpretation of test results by experienced personnel is important in these situations.

10.11 PRACTICAL ISSUES AND CONSIDERATIONS

Several practical issues and considerations should be clearly understood by the parties responsible for analyzing and reviewing dynamic test results. Some of the more common issues encountered include:

- Understanding that specifying “a dynamic test” does not implicitly require the dynamic test personnel to furnish driving criteria.

  Like any other engineering service, dynamic test results should be analyzed and reviewed. While it is often desirable to continue driving production piles immediately after the dynamic test is complete, time should be allocated for analysis and reporting of test results. If the driving criteria is to be determined from the dynamic test results, that should be clearly identified in the project specifications as well as when the driving criteria is to be furnished to the owner or contractor.

- Understanding the limitations of dynamic tests in easy driving and hard driving situations.

  At pile penetration resistances less than 24 blows per foot and above 120 blows per foot, dynamic test and analysis methods can overpredict and underpredict the nominal resistance, respectively. At low blow counts (high set per blow), it is difficult for dynamic methods to easily separate the static and dynamic soil resistance effects resulting in a tendency to overpredict the static resistance. Use of a reduced hammer stroke or lower fuel setting can help improve the accuracy of dynamic methods in low blow count situations. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound nominal resistance estimates as not all of the resistance (particularly at and near the toe) is fully activated. In these high blow count situations, use of a larger hammer stroke, higher fuel setting, pile hammer with a greater rated energy, or variable stroke drop hammer can help improve dynamic method accuracy.

- Understanding that a dynamic test provides the mobilized nominal resistance at the time of testing.

  The nominal resistance determined in a dynamic test is the mobilized resistance at that particular time. The nominal resistance of a pile typically changes over time. It may increase (soil setup) or decrease (relaxation).

Dynamic tests must be performed at both the end of initial driving and during restrike at a later time in order to quantify time dependent changes in nominal resistance.

- Knowing how to perform and evaluate restrike dynamic test results.

  Ideally, the hammer stroke or fuel setting is selected such that the penetration resistance at the beginning of restrike falls between 3 and 10 blows per inch. In this situation, the test record to select for signal matching analysis is readily apparent. An early, high energy blow, with good data quality should be selected and analyzed for the nominal resistance. The restrike blow count should be carefully recorded over the full restrike event as the rate by which the blow count decreases from inch to inch can be helpful. When the restrike blow count is less than 24 blows per foot, a lower energy restrike blow should be chosen for signal matching analysis to reduce the potential for overpredicting the nominal resistance.

  In more difficult situations, limited pile movement may occur during restrike and several records may need to be analyzed with signal matching. Superposition of the activated shaft resistances under various restrike hammer blows may be used to assess the nominal pile resistance. In initial restrike blows, the shaft resistance may be mobilized along the upper portion of the pile shaft. Later restrike blows may indicate more shaft resistance on the lower portion of the pile once the upper shaft resistance has started to breakdown. The toe resistance and shaft resistance on the lower portion of the pile from the end of drive analysis should also be reviewed and, if appropriate, used in a superposition case. When using the toe resistance from an end of drive situation, the analyst should be confident that relaxation in the toe bearing layer is not a consideration or overestimation of the nominal resistance could result by using superposition.

- Difficulty to accept that, in some cases, dynamic measurements may provide conservative predictions of the true geotechnical resistance and correlations and extrapolation between dynamic and static load test results are necessary.

  Dynamic methods can yield conservative estimates of the true geotechnical resistance in some situations. Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loading conditions, the soil inside a pipe pile or between H-pile flanges may slip and produce internal shaft resistances.

Under static loading conditions, this soil may plug and move with the pile resulting in toe resistance over the full pile cross section. Hence both shaft and toe resistances may be different in open profile pile sections under static and dynamic loading conditions. Plugging behavior can also vary in different geomaterials. Careful interpretation and extrapolation of dynamic results is required in these situations.

- Understanding modeling uncertainties in signal matching analysis that can affect the reported soil resistance distribution.

  A portion of the soil resistance calculated on an individual soil segment in a signal matching analysis can usually be shifted up or down the shaft one soil segment without significantly altering the overall match quality. Similarly, it may be possible to shift a portion of the soil resistance from the last shaft segment to the pile toe or vis versa without significantly altering match quality. Therefore, use of the signal matching determined soil resistance distribution for uplift, scour, drag force, and other geotechnical considerations should be made with an understanding of these analysis limitations.

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CHAPTER 11

RAPID LOAD TESTING

The nominal resistance of driven pile foundations in axial compression can be evaluated by rapid load test methods. Rapid load tests methods can be applied to all driven pile types on land or over water. The test methods can provide time and cost savings where high loads are required or access is difficult. ASTM D7383, Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations, provides additional details on rapid load test methods and procedures.

11.1 REQUIREMENTS FOR RAPID LOAD TESTS

In general, the previously stated reasons and prerequisites for a static load test program described in Section 9.1.1 are valid for rapid load test programs. Rapid load tests can be used to provide verification or refinement of the foundation design, confirm the nominal geotechnical resistance, and, in some instances, quantify the p-y response of laterally loaded piles. Knowledge of the subsurface stratigraphy including the engineering parameters of the geomaterials should be known prior to performing a rapid load test.

Rapid Load Tests procedures are standardized in ASTM D7383 (2010). In this method, a loading apparatus generates a force pulse that will result in an applied pile head force versus time plot as shown in Figure 11-1. A target peak force is determined that should exceed the nominal geotechnical resistance plus the dynamic soil resistance. The target peak force is based on soil types, pile type and other project requirements. The applied force should exceed 50% of the actual peak force for a time duration of 4L/C or four times the pile length, L, divided by the pile material wave speed, C. The force applied must also exceed the static weight applied to the pile head due to the apparatus prior to the test, known as the pre-load force, for a time duration of at least 12L/C. Shorter time durations than 12L/C can be acceptable if additional force or movement measurement devices are used along the pile length at a distance described in the ASTM standard.

The force pulse should be measured by a calibrated force transducer placed between the loading apparatus and the pile head with the rated transducer capacity being at least 10% higher than the target peak force. The resulting pile head displacement should be measured by one or more calibrated displacement transducers. The ASTM standard also allows the Engineer to approve secondary pile head displacement measurement devices including redundant displacement transducers or accelerometers. As noted above, additional devices to measure force or displacement can be used along the pile's length when the force pulse duration is less than 12L/C. All force and displacement measurements should be recorded versus time on an appropriate recording device with an appropriate sampling frequency and signal conditioning.

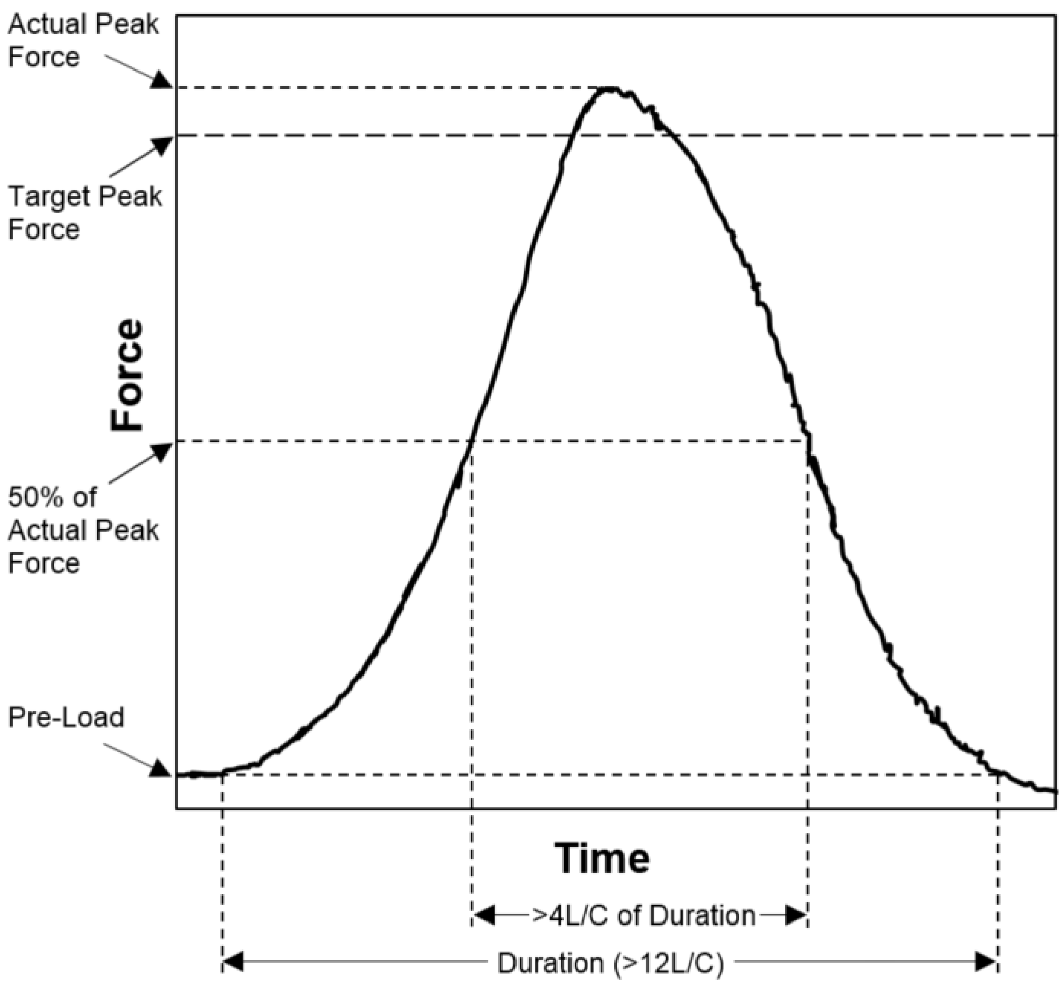

Figure 11-1 Typical axial compressive force pulse (after ASTM D7383).

11.2 BACKGROUND ON RAPID LOAD TEST METHODS

Rapid load testing can be performed using a combustion gas and a reaction mass or with a cushioned drop weight system. These methods will be described in subsequent sections.

11.2.1 Combustion Gas and Reaction Mass Apparatus (Statnamic)

The Statnamic testing method was developed in 1988 by Berminghammer Foundation Equipment and TNO, the Dutch governmental organization for applied scientific research. Bermingham and Janes (1989) described the method which uses solid fuel burned within a pressure chamber to rapidly accelerate upward the reaction mass positioned on top of 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. A load cell and accelerometers measure load and acceleration. Typically, the reaction mass weighs a minimum of 5% of the target peak force. The Statnamic test method is licensed to a single source in the US.

Statnamic tests for evaluation of static pile capacity have been performed on steel, concrete and timber piles. At present, individual piles, or pile groups with a combined static and dynamic resistance less than 9,000 kips can be tested. Axial compression tests have been conducted on both vertical and battered piles. The test method has been used on land and over water.

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 11-2.

Development began in 1988 with a Statnamic device capable of a 22 kip test load. From 1988 through 1992, the test load capability was incrementally increased to 3600 kips. In 1994, a 6800 kip testing device was introduced. In 1998, a hydraulic catch mechanism was developed. The maximum test capacity was increased to 9000 kips in 2005.

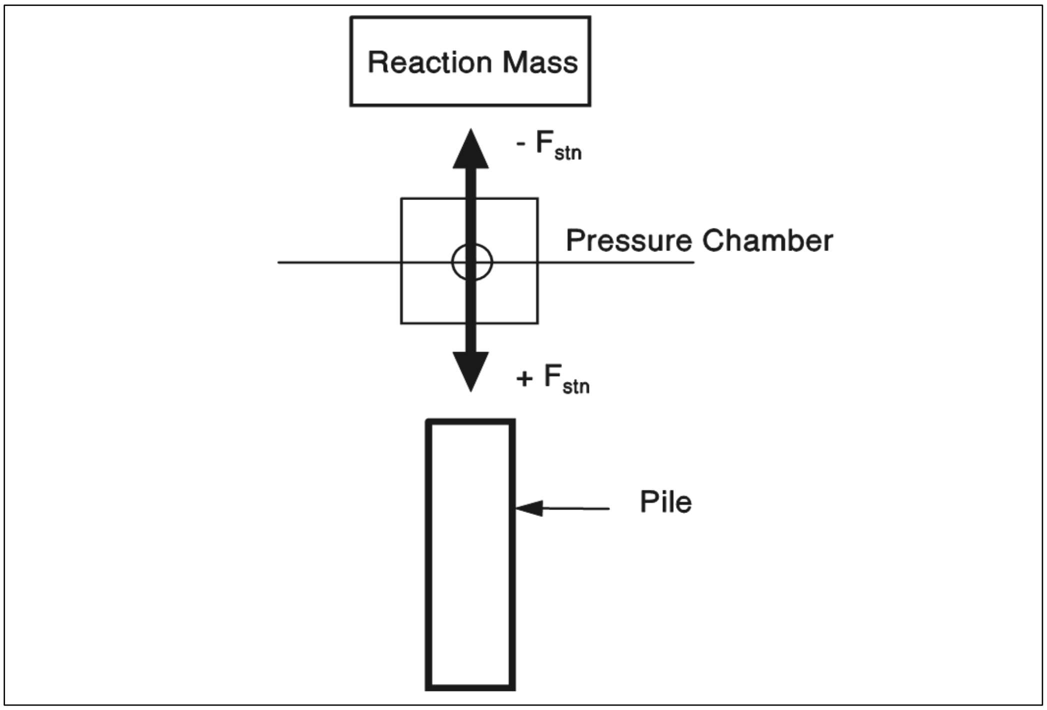

Figure 11-2 Statnamic concept (courtesy of Berminghammer Foundation Equipment).

A base plate is attached to the pile head. The load cell, accelerometer, 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 segmental reaction mass is then stacked on the launching cylinder and a catching mechanism is placed around the reaction mass.

Depending upon the test load, a hydraulic catch, mechanical catch, or gravel retention structure is used to catch the reaction mass. The hydraulic catch system shown in Figure 11-3 is used for test loads of 1000 kips. A mechanical catch system is used for test loads of up to 4,400 kips and a gravel retention structure as shown in Figure 11-4 is used for loads of up to 9,000 kips. For the gravel retention structure, gravel backfill is placed in the annulus between the reaction mass and the retention structure. After propellant ignition and reaction mass launch, the granular backfill slumps into the remaining void to cushion the reaction mass fall.

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. Load and displacement data from the load cell and accelerometers are recorded, digitized, and displayed immediately in the field. Typical raw signals of the load and displacement records are given in Figure 11-5. These signals can then be converted into a raw load - displacement curve as given in Figure 11-6, which requires interpretation to derive the static pile capacity.

Figure 11-3 1000 kip hydraulic catch device on prestressed concrete pile (courtesy of Applied Foundation Testing).

Figure 11-4 Statnamic test in progress with gravel catch mechanism - 9,000 kip device (courtesy of Applied Foundation Testing).

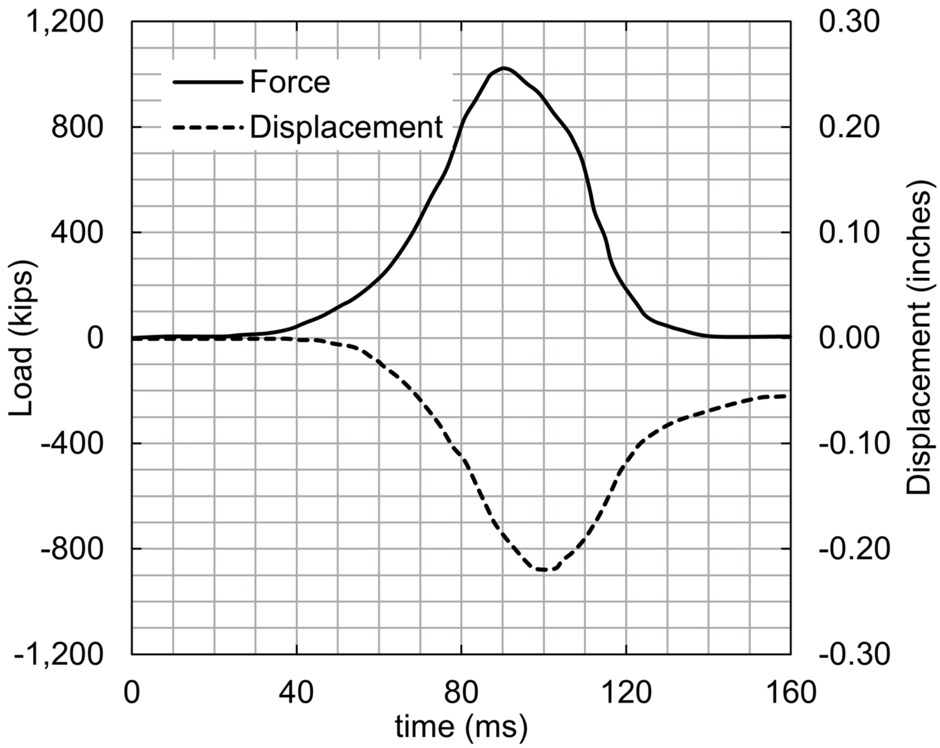

Figure 11-5 Statnamic raw force and displacement measurements versus time (courtesy of Berminghammer Foundation Equipment).

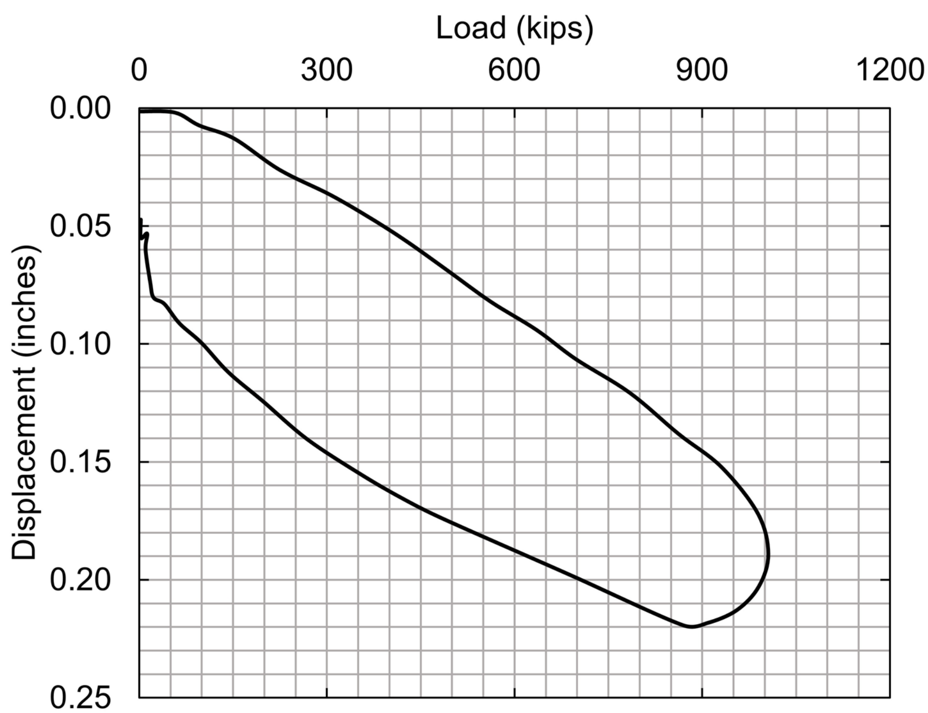

Figure 11-6 Statnamic load versus displacement (courtesy of Berminghammer Foundation Equipment).

11.2.2 Cushioned Drop Weight Systems

Rapid load tests can also be performed using cushioned drop weight systems that can generate the required peak force and force pulse duration. A drop weight, typically weighing between 5 and 15% of the target peak force, is mobilized to the site. A system (a crane, jack or some other mechanism) capable of lifting the drop weight to the required drop height and guiding it to strike the pile on center is also required. A load cell is once again located atop the pile head. However, in the cushioned drop weight systems, a spring system or a package of cushioning material is placed between the drop weight and the load cell. The springs or cushions lengthen the duration of the force pulse imparted to the pile. Some systems also include a clamping or catching mechanism to catch the rebounding drop mass after the force pulse has been applied. The catch mechanism prevents the application of additional force and improves the measurement of the pile head displacement from the main impact event.

The Pseudo Static Pile Load Tester, developed by Fundex (Schellingerhout and Revoort 1996), is a system with a clamp. This device, shown in Figure 11-7a, typically applies a series of drops from increasing release heights. Gradually, higher peak forces are applied and greater nominal resistances are mobilized, if present. The data acquisition and processing equipment is also shown in the photograph. Williams (2014) reported the system can achieve test loads up to 800 kips.

A large cushioned drop weight system for rapid load testing has also been reported by Miyasaka et al. (2009). In this system, named Hybridnamic, a modular ram of up to 85 tons is dropped on a cushion product specially developed to lengthen the force pulse duration. Other cushion materials, such as wood, are also viable. The drop height, drop weight and cushion thickness can be simulated with wave equation analysis. Drop weights of up to 170 kips are available, mobilizing nominal resistances of on the order of 1700 kips. A photograph of the Hybridnamic system is presented in Figure 11-7b.

Another drop weight system that can be used for rapid load testing is the APPLE system (Rausche et al. 2008). This system, shown in Figure 11-8, can be configured with a ram weight of up to 160 kips. The system is capable of mobilizing nominal resistances of on the order of 1600 kips using rapid load test procedures.

Figure 11-7 Drop weight rapid load test systems: (a) Fundex system (courtesy Foundation Constructors Inc.) and (b) Hybridnamic system.

Figure 11-8 APPLE 32 ton modular rapid load test system with transducer.

11.3 RAPID LOAD TEST APPLICATIONS

Rapid load tests are primarily used to determine the nominal axial compressive resistance. A variety of pile types and sizes from small diameter timber, steel, and concrete piles up to large diameter open end steel pipes and concrete cylinder piles have been tested using rapid load tests. Internal and external strain gage instrumentation has also been used on rapid load tested piles for both nominal resistance interpretation as described in Section 11.4 and for load transfer assessments.

Rapid load tests are attractive for testing large diameter open end pipe piles for two reasons. First, large diameter open end pipe piles often have high nominal resistances which are difficult to statically load test economically. Second, the longer duration, lower acceleration force pulse generated by a rapid load test, sometimes reduces slippage of the internal soil plug under the dynamic loading event.

Rapid load testing with a combustion gas apparatus has also been used for lateral load testing of piles and pile groups.

11.4 RAPID LOAD TEST INTERPRETATION METHODS

Initial correlations of Statnamic rapid load test results with static load tests for projects with toe bearing piles founded in till and rock showed good agreement without adjustment of the load - displacement results (Janes et al. 1991). However, in later tests, Statnamic rapid load test results overestimated the nominal resistance in some soils due to the dynamic loading rate effects (Janes and Campanella 1994). Several analysis procedures depending on pile length and pile response, as well as adjustment factors based on loading rate have subsequently been developed to derive the nominal resistance from Statnamic rapid load test results. These analysis procedures include the Unloading Point Method (UPM), the Modified Unloading Point Method (MUP), the Segmental Unloading Point Method (SUP), the Fully Mobilized UPM, and for cohesive soils the Sheffield Method. These methods are described in sections 11.4.1 through 11.4.5. Recommended loading rate reduction factors from NCHRP 21-08 by Paikowsky (2006) on Innovative Load Testing Systems for results analyzed with these rapid load test methods are discussed in Section 11.4.6.

In addition to selecting the appropriate interpretation method and applying an appropriate loading rate reduction factor, rapid load tested piles should have sufficient maximum displacement of the pile head. Miyasaka et al. (2009) noted that rapid load tests using the unloading point method with a maximum displacement of 1 to 3% of the equivalent pile diameter potentially overpredicted the nominal resistance compared to static load tests. Therefore, Miyasaka recommended running rapid load tests to a sufficiently large permanent displacement, on the order of 3% or more of the pile diameter. Holscher et al. (2012) recommended the maximum displacement of the pile head during the test be larger than 5% of the equivalent pile diameter to achieve geotechnical failure. Geotechnical failure is needed so that the dynamic soil resistance can be properly assessed and subtracted from the nominal resistance.

Middendorp and Bielefeld (1995) proposed the wave number, N_w, as a guide for determining whether the Statnamic test was influenced by stress wave behavior and to determine the analysis procedure to be used. The wave number considers the foundation length, the wave speed of the pile material, and the duration of loading and is calculated from:

$$N_w = \frac{D_w}{L} = \frac{C\ t_L}{L} \qquad \text{Eq. 11-1}$$

Where:

- N_w = wave number (unit less).
- D_w = wave length (feet).
- L = total pile length (feet).
- C = wave speed of pile material (ft/s).
- t_L = load duration (seconds).

11.4.1 Unloading Point Method (UPM)

The first widely used analysis method to adjust the raw Statnamic load - displacement results for dynamic loading rate effects was the Unloading Point Method (UPM) proposed by Middendorp et al. (1992).

In the UPM, all elements of the pile are assumed to 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 as long as the pile has a wave number, N_w, greater than 12. 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 11-9. 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:

$$F_{stn}(\mathrm{t}) = F_a(\mathrm{t}) + F_u(\mathrm{t}) + F_v(\mathrm{t}) + F_p(\mathrm{t}) \qquad \text{Eq. 11-2}$$

Where:

F_{stn} = Statnamic induced load (kips).
F_a = pile inertia force (kips).
F_u = static soil resistance (kips).
F_v = dynamic soil resistance (kips).
F_p = resistance from pore pressure (kips).

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

$$F_u(\mathrm{t}) = F_{stn}(\mathrm{t}) - F_a(\mathrm{t}) - F_v(\mathrm{t}) - F_p(\mathrm{t}) \qquad \text{Eq. 11-3}$$

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

$$F_u(\mathrm{t}) = F_{stn}(\mathrm{t}) - F_a(\mathrm{t}) - F_v(\mathrm{t}) \qquad \text{Eq. 11-4}$$

Consider the Statnamic load - displacement data presented in Figure 11-10. The Statnamic load - displacement data can be separated into five stages. Stage 1 includes the 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.

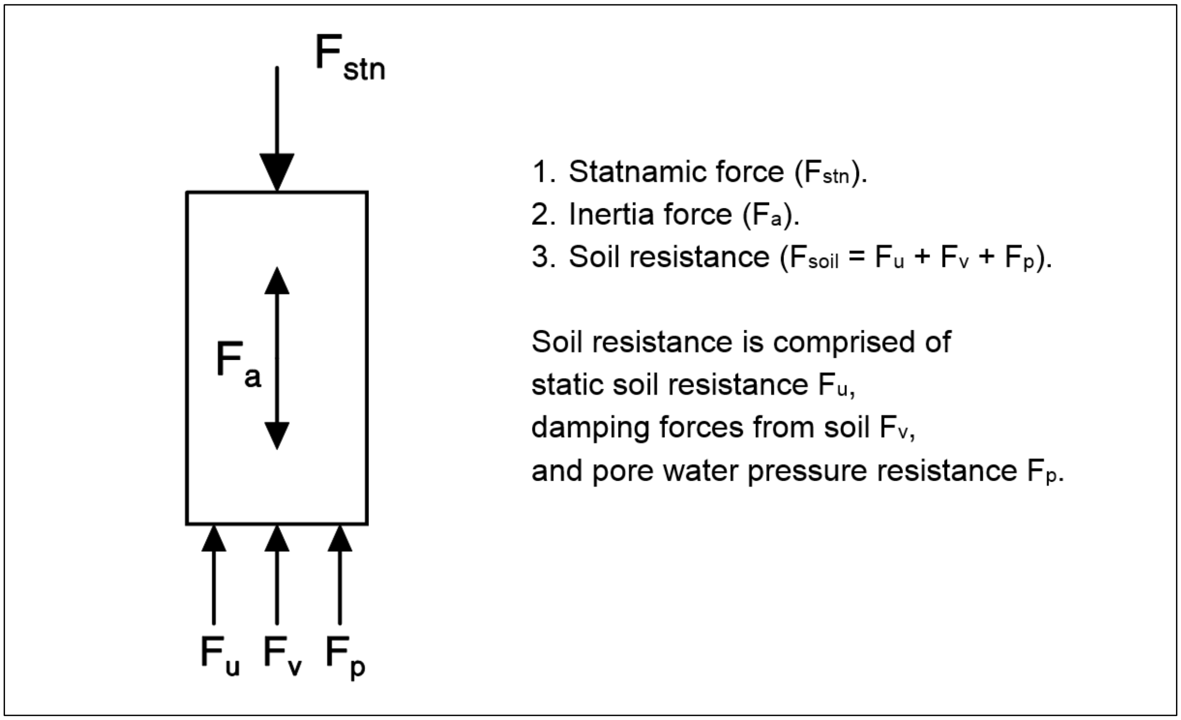

Figure 11-9 Free body diagram of pile forces in a Statnamic test (after Middendorp et al. 1992).

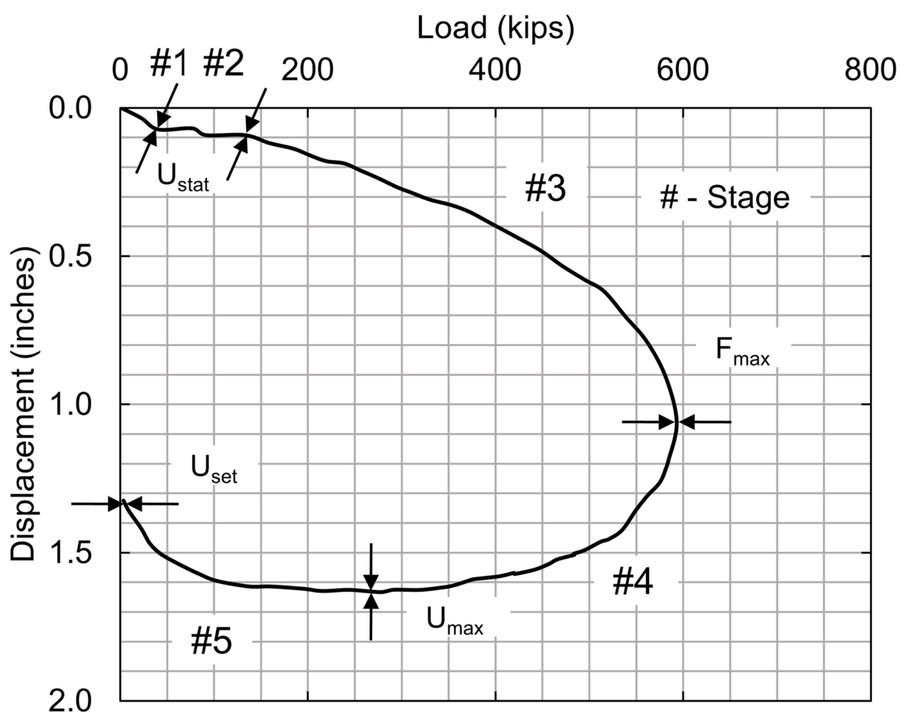

Figure 11-10 Five stages of a Statnamic test (after Middendorp et al. 1992).

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:

$$F_u(\mathrm{t}_{umax}) = F_{stn}(\mathrm{t}_{umax}) - F_a(\mathrm{t}_{umax}) \qquad \text{Eq. 11-5}$$

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 nominal 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 nominal 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 Chapter 9 is used for load test interpretation, the nominal resistance may be less than this nominal 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 nominal resistance, 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:

$$C_4 = \frac{F_{stn}(\mathrm{t}_4) - F_u(\mathrm{t}_{umax}) - \mathrm{ma}(\mathrm{t}_4)}{V(\mathrm{t}_4)} \qquad \text{Eq. 11-6}$$

Where:

C_4	=	damping constant.
F_{stn}	=	Statnamic induced load.
F_u	=	static soil resistance.
m	=	mass of pile.
a	=	acceleration of pile.
V	=	velocity of pile.
t_4	=	time at beginning of Stage 4 (see Figure 11-10).
t_{umax}	=	time at maximum displacement (see Figure 11-10).

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:

$$F_u(\mathrm{t}) = F_{stn}(\mathrm{t}) - \mathrm{ma}(\mathrm{t}) - \mathrm{C}_4\mathrm{V}(\mathrm{t}) \qquad \text{Eq. 11-7}$$

Where:

F_u	=	static soil resistance.
F_{stn}	=	Statnamic induced load.
m	=	mass of pile.
a	=	acceleration of pile.
C_4	=	damping constant.
V	=	velocity of pile.

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 with the Unloading Point Method illustrating how the dynamic rate effects are subtracted from the Statnamic results is presented in Figure 11-11.

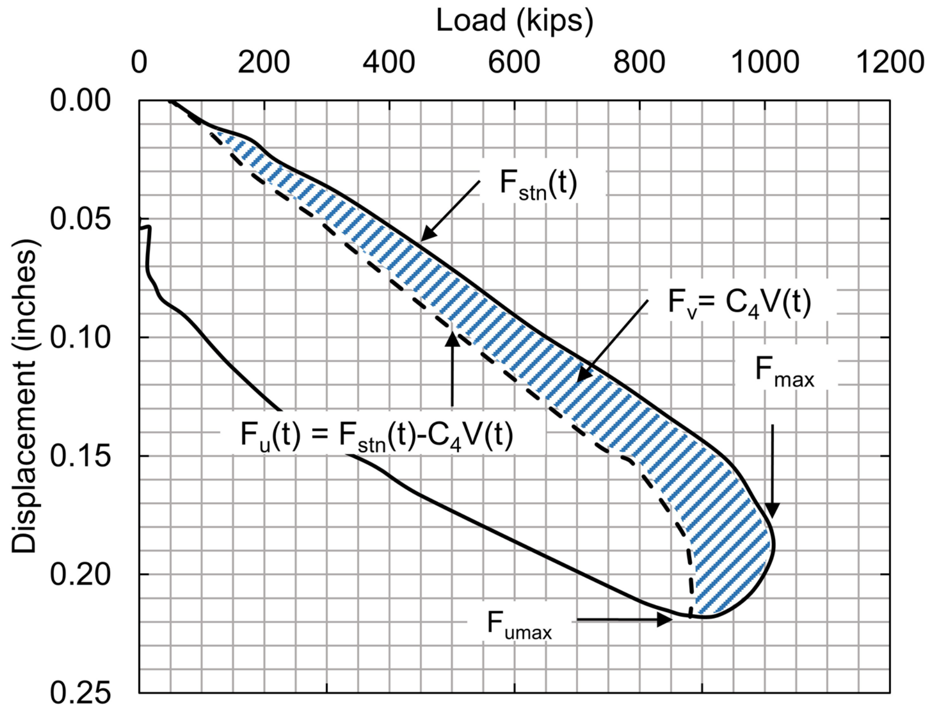

Figure 11-11 Derived Statnamic load displacement curve with rate effects (courtesy of Berminghammer Foundation Equipment).

11.4.2 Modified Unloading Point Method (MUP)

The Unloading Point Method rigid body assumption is not applicable for piles with a high toe resistance. On these piles, the pile head response (acceleration, velocity, and displacement) is significantly different than that at the pile toe. Because of the shortcomings of the UPM in this condition, the Modified Unloading Point Method (MUP) was developed by Justason (1997). The MUP method requires adding an additional accelerometer at the pile toe to define the toe behavior. The MUP method still assumes the pile to be a single mass but the acceleration of the mass is defined from the average of the pile head and toe displacement. The MUP method then uses the previously described UPM analysis procedure using the applied Statnamic force and the average accelerations and velocities.

11.4.3 Segmental Unloading Point Method (SUP)

Analytical studies by Brown (1995) have shown that the rigid body assumption used in the Unloading Point Method can result in overprediction of the nominal resistance and is not appropriate for long slender piles. Analysis of relatively long piles with a wave number, N_w, less than 10 was also problematic with the averaging techniques used in the Modified Unloading Point Method because of the time delay between the movement of the pile head and the movement of the pile toe and the resulting phase shift of the signals. To address this condition, the Segmental Unloading Point (SUP) Method was developed, and is described in Mullins et al. (2002). The SUP method separates the pile into discrete segments of shorter length where strain gages and accelerometers are used to calculate the force, acceleration, velocity and displacement of each segment. Details of the computation procedures may be found in Mullins et al. (2002), and in NCHRP 21-08, Paikowsky (2006).

The maximum number of segments is generally controlled by the number of strain gages. However, each strain gage level does not constitute a segment. Strain gage placement is usually determined by soil stratigraphy considerations. Multiple strain gages can be placed in a segment. The segment length is selected independent of gage location and must produce a wave number greater than 12.

The SUP method performs MUP analyses for each segment. The pile head derived static resistance is then calculated by summing the derived static response of each pile segment.

11.4.4 Fully Mobilized UPM

Miyasaka et al. (2009) proposed a modification to the unloading point method to avoid overprediction of the nominal resistance in cases where limited axial movement occurs. A case study was presented for a statically and rapidly load tested steel sheet pile installed in a mixed profile consisting of 9.5 feet of medium sand (SPT N=10), 8.9 feet of very loose silt (N=0), 16.4 feet of gravelly sand (N = 10 to 20), and 3.9 feet of stiff, gravelly clay, underlain by fine dense sand (N = 50+). UPM results indicated an overprediction by as much as 1.43 times the nominal resistance if sufficient maximum and net pile head movement were not achieved. A comparison of the UPM results for each load cycle with the static load test result is presented in Figure 11-12. The drop height for rapid load tests range from 0.6 feet to 4.9 feet. Note the UPM and static load test results agree reasonable well once a maximum nominalized displacement exceeding 2.8% and a normalized net displacement in excess of 1.5% are achieved. Thus, for full mobilization of the geotechnical resistance using UPM results, Miyasaka proposed obtaining a minimum net permanent displacement of at least 3% of the pile diameter. ASTM D7383 also cautions on possible overprediction of the nominal resistance in rapid load test analysis cases with insufficient axial movement.

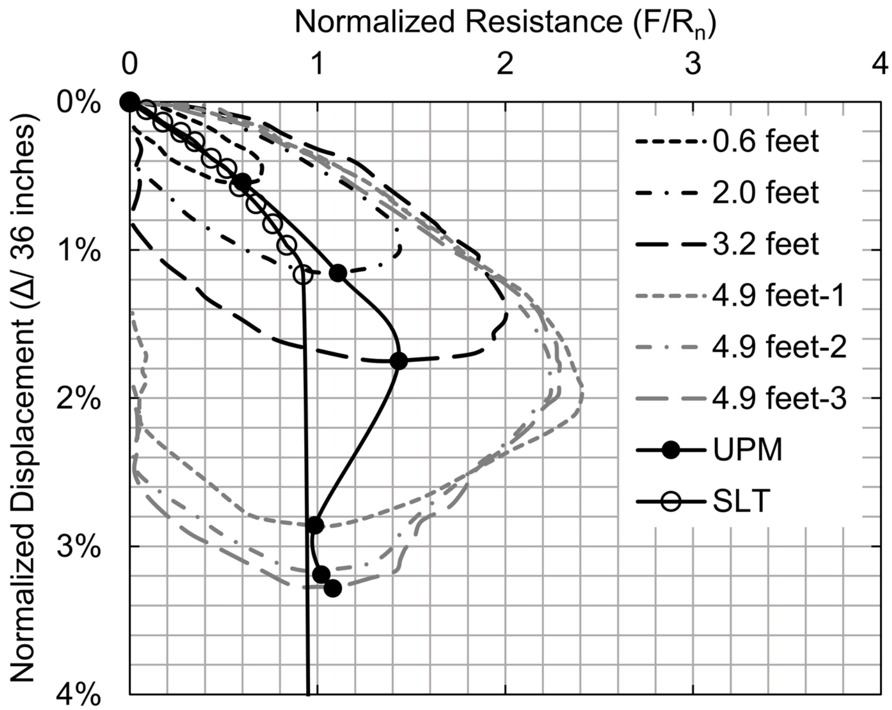

Figure 11-12 Normalized resistance versus normalized displacement (after Miyasaka et al. 2009).

11.4.5 Sheffield Method for Cohesive Soils

Brown (2004), as well as Brown and Hyde (2006), describe an analysis method that incorporates loading rate effects in cohesive soils. Details of this method, the Sheffield Method, can be found in Holscher et al. (2012) Appendix D. The Sheffield Method incorporates the rate effect variation with pile settlement, and does not include rate reduction factors from Paikowsky (2006). It is applicable for piles installed in clay and assumes the majority of the nominal resistance is derived from shaft resistance. Holscher et al. (2012) cautioned strongly that this method not be used in sands or rock. The Sheffield Method depends on specific rate parameters, denoted α and β. It is preferable that the parameters α and β be determined from triaxial tests performed at variable displacement rates. However, if laboratory test data is not available, published literature values may be used. Rule of thumb values for α have also been empirically correlated to soil index tests from a relatively small data set, predominantly from cohesive soils from Europe, and $\beta = 0.2$. The Sheffield Method is show in Equation 11-8.

$$F_u(\mathrm{t}) = \frac{F_t(\mathrm{t}) - \mathrm{ma(t)}}{1 + \frac{F_t(\mathrm{t})}{F_{max}}\alpha\left[\left(\frac{\mathrm{V(t)}}{V_o}\right)^{\beta} - \left(\frac{V_{static}}{V_o}\right)^{\beta}\right]} \qquad \text{Eq. 11-8}$$

In which:

$$\alpha = 0.031\,\mathrm{PI} + 0.46 \qquad \text{Eq. 11-9}$$

Where:

F_u	=	static soil resistance.
F_t	=	force measured on pile head.
F_{max}	=	maximum measured force applied during testing.
m	=	mass of pile.
a	=	acceleration of pile.
V	=	velocity of pile.
V_{static}	=	velocity used to determine parameters α and β.
V_o	=	normalizing constant.
α	=	material dependent parameter 1.
β	=	material dependent parameter 2.
PI	=	Plasticity Index.

11.4.6 Loading Rate Reduction Factors

In NCHRP 21-08, Innovative load Testing Systems, Paikowsky (2006) reported the correlation of 34 Statnamic rapid load test results with static load test results. The correlation database included driven H-piles, pipe piles and concrete piles as well as drilled shafts. Based on the correlation results, a loading rate reduction factor was recommended depending on the site soil or rock conditions. The loading rate reduction factor is to be applied to the derived static load-movement curve to account for overpredictions of the nominal resistance. Loading rate reduction factors of 0.96, 0.91, 0.69, and 0.65 were recommended for rock, sand, silt, and clay, respectively. Paikowsky recommended these loading rate reduction factors be applied to UPM, MUP, and SUP analyses of Statnamic test results.

Weaver and Rollins (2010) reviewed a different limited database of static load test and rapid load test results for drilled shafts with cohesive soils along the shaft and beneath the toe. That review indicated an average loading rate reduction factor of 0.47 with a range of plus or minus 0.14 when used with the UPM. While not measured on driven piles, the results suggest a lower loading rate reduction factor should be considered when evaluating rapid load test results in cohesive deposits. This is particularly true if the pile head displacement is less than the 5% of the equivalent pile diameter needed to achieve geotechnical failure.

Brown and Powell (2013) reviewed the correlation between published static load tests and rapid load test results evaluated by UPM analysis. Based on this review, they proposed the loading rate reduction factor, ξ, for UPM analysis be selected based on the liquid limit, LL, of the clay soil in accordance with Equation 11-10.

$$\xi = -0.0033\,\mathrm{LL} + 0.69 \qquad \text{Eq. 11-10}$$

Where: LL = Liquid Limit (%)

Their compilation of correlation results is presented in Figure 11-13 .

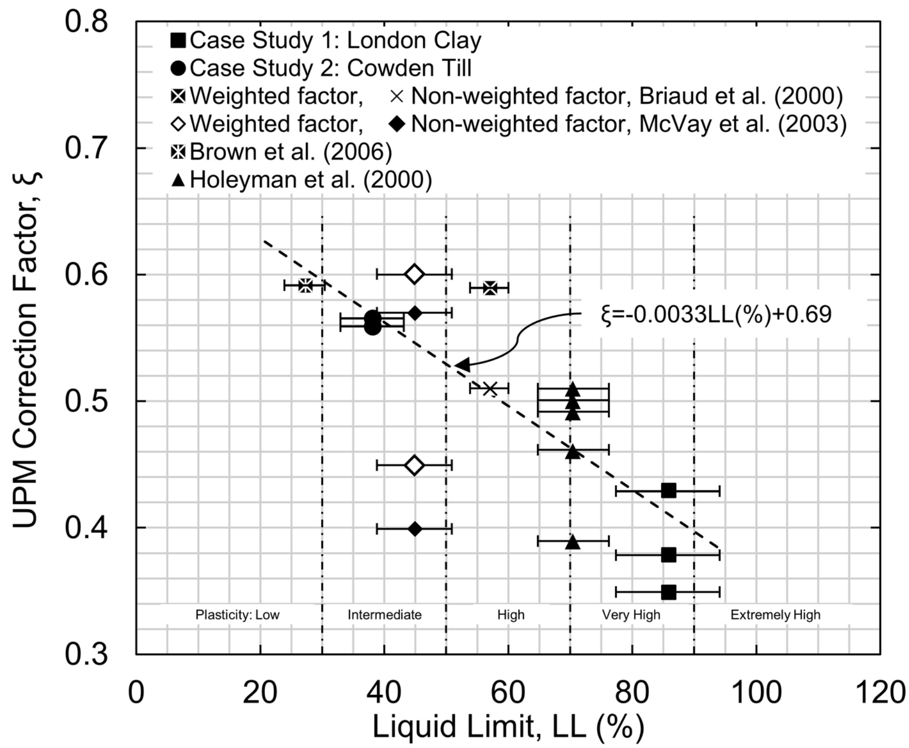

Figure 11-13 Variation of the UPM loading rate reduction factor versus the soil liquid limit (after Brown and Powell 2013).

Correlations of rapid load test results with cushioned drop weight systems using the UPM, MUP or SUP analysis procedure with static load test results on driven piles in clays have not been reported in the literature. However, UPM, MUP or SUP analyses from cushioned drop weight systems would be subject to similar loading rate considerations.

11.4.7 Resistance Factors

AASHTO (2014) does not address resistance factors for rapid load test results analyzed with any of the previously described interpretation methods. Some agencies have stated resistance factor guidance or have sponsored research efforts to determined resistance factors for nominal resistance determined by rapid load test methods.

For redundant piles, the South Carolina Department of Transportation's Geotechnical Design Manual, Version 1.1 (2010) suggests strength limit state resistance factors of 0.70 for a rapid load test result on a toe bearing pile in rock or very dense sand, and 0.65 for the nominal resistance determined by a rapid load test on a friction pile. For non-redundant piles, a reduced resistance factor of 0.55 is recommended for either end bearing or friction piles. These resistance factors for redundant and non-redundant piles are applicable to piles tested in axial compression with either the Statnamic rapid load test method or by high strain dynamic monitoring. The SCDOT manual notes that no increase in resistance factor is allowed on sites with multiple Statnamic tests unless the Statnamic rapid load test results are calibrated to a static load test.

A Florida Department of Transportation sponsored research project for determination of the resistance factor for rapid load tests was performed by McVay et al. (2003). The developed database was described as "small for statistical analysis purposes" and McVay calculated resistance factors using the FOSM approach to a reliability index, β, of 2.5. The calibration was performed using only the UPM analysis method with the loading rate reduction factors proposed by Mullins (2002), and summarized by Paikowsky (2006). The database used by McVay included 34 driven pile cases. Fifteen of the cases consisted of steel pipe piles ranging from 36 to 126 feet in length and 13 to 31 inches in diameter. Another 15 of the driven pile cases consisted of prestressed concrete piles ranging from 23 to 177 feet in length and 16 to 36 inches in diameter. The data sets were from project sites in the United States, Canada, and Japan.

Due to the limited size of the correlation database, McVay recommended a resistance factor of 0.70 for the nominal axial resistance determined by a rapid load test on redundant driven piles in rock and non-cohesive soils, and a resistance factor of 0.60 for redundant driven piles in sands-clays-rocks mixed layers. For non-redundant driven piles, McVay recommended resistance factors of 0.60 and 0.50 for these soils conditions, respectively. For driven piles embedded primarily in clays, the Statnamic rapid load test method was not recommended by McVay without a calibrated static load test. This reflects the variability of the loading rate reduction factors described in the previous section. It should also be noted these resistance factors were developed using the unloading point method (UPM) of analysis, and that modified or segmental unloading point methods were not used in the calibration.

Cushioned drop mass apparatus testing has very little written in terms of rapid load tests calibrated to static load tests. Presumably, if signal matching is used on the rapid load test results as calibrated for high strain dynamic testing, the resistance factor for high strain dynamic testing with signal matching could be used. In all cases, the designer must decide on appropriate resistance factors considering redundancy of the foundation elements, number of tests, and the required reliability index prior to proceeding with a rapid load test program.

11.5 LATERAL LOADING APPLICATION

The use of the Statnamic test for lateral load application was also studied in NCHRP Report 461, Static and Dynamic Lateral Loading of Pile Groups by Brown et al. (2001). A lateral Statnamic test on a nine pile group is shown in Figure 11-14. The maximum lateral load applied to date in a Statnamic test is 2,700 kips. However, this is not a limit of the Statnamic test device but rather of the pile group response.

Figure 11-14 Lateral Statnamic test on nine pile group (courtesy of Utah State University).

11.6 CASE HISTORY

In 2004, a Statnamic rapid load test was conducted on a 42 inch O.D. x 0.75 inch thick wall, open end pipe pile. This 111 foot long pile was driven into a mixed soil profile. The soil conditions at the site were generally described as 46 feet of clay with interbedded layers of sands and silts over the upper 46 feet. This layer was underlain by sands to a depth of 95 feet, and then interbedded layers of sands and clayey silts from 95 to 118 feet. The rapid load test was performed on the open end pipe pile approximately 7 months after installation. The load-displacement plot for this test is presented in Figure 11-15.

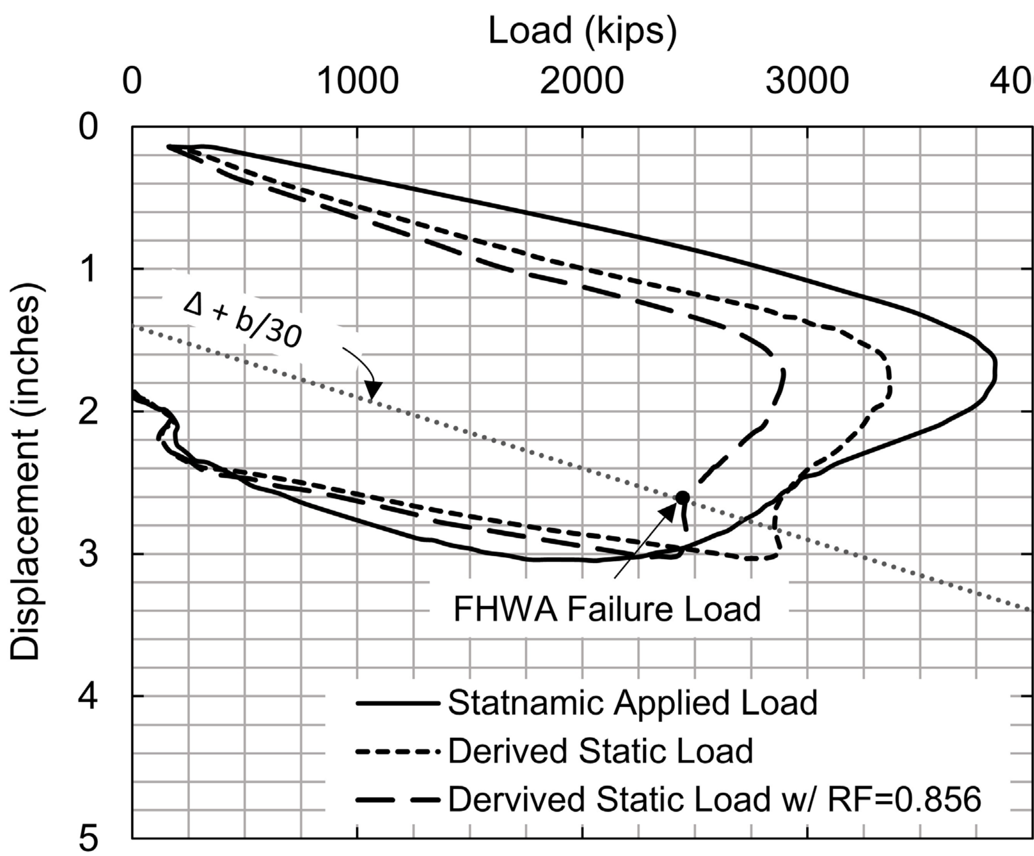

Figure 11-15 Statnamic test result (courtesy Minnesota DOT).

The Statnamic test apparatus had a maximum capacity of 4270 kips and applied a load of 3840 kips. The Modified Unloading Point (MUP) Method was used to evaluate the Statnamic test result. The pile was originally driven without anticipating that a Statnamic test would be later conducted. Therefore, the pile was not equipped with a pile toe accelerometer. The testing firm assumed that the pile toe acceleration was one half of the measured pile head acceleration, and the average of the measured and assumed acceleration was then used to conduct the MUP analysis.   A loading rate reduction factor of 0.856 (weighted for the site stratigraphy) was also applied to the test result. The Statnamic derived static load-displacement curve was then evaluated according to the FHWA recommended static load test interpretation criterion for large piles in use at that time. That criterion defined failure as the sum of the elastic deflection plus the ratio of the pile diameter over 30. The assigned failure load following this approach was 2,480 kips.

11.7 ADVANTAGES, DISADVANTAGES, AND LIMITATIONS

Advantages of rapid load testing relative to static testing include lower cost, shorter time required for test setup, and on-site mobility. The cost of a rapid load test can be on the order of one quarter to one half the cost of an equivalent static load test. The range in the cost of a rapid load test relative to a static load test depends upon the magnitude of the required load, the location of the site, and labor costs.

Cost savings may increase for rapid load tests when multiple tests are performed at the same site, or when rapid load tests are performed on piles having higher nominal resistances. Once a rapid load test apparatus is mobilized to a site, one or two rapid load tests can typically be performed in one day using the gravel catch structure. A greater number of rapid load tests per day can typically be performed using either the hydraulic catch device with the combustion gas and reaction mass apparatus or the cushioned drop weight systems.

The segmental reaction mass design for rapid load testing systems is advantageous as it 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.

Another advantage of a rapid load test is the mobilization of the soil resistance due to soil plugging in open ended sections. The slower loading rate in a rapid load test can be helpful in maintaining plugged behavior compared to a high strain dynamic test where the plug may slip under higher accelerations. Hence, plug behavior in a rapid load test may more closely resemble soil plug response in a static load test.

Rapid load tests do not require reaction piles. The smaller footprint associated with a rapid load test can be an advantage on sites with limited work areas.

Rapid load tests also have disadvantages. Rapid load test results must include both the influence of and correction for loading rate effects in all soil types. Correlations with conventional static tests are still being obtained to further address this issue as well as for calibration of rapid load test resistance factors.

The applied force pulse in a rapid load test necessary to mobilize the nominal resistance must be larger than the combined static and dynamic soil resistances. This disadvantage can be problematic in soils with high dynamic resistances. The pile in the presented rapid load test case history was driven into a mixed soil profile with low dynamic resistance based on the applied loading rate reduction factor of 0.856. The Statnamic test applied load in this soil was 55% greater than the derived nominal resistance. The Statnamic applied pile stress was 40.3 ksi, or roughly 90% of the guaranteed yield strength of the pile material. The applied stress would have been larger to achieve this same nominal resistance in a soil profile having higher dynamic resistance.

A maximum net pile head displacement of at least 3% of the pile diameter is required for full mobilization of the soil resistance with the UPM analysis technique to obtain geotechnical failure. Miyasaka et al. (2009) demonstrated that normalized pile head displacements less than 3% can result in overprediction of the nominal resistance.

Due to the limited number of rapid load tests systems and their locations, the mobilization cost for a rapid load test can be high. This is particularly true if the project requires only a single rapid load test, or if the test equipment must be mobilized from a distant location.

A limitation of a rapid load test can be instrumentation redundancy. Rapid load tests conducted without pile strain gage information lack the redundant check available in a conventional static load test (load cell and pressure gage) to check the calibration accuracy of the applied load.

11.8 PRACTICAL ISSUES AND CONSIDERATIONS

In a conventional static load test, a conservative estimate of the nominal geotechnical resistance is obtained when limited pile head movement occurs under the applied static loads. However, this is not necessarily the case in a rapid load test. When a rapid load test exhibits load-displacement behavior with a limited maximum displacement and permanent set, the nominal resistance may be either underpredicted or overpredicted. For piles driven into a dense granular deposit or on hard rock, a pile exhibiting a small permanent set likely indicates the nominal resistance has been underpredicted. However, in other soils and soft rock, the nominal resistance may be overpredicted as the force pulse is insufficient to cause geotechnical failure of the soil or soft rock and allow separation of the static and dynamic soil resistance effects. Rapid load tests in soils or soft rock should therefore have a permanent displacement of at least 3% of the pile diameter.

In layered soil profiles, a weighted loading rate reduction factor, ξ, must be determined and applied. The weighted loading rate reduction factor should be estimated based on the percentage of nominal resistance carried in a given layer and the appropriate loading rate reduction factor for the corresponding soil. Using the NCHRP 21-08 loading rate reduction factors described in Section 11.4.6, the loading rate reduction factor for a pile obtaining 50% of its nominal resistance in sand, $\xi = 0.91$, and 50% in clay, $\xi = 0.65$, would be 0.78. Similarly if 75% of the nominal resistance is obtained from the clay layer and 25% from the sand layer, the loading rate reduction factor would be 0.71.

In soils with high dynamic resistances, pile stresses should also be considered when selecting a rapid load test for nominal resistance verification.   Pile stresses should be kept below the driving stress limits of the strength limit state when performing a rapid load test.

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CHAPTER 12

WAVE EQUATION ANALYSIS

12.1 WAVE EQUATION ANALYSIS INTRODUCTION

As discussed in AASHTO (2014) Article C10.7.3.8.5 as well as in Chapter 13 of this manual, dynamic formulas, together with observed penetration resistances, do not yield acceptably accurate predictions of actual nominal pile resistances. 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 along with an associated computer program for the convenient calculation of the dynamic 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 determine driving stresses and 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, until 1986 (Goble and Rausche 1986). Later, additional options, improved data files, refined mathematical representations and modern user conveniences were added to this program on a proprietary basis, and the program is now known as GRLWEAP, (Pile Dynamics, Inc. 2010). Similar computer programs have been developed, such as PDPWAVE (Bielefeld and Middendorp 1992), that are based on the method of characteristics, a mathematical model that differs from Smith's lumped mass model.

The wave equation approach has been subjected to a number of checks and correlation studies. Studies on WEAP performance of have produced publications demonstrating that program's performance and utility (e.g., Blendy 1979; Soares et al. 1984; Rausche et al. 2004). Documentation of the most recent version of this program, GRLWEAP 2010, has been prepared by Pile Dynamics, Inc. (2010).

AASHTO (2014) design specifications recommend a resistance factor of 0.50 when wave equation analysis is used for the determination of the nominal resistance without the benefit of dynamic measurements or load test results but with field confirmation of hammer performance. The wave equation database used for LRFD calibration consisted of nominal resistance predictions determined using the GRLWEAP program and its associated pile and soil models.

This chapter explains what a wave equation analysis is, how it works, and what problems it can solve. Example problems, highlighting wave equation program applications, are demonstrated. Basic wave equation program input and output are presented for demonstration purposes using the GRLWEAP software. However, this should not be construed as a promotion or endorsement of this particular software product by the FHWA.

12.2 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 soil parameters for which standard recommendations are given. Thus, a wave equation program can be run with minimal specialized knowledge. However, interpretation of calculated results is facilitated, and errors in result application may be avoided, by knowledge of the mechanics of stress wave propagation and familiarity with the particular project’s design requirements and constraints.

In the first moment, after a hammer has struck the pile top, only the pile particles near the ram-pile interface are compressed. This compressed zone, or force pulse, as shown in Figure 12-1, 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 reflect from the pile toe either as a tension or a compression force pulse, which travels back to the pile head. Both incident and

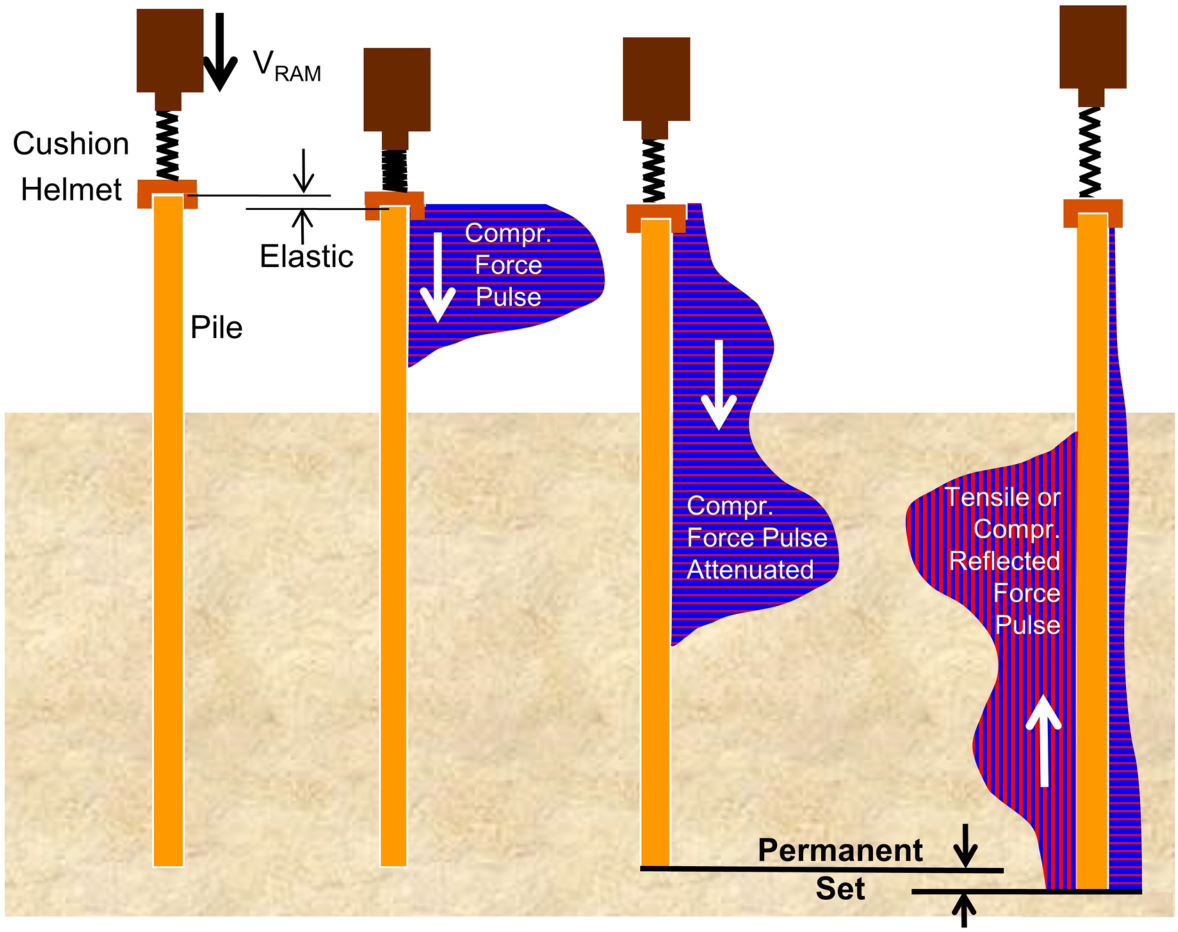


Figure 12-1 Wave propagation in a pile (adapted from Cheney and Chassie 2000).

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.

12.3 WAVE EQUATION METHODOLOGY

In a Smith-type 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 occasionally improve the accuracy of the numerical solution at the expense of longer computer run times (Rausche et al. 2004). Spring stiffness and mass values are calculated from the cross sectional area, modulus of elasticity, and specific weight of the corresponding pile section. Hammer and pile cushions are represented by additional springs whose stiffness 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 1.0 for a perfectly elastic collision which preserves all energy, and is equal to 0.0 for a perfectly plastic condition which loses all deformation energy. The usual condition of partially elastic collisions is modeled with an intermediate COR value. For example, the default value for the COR of wood is 0.50. However, when a softwood cushion is badly worn, a value of 0.25 may be more appropriate and can be entered into the wave equation input together with other cushion properties.

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 soil resistance force which is pile displacement related, and a dynamic soil resistance forces which is pile velocity related act on every embedded pile segment. The static soil resistance forces are modeled by elasto-plastic springs and the dynamic soil resistance by dashpots. The displacement at which the static soil resistance 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 temporary static soil resistance. Additional discussion on quake and damping factors is presented in Section 12.6.7. A schematic of the wave equation hammer-pile-soil model is presented in Figure 12-2.

As the analysis commences, a calculated or assumed nominal resistance (ultimate capacity, R_{ut}) from user specified values is distributed along the shaft and toe according to user input or program calculation among the elasto-plastic springs. 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 based on hammer efficiency and stroke inputs. 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 both static and dynamic 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 spring 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 variables, velocity and displacement, 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.

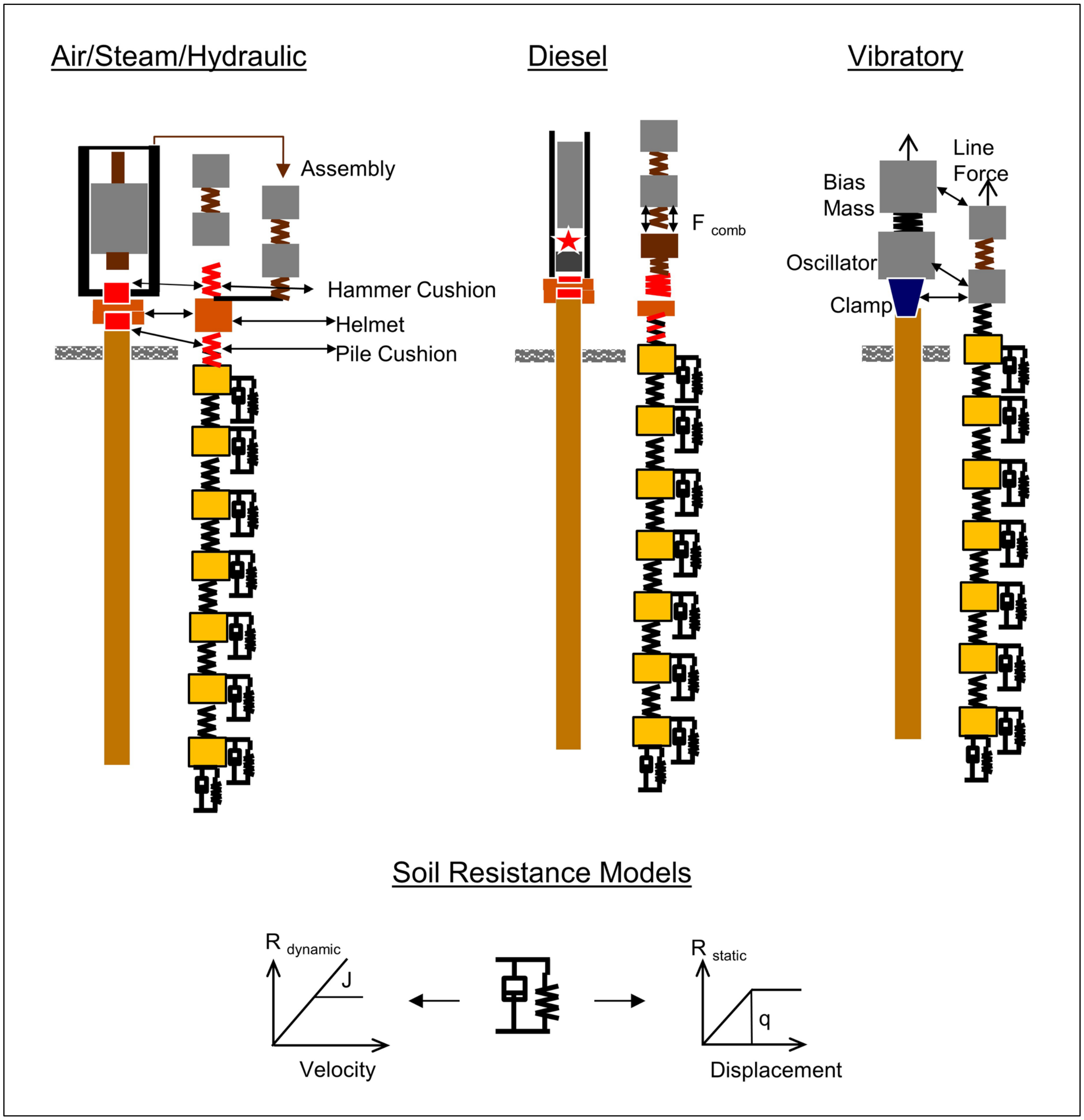


Figure 12-2 Typical wave equation pile and soil models.

The permanent set in inches of the pile toe is calculated by subtracting a weighted average of the shaft and toe quakes from the maximum pile toe displacement. An alternation calculation option available in the GRLWEAP program is called Residual Stress Analysis (RSA) which analyzes several hammer blows in sequence and then determines the permanent set from the pile top displacements of consecutive blows. However, to date, this more accurate approach has only been adopted for and checked against records from fluted and tapered piles. The inverse of the permanent set is the penetration resistance (blow count) in blows per foot or blows per inch that corresponds to the input nominal resistance. By performing wave equation analyses over a wide range of nominal resistances a curve or "bearing graph" can be plotted which relates nominal resistances to the pile penetration resistance or blow count.

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 maximum calculated compression and tension stresses.

In addition to the bearing graph, wave equation analysis can provide two alternative results, the constant capacity analysis, or "inspector's chart", and the “drivability analysis.” The inspector's chart establishes a relationship between variable hammer energy or stroke and pile penetration resistance for one particular, user specified, nominal resistance (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 penetration resistances and driving stress control. This analysis option is described in greater detail in Section 12.5.2.

The drivability analysis calculates penetration 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 nominal resistance 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 drivability option is described in greater detail in Section 12.5.3.

12.4 WAVE EQUATION APPLICATIONS

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

1. It establishes a relationship between nominal resistance and pile penetration resistance or blow count. From the user's input data of the resistance values along shaft and at the toe, the wave equation analysis estimates the permanent set in inches under one hammer blow. Specifying up to ten nominal resistance (ultimate capacity) values yields a relationship between nominal resistance and penetration resistance in blows per foot or blows per inch.

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

3. The analysis also relates hammer stroke or hammer energy to the pile penetration resistance for a given nominal resistance.

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

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 drivability to the required depth and/or nominal resistance.

For example, design considerations such as scour or consolidation settlements due to lower soft layers may make it necessary to drive a pile through a hard layer whose penetration 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 drivability, 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 factored load has to be chosen for drivability 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 nominal resistance requirements.

Suppose that it would be possible to drive a thinner walled pipe pile or lower weight H-pile section to the desired depth, but with excessive driving stresses. More cushioning or reduced hammer energy would lower the stresses, but would result in a refusal penetration resistance. Choosing a high strength steel grade for the pipe or H-section 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, factored 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 must be selected. These parameters 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 increased 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 penetration 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 the addition of hammer cushion material is relatively ineffective for limiting driving stresses.

Hammer size, energy setting, and cushion materials should always be chosen such that the maximum expected penetration resistance is less than 120 blows/ft. One exception to this upper limit is piles driven through soft soils that transition abruptly to hard rock. In this unique case, piles are often final seated on the hard rock at 5 blows per ¼ inch (240 blows/foot equivalent) and the use of a larger hammer may result in driving stress concerns.

For most situations where wave equation simulation is used for hammer selection, it is prudent to select a hammer that can achieve the required nominal resistance at a blow count not exceeding 100 blows/ft. The blow count should also be greater than 30 blows/t for reasonably accurate construction control. This is required, because (a) the relative error of an inaccurate blow count measurement is greater for lower penetration resistances and (b) the dynamic methods of nominal resistance assessment tend to over-predict when driving is very easy. Of course, adjustable hammers may be accepted based on their lower energy settings. Exceptions should also be made when the accuracy of the blow count measurement is irrelevant. Such situations arise when the pile has to be driven to depth at expected capacities above the required minimum or because a large component of the nominal resistance is derived from soil setup.

2. Contractors – to select an economical driving system to minimize installation cost.

While the construction engineer is interested in a safe pile installation method, contractors would like to minimize equipment size and driving time for cost considerations. 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.

For concrete piles, 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 result in a considerable cost.

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

12.5 WAVE EQUATION EXAMPLES

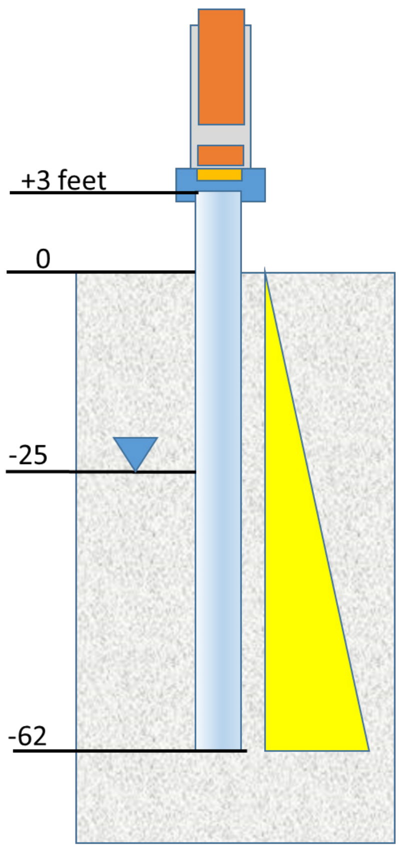
This section presents several examples that illustrate the application of the wave equation analysis for the solution of design and construction problems. The resistance factor applied to the nominal resistance in the following examples is 0.80 unless otherwise stated in the example. This assumes that a static pile load test was performed in addition to and to calibrate the wave equation result. As noted in Chapter 7, a resistance factor of 0.65 should be applied to the nominal resistance if dynamic testing with signal matching would be performed on the project instead of a static load test. If only wave equation analyses were used for resistance verification, then a resistance factor of 0.50 would be applicable. Furthermore, the nominal resistance times the resistance factor in a wave equation analysis should be greater than the sum of the critical factored loads plus resistances from any overlying layers unsuitable for or not present during long term support. Note that the nominal resistance (Chapter 7) corresponds to ultimate capacity in the literature. Hence, input or output for wave equation analysis programs often identify the nominal resistance as ultimate capacity or R_n.

12.5.1 Example 1 – General Bearing Graph

One of the primary applications of a wave equation analysis is to develop a bearing graph relating the nominal resistance to the pile penetration resistance or blow count. A bearing graph is often used to develop the driving criterion. For a desired nominal resistance, the required blow count can be obtained from the bearing graph.

Consider the soil profile in Figure 12-3. In this example, the foundation designer has estimated that a 65 foot long, 14 inch by 0.312 inch wall, closed end pipe pile with a steel yield strength of 35 ksi will be driven 62 feet into a deep deposit of medium sands. The foundation designer anticipates that the pile would achieve a nominal resistance of 350 kips at that depth. Using only wave equation analysis for the field verification method, the AASHTO resistance factor is 0.50. This would allow a maximum factored load of 0.50 times 350 kips or 175 kips. The designer also anticipated that the resistance on the shaft (pile toe) would be 52% (48%) of the total nominal resistance and that it would be triangularly distributed. Note that the calculated resistance is the expected long term nominal resistance and because the soil type is sand it can be expected that the resistance during driving will be essentially the same. Also, based on program recommendations for this displacement pile driven into a medium sand, the toe quake is input as the pile diameter divided by 60 (0.23 inches) while the shaft damping in these cohesionless soils is set to 0.05 s/ft. Shaft quake and toe damping are generally left at the defaults of 0.10 inches and 0.15 s/ft, respectively. These recommendations have, on the average, yielded satisfactory agreement with field observation and dynamic testing results. However, once piles have been driven at specific sites, load tests may indicate that different parameters may be more appropriate. Figure 12-3 summarizes the input quantities.

The contractor selected a Delmag D12-42 single acting diesel hammer for driving the pipe piles. The contractor's hammer submittal indicates that the hammer cushion will consist of 1 inch of aluminum and 1 inch of Conbest with a cross sectional area of 227 in^2 and an averaged (Conbest and aluminum) elastic modulus of 530 ksi. A helmet weight of 1.7 kips is reported.



Hammer	DELMAG D12-42
Rated energy	34 ft-kips
Hammer cushion	1 inch Conbest + 1 inch aluminum
	A = 227 in^2; E = 530 ksi; COR = 0.8
Helmet Weight	1.7 kips
Pile Size	14 in. O.D. x 0.312 in. wall
Pile Type	closed ended pipe
Pile Yield strength	35 ksi
Pile Length	65 ft
Penetration	62 ft
Soil Type	medium dense sand
Water table	25 ft below grade
Shaft/toe quakes	0.10 / 0.23 inch
Shaft/toe damping	0.05 / 0.15 s/ft
Shaft resistance	52% - triangular
Toe resistance	48%

Figure 12-3 Example 1 – Problem profile.

Based on this information, a wave equation analysis can be performed. The nominal resistance of 350 kips is input along with selected additional values for which the wave equation analysis calculates the net permanent displacement and, inversely, the penetration resistance in blows/foot. By plotting calculated penetration resistances versus the corresponding input nominal resistance values, a bearing graph is developed. The results of the example analysis are shown in Table 12-1 and in Figure 12-4.

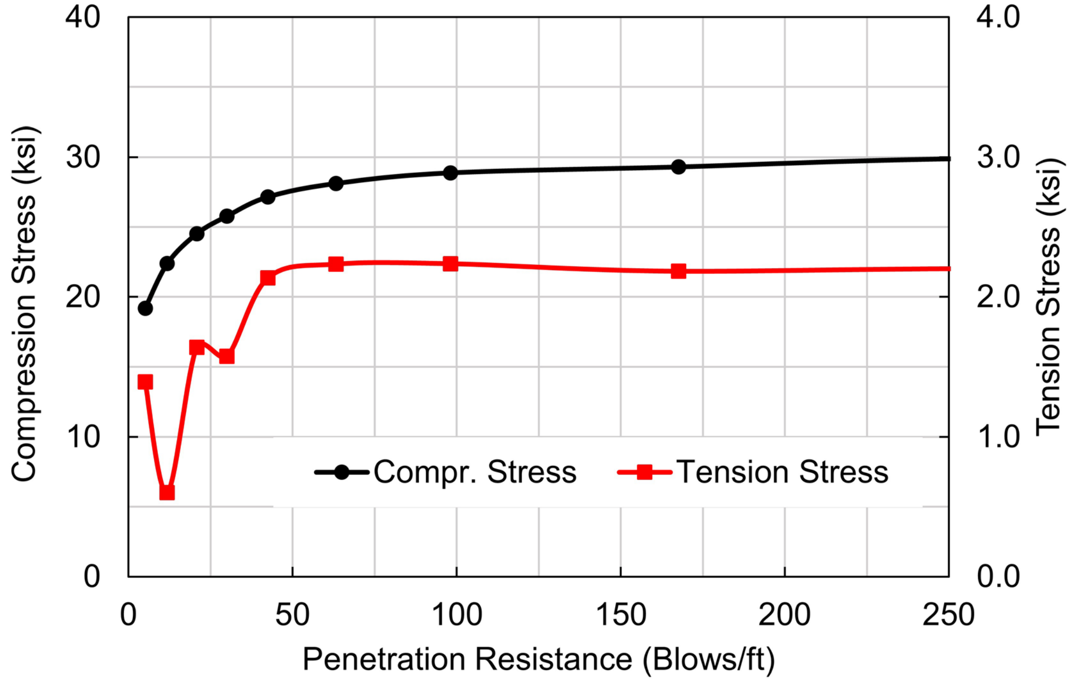
In the bottom half of the bearing graph, the nominal resistance versus penetration resistance in blows/foot is represented by the solid line. This graph shows that for a nominal resistance (ultimate capacity) of 350 kips, a penetration resistance of 99 blows/foot is required. This is less than the AASHTO recommended limit of 120 blows/foot. Although this is a relatively high blow count, it will neither require an excessive pile driving effort nor would it be too low to be inaccurate as far as construction control is concerned.

Table 12-1 Bearing Graph Numerical Results

Nominal Resistance kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Penetration Resistance (Blow Count) bl/ft	Hammer Stroke ft	Transferred Energy ft-kips
50	19.15	1.19	5.1	5.70	17.3
100	22.40	0.58	11.8	6.54	15.8
150	24.52	1.57	20.7	7.21	15.6
200	25.82	1.52	29.9	7.64	15.7
250	27.17	2.05	42.5	8.16	16.3
300	28.14	2.21	63.4	8.52	16.9
350	28.89	2.18	98.8	8.82	17.3
400	29.34	2.11	169.6	8.99	17.4
450	29.97	2.11	279.9	9.24	17.7
500	30.34	2.08	631.3	9.41	17.9

Also in the bottom half of the bearing graph, the corresponding hammer stroke versus penetration resistance is represented by the dashed line. This curve is important for variable stroke hammers as a check on hammer performance when the driving criterion is applied. In this case, the penetration resistance of 99 blows/foot for 350 kip nominal resistance is based upon the hammer operating at a hammer stroke of 8.8 feet. Should field observations indicate significantly (say more than 10% difference) higher or lower strokes, then a lower or higher penetration resistance would be necessary for the same nominal resistance, because the hammer force and/or energy would be higher or lower. Hammer stroke information is therefore essential for field evaluation and control of the pile installation process.

An inspector's chart analysis (see Example 2) provides this information determining required blow count as a function of hammer stroke. For significantly differing hammer field performance, a new wave equation analyses would be necessary with a modified maximum combustion pressure or a fixed input stroke.



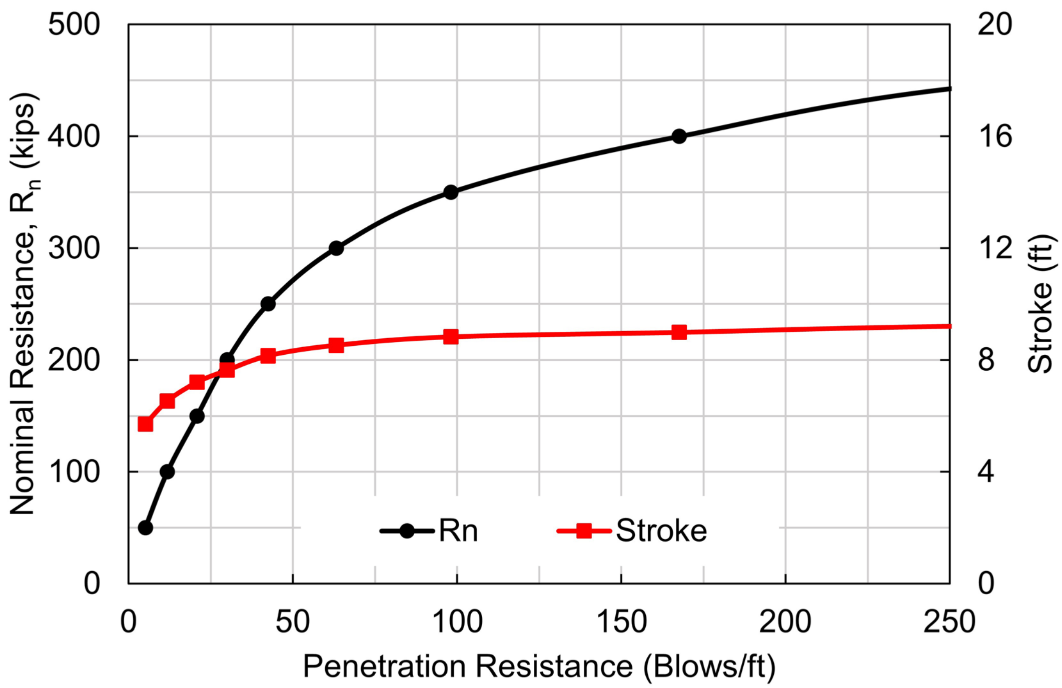


Figure 12-4 Typical wave equation bearing graph. Top plot – driving stresses; bottom plot – nominal resistance and diesel hammer stroke.

In the upper half of Figure 12-4, maximum compression and tension driving stresses are plotted as a function of penetration resistance. Of primary interest for a steel pile is the compression driving stress, which is represented by the solid black line. This plot shows that, at 99 blows/foot (and at the required nominal resistance), the maximum compression stress calculated in the pile is almost 29 ksi. This is less than 90% of the yield strength of 35 ksi or 31.5 ksi. This compression stress level is acceptable according to Article 10.7.8 of AASHTO (2014) design specifications and as discussed further in Chapter 8. However, any non-uniform stress components (e.g. bending caused by poor hammer-pile alignment or pile-toe contact with sloping rock) are not included in the wave equation results and would be additional. In any case, the 90% yield limit applies to the stress averaged over the pile cross section.

Though the analysis was conducted for an estimated penetration of 62 feet, in the field the required penetration resistance may be reached at a lesser or greater depth. 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 required blow count is already reached at 50 feet of penetration), an additional analysis should be performed to better match field observations. In general, the nominal resistance versus pile penetration resistance relationship is relatively insensitive to changes in the penetration depth and, therefore, to the distribution of the resistance along the pile unless there is a significant change in the soil profile. Of course, if unexpected changes in hammer performance, driving system components, or soil properties appear to cause the difference, then it would be prudent to check the equipment performance and soil resistance by dynamic measurements. Higher penetration resistances from penetrating embankment fills or scour susceptible material, etc., should also be considered in this assessment.

12.5.2 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 penetration resistance required for the 350-kip nominal resistance is evaluated over a range of hammer strokes. The resulting inspector's chart would be helpful for field personnel in determining when pile driving can be terminated if the field observed hammer stroke varies from that originally predicted by the wave equation bearing graph analysis. Figure 12-5 shows the resulting inspector's chart. The lower half of Figure 12-5 presents the hammer stroke versus penetration resistance plot for the required nominal resistance of 350 kips. Where the point of intersection of the observed stroke and penetration resistance plots below the curve, the nominal resistance has not been obtained. Any combination of stroke and penetration resistance plotting above the curve indicates that the required resistance level has been reached. For example, any stroke greater than 8.2 feet at a penetration resistance of 90 blows/foot is acceptable. The upper half of either graph shows the stress maxima associated with a particular driving resistance. Hence, the inspector's chart analysis aids the inspection personnel in field control.

12.5.3 Example 3 – Drivability Studies

Scour and seismic design considerations often result in increased pile penetration requirements. Therefore, the ability of a given pile to be driven to the required penetration depth should be evaluated in the design stage in a wave equation drivability study, as presented in this example.

Figure 12-6 illustrates the installation conditions at an interior bridge pier in a river. A cofferdam will be required for pier construction. The interior of the cofferdam will be excavated 16.5 feet below riverbed prior to pile installation. The excavated layer consists of loose silt. Below the silt, a 13.5 foot thick layer of extremely dense sand and gravel layer with some clay was noted which in turn overlies a medium dense sand layer. Bedrock was encountered approximately 70 feet below riverbed.

The contractor has selected a Conmaco C 160 air hammer which has a ram weight of 16.25 kips, a rated stroke of 3.0 feet for a rated energy of 48.75 ft-kips. The hammer cushion consists of Nylon, the helmet weight is 4.07 kips and the pile cushion was chosen as 10 inches of plywood. A new pile cushion is to be used for every pile. For that reason the elastic modulus of 30 ksi for "new plywood" is used at

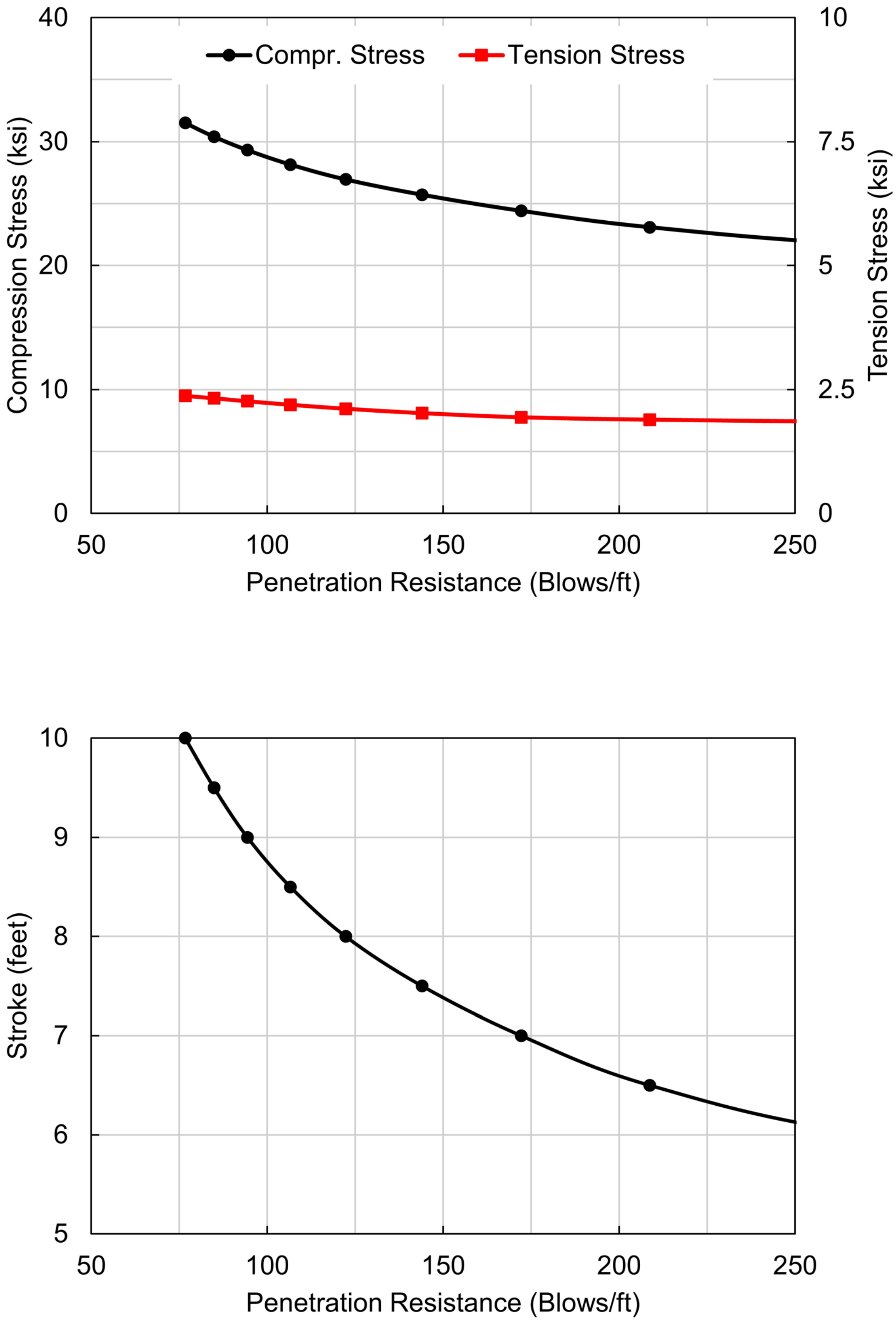


Figure 12-5 Constant capacity analysis (inspector's chart).

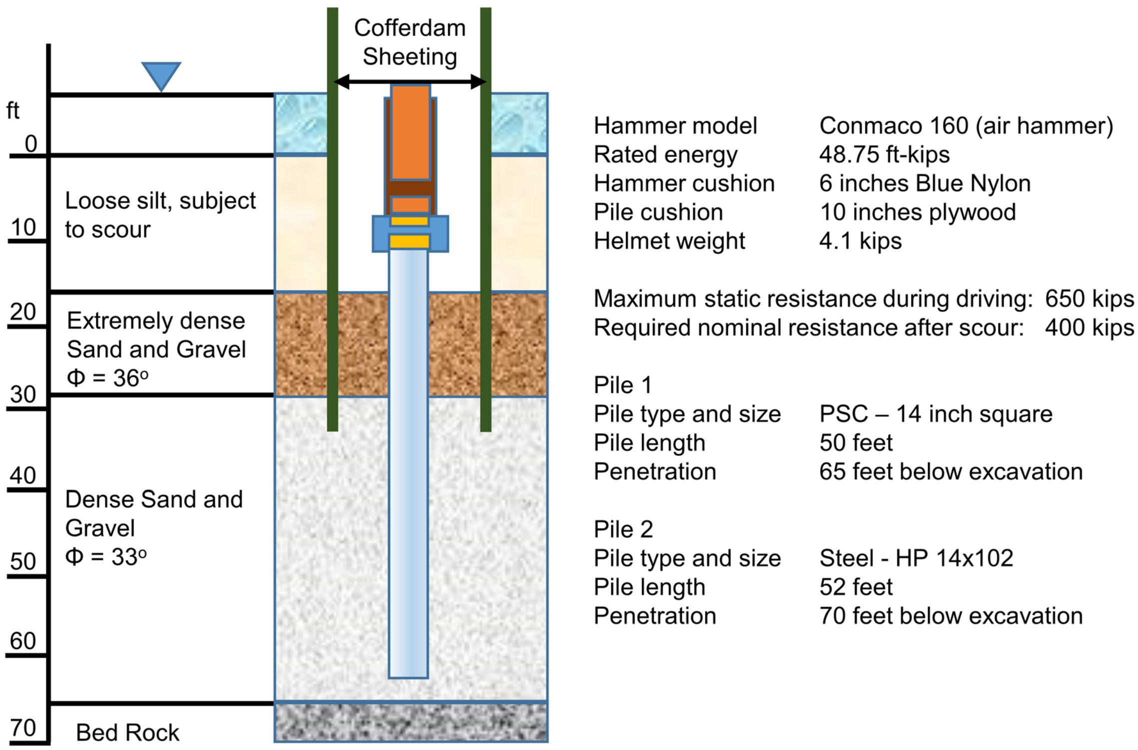


Figure 12-6 Example 3 – Problem profile.

the beginning of driving. However, as the plywood compresses during driving and gets harder, a "used plywood cushion" elastic modulus of 75 ksi should be chosen to reflect the associated higher stiffness of the harder cushion at end of driving. This recommendation is based on correlation studies (Rausche et al. 2004) which indicated that the stiffness of plywood cushions at the end of driving are typically 2.5 times higher than in their new condition (for oak boards the factor is 1.5). Note that for the correctly effecting the stiffness increase, only the elastic modulus should be modified while the cushion thickness is always entered as the nominal thickness in the "new" cushion conditions.

The extremely dense sand and gravel layer was estimated to have a soil friction angle $\phi = 43^{\circ}$. This friction angle was used in the static calculations of toe resistance. However, the friction angle was limited to 36° for the hard angular gravel when calculating shaft resistance. The friction angle for the medium dense sand layer was estimated to be $\phi = 33^{\circ}$.

During construction, the silt soils will be removed from within the cofferdam area. However, 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 with consideration for the overburden pressure from these materials. However, hydraulic experts predict that the 16.5 feet of loose silt may erode completely due to channel degradation scour. Thus, for long term pile nominal resistance, static calculations should ignore the effective weight of the silt layer. As a result, a higher soil resistance than required to meet the static load requirements must be anticipated during pile installation.

Initial static analysis indicates that a 14 inch square prestressed concrete pile would develop the required nominal resistance of 400 kips, primarily through end bearing, at a depth of 10 feet below the cofferdam excavation level (26.5 feet below river bed). However, when considering the reduction in the effective overburden pressure from the scouring of the silt layer, the pile would have an nominal resistance of only 241 kips at a penetration depth of 10 feet below cofferdam excavation level and 318 kips immediately above the dense sand where the end bearing would be unreliable. Additional static nominal resistance calculations were performed at increased pile penetration depths for the pre-scour profile. These analyses show that when punching through the upper, extremely dense sand layer the nominal resistance would at first be lower in the dense sand and gravel but would again reach a 400 kip nominal resistance at a depth of 49 feet below cofferdam excavation level (65.5 feet below river bed).

For the installation condition, a pre-scour analysis indicates that the nominal resistance at a depth of 49 feet would be 472 kips and this is the soil resistance that must be overcome at the end of driving. (An almost identical driving resistance exists at a depth of 13 feet depth below excavation (29.5 feet below river bed)).

Next, nominal unit resistance values for both shaft and toe resistance versus depth with consideration of the silt overburden stress were calculated by static analysis and input into a wave equation program. The soil profile consists primarily of sandy materials. Significant soil setup or relaxation in these materials is not considered likely and therefore no gain or loss factors due to driving had to be considered in the drivability analysis (gain/loss factors for both shaft and toe were set to 1.0). Since the study is conducted in the design stage, the use of a locally available single acting air hammer driving system was assumed.

The following additional input considerations should be mentioned:

1. The concrete pile is a displacement pile. For that reason, the toe quake for the extremely dense sand and gravel was set to the pile width, D, divided by 120 (0.12 inches).

2. For the dense sand the toe quake was set to D/60 – 0.23 inches.

3. The pile cushion becomes stiffer (elastic modulus increases and thickness decreases) during pile installation. GRLWEAP Help (Rausche et al. 2004), recommends using a pile cushion stiffness for the end of driving which is 2.5 times the initial stiffness which is based on an elastic modulus of 30 ksi. This correction factor includes both the effect of the cushion reduction in thickness as well as the increase in elastic modulus. Thus for the various depths analyzed, the cushion stiffness was gradually increased between the initial and final driving depths.

The drivability analysis result (Figure 12-7) indicated that the 14 inch concrete pile would encounter a maximum penetration resistance of 158 blows/foot in the upper extremely dense sand and gravel deposit just before breaking through to the dense sand layer. For the final depth of 49 feet (65.5 feet below river bed) a penetration resistance of 60 blows/foot was calculated. Since the high blow count in the extremely dense sand layer is only present for a short distance, it could be concluded that the 14 inch concrete pile could be driven to the required penetration depth of 49 feet. This might be an erroneous conclusion. Although the static analysis would likely provide an adequate assessment of soil resistance for the first pile driven, an increase in the friction angle from group densification could significantly affect the resistance to driving of additional displacement piles, particularly within the tight confinement of the cofferdam. Also, dense deposits tend to develop negative pore pressures during shear, resulting in temporary increases in soil resistance which may later dissipate. If it is assumed that these factors cause a 30% increase in both shaft and toe resistances during the driving of subsequent piles, a second drivability analysis for the densified condition would indicate that the later piles essentially refuse (blow count greater than 240 blows/foot) at a depth of 11 feet below excavation (27.5 feet below river bed). The nominal resistance at that depth is 579 kips. Maximum calculated driving stresses were 3.7 ksi discouraging the use of a larger hammer, unless a sufficiently high concrete strength would be chosen. If displacement piles were indeed used, predrilling or jetting would likely be required to advance the piles through the upper stratum.

A low displacement pile such as an H-pile or open end pipe pile, which would cause a lower or no densification, presents a more attractive foundation solution. Thus, the analysis was repeated for a 55 feet long, HP 14x102 H-pile which would allow the pile to reach bedrock at 53.5 feet depth (70 feet below river bed). Note that it is common practice to analyze H-pile drivability with a toe resistance as though the H-pile would be plugging while at the same time assuming a toe quake of only 0.10 inches as for a non-displacement pile. In other words it is tacitly assumed that the pile is only partially plugging or that the plug slips during driving. This also means that there is no major densification effect when driving the H-pile.

The drivability results, shown in Figure 12-7, include the pile penetration resistance, maximum compression stresses and transferred energy for the normal (first pile) and later (densified) conditions. The concrete pile results are also shown in Figure 12-8. Along with the wave equation drivability results for the low displacement HP 14x102 H-pile. Figure 12-8 allows for a comparison of penetration resistance and transferred energy and shows that the low displacement steel pile will drive with a significantly lower penetration resistance though the extremely dense sand and gravel layer.

Note that the penetration depth in these figures corresponds to the depth below cofferdam excavation level. The maximum penetration resistance calculated for the H-pile to penetrate the extremely dense sand and gravel stratum is only 45 blows blows/foot. Corresponding compression driving stresses do not exceed 32 ksi and are within driving stress limit. The nominal resistance increases significantly at deeper pile penetration depths. However, the penetration resistance increases to only 44 blows/foot before quickly transitioning to refusal conditions when the pile reaches rock. The results indicate that the H-pile could be driven to bedrock allowing for a possible higher factored load, reduced number of piles and a more cost effective design.

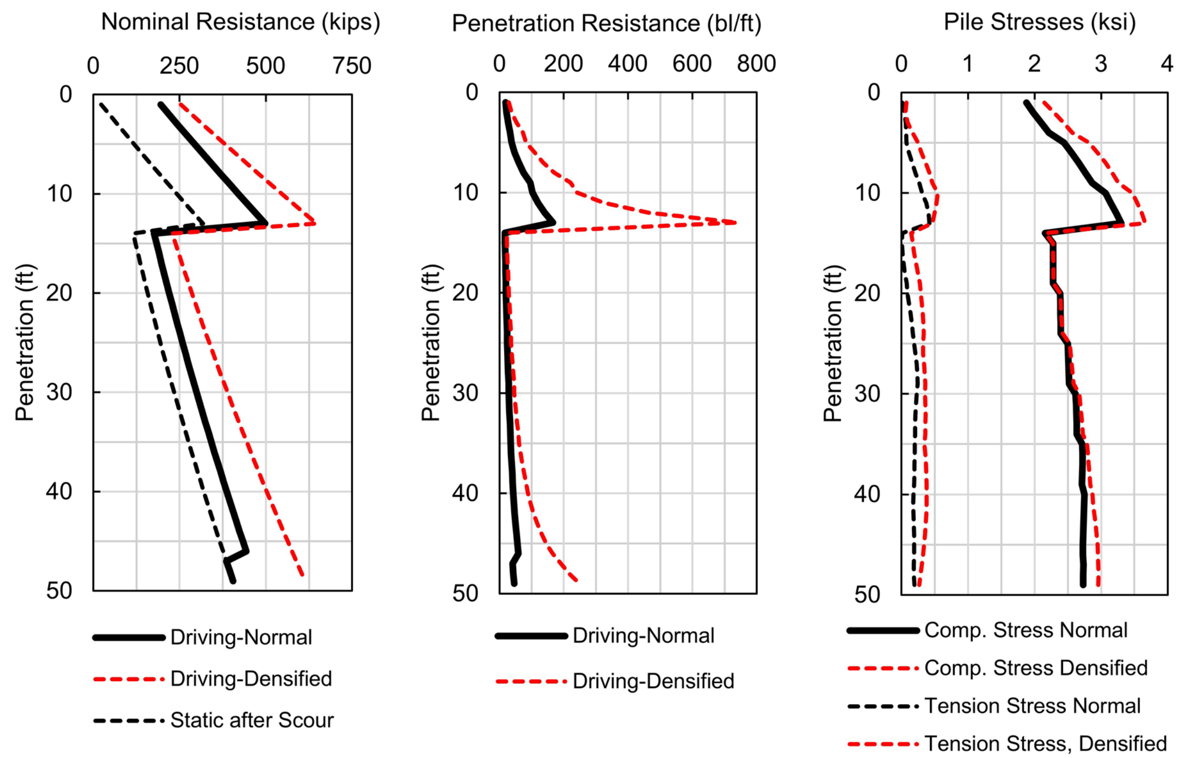


Figure 12-7 Nominal resistance, calculated blow count and stresses for before and after densification and nominal resistance for the after scour condition.

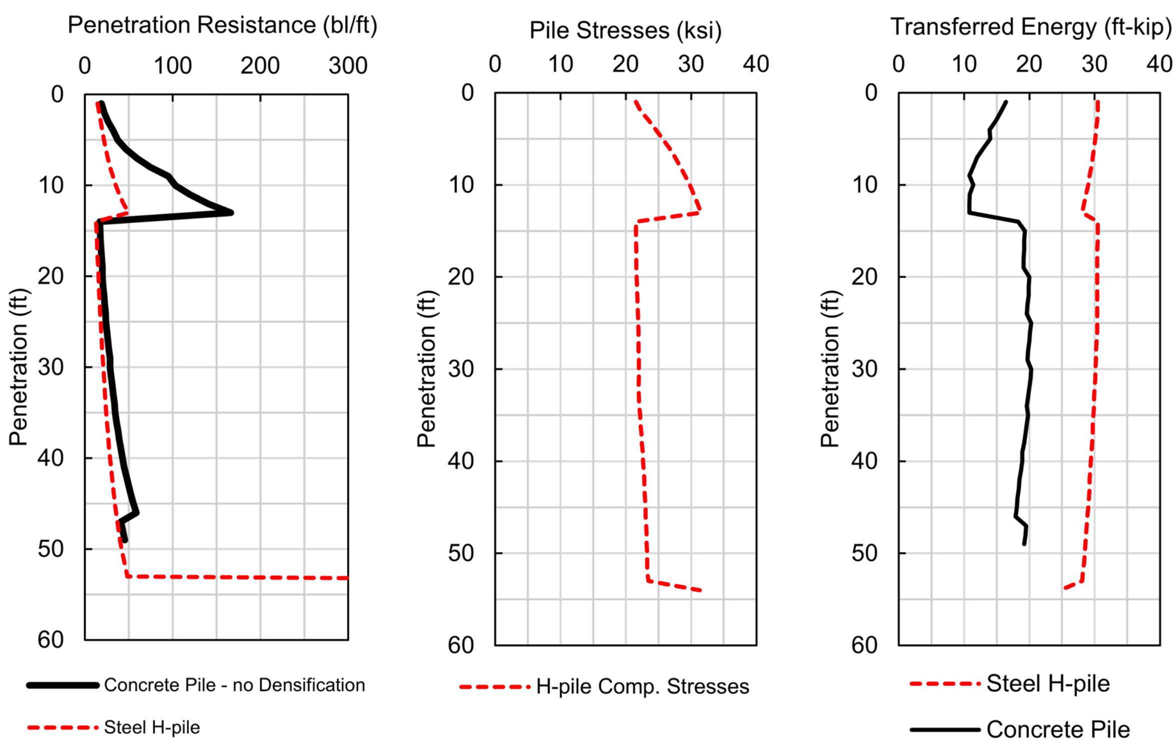
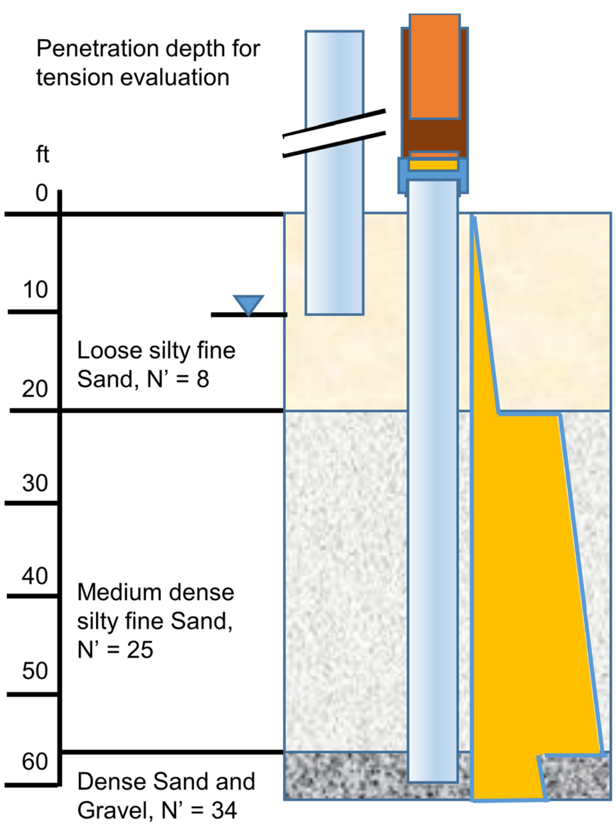


Figure 12-8 Drivability results for H-pile with corresponding concrete pile results.

12.5.4 Example 4 – Tension and Compression Stress Control

Example 4 illustrates the use of the wave equation for the control of tension stresses in a 60 feet long concrete pile. Static calculations indicate a 14-inch square, prestressed concrete pile driven through 20 feet of loose silty fine sand, 35 feet of medium dense sandy silt, and 3 feet into a dense sand and gravel deposit could develop a required nominal resistance of 400 kips. The static analysis also indicates that the nominal resistance is distributed as 55% shaft resistance and 45% toe resistance with a variable shaft resistance distribution along the pile shaft as shown in Figure 12-9.

The contractor selected a Junttan HHK 3A hydraulic hammer for driving the prestressed concrete piles. This hammer has a ram weight of 6.6 kips and a rated energy of 26 ft-kips. The contractor's hammer submittal indicates that the hammer cushion will consist of 8 inches of a material that has an elastic modulus of 360 ksi and a coefficient of restitution of 0.90 and a cross sectional area of 250 in^2. A helmet weighing 1.0 kip is also planned for the driving system.



Hammer model:	Junttan HHK 3A
Hammer cushion thickness:	8 inches
Hammer cushion modulus:	360 ksi
Helmet weight:	1.01 kips
Pile cushion	3 inch (plywood)
Pile type	PSC
Pile size	14 inch square
Pile length	60 ft
Final pile penetration	58 ft
Nominal resistance	400 kips
Nominal shaft resistance	55% (220 kips)
Nominal toe resistance	45% (180 kips)

Figure 12-9 Example 4 – Problem profile.

The concrete pile will have a compression strength of 5.5 ksi and an effective prestress after losses of 0.70 ksi. Using the AASHTO (2014) driving stress recommendations, discussed in Chapter 8, results in maximum recommended driving stresses of 4.1 ksi in compression and 0.92 ksi in tension.

One of the main concerns with the drivability of 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, an evaluation of tension stresses during easy driving is presented. The weight of the pile and driving system is anticipated to be on the order of 20 kips. 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, or approximately 10 feet. At this depth, the pile is still within the loose silty fine sand stratum and tension driving stresses are anticipated to be near their peak. Although not strictly correct, for the first low resistance analysis of 20 kips it is accurate enough to assume the same shaft resistance percentage (55%) as for the final penetration of 58 feet. For a complete bearing graph not only the 20-kip nominal resistance, but also other higher values are input and analyzed.

The contractor submitted a plywood pile cushion design comprised of four 3/4 inch sheets with a total thickness of 3 inches. Pile cushion stiffness significantly affects tension driving stresses. Therefore, it is necessary to determine whether or not the contractor's proposed pile cushion thickness is sufficient to maintain tension stress levels below specified limits. In the first trial, the 3 inch pile cushion is assumed to possess the properties of new plywood. Thus, the original pile cushion thickness of 3 inches and the new cushion elastic modulus of 30 ksi are input. Based on this information, the wave equation analysis indicates for the 20-kip nominal resistance a maximum tension stress of 1.47 ksi. The magnitude of the calculated tension stress exceeds the allowable driving stress limitation of 0.92 ksi.

A second wave equation analysis was therefore performed with an increased pile cushion thickness of 6 inches. By using the thicker pile cushion, the maximum tension stress was reduced from 1.47 ksi to 0.89 ksi which was less than the specified driving stress limit. The original and second wave equation analysis results for the easy driving condition at a pile penetration depth of 10 feet are presented in Figure 12-10.

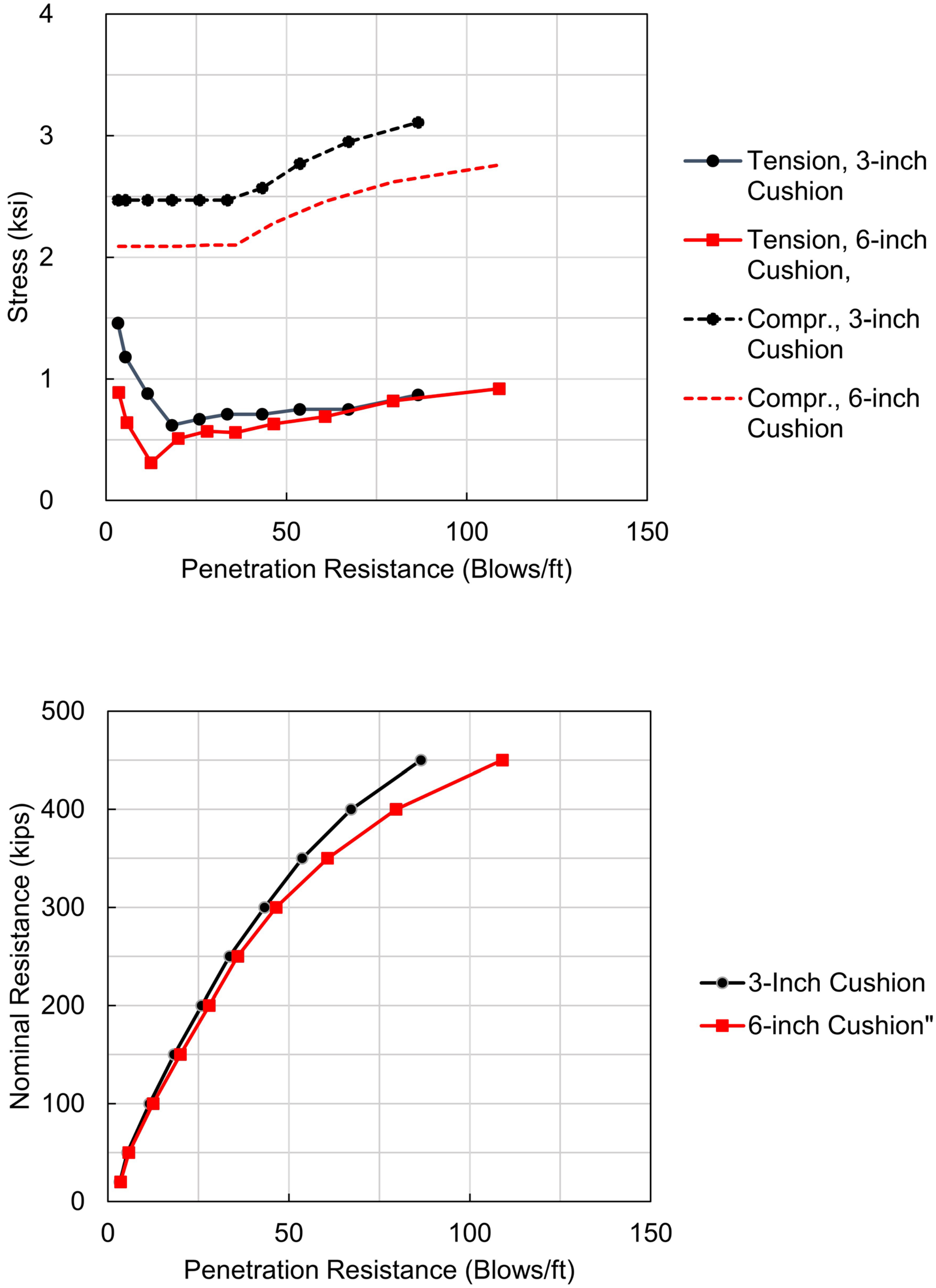


Figure 12-10 Bearing graphs for easy driving condition, two pile cushion thicknesses.

The Junttan HHK 3A hydraulic hammer has an adjustable stroke height (or energy level). In the previous two analyses, it was assumed that the hammer would be run at the maximum rated energy of 26 ft-kips which corresponds to an equivalent stroke of 3.94 feet. To reduce pile cushion costs, the contractor suggested piles be driven with a reduced stroke and thinner less costly pile cushion. The wave equation analysis was therefore repeated for a 3 inch cushion and a 1.75 foot hammer stroke, corresponding to a potential energy of 11.6 ft-kips. For this configuration, the calculated maximum tension stress was 0.90 ksi at the 20-kip nominal resistance.

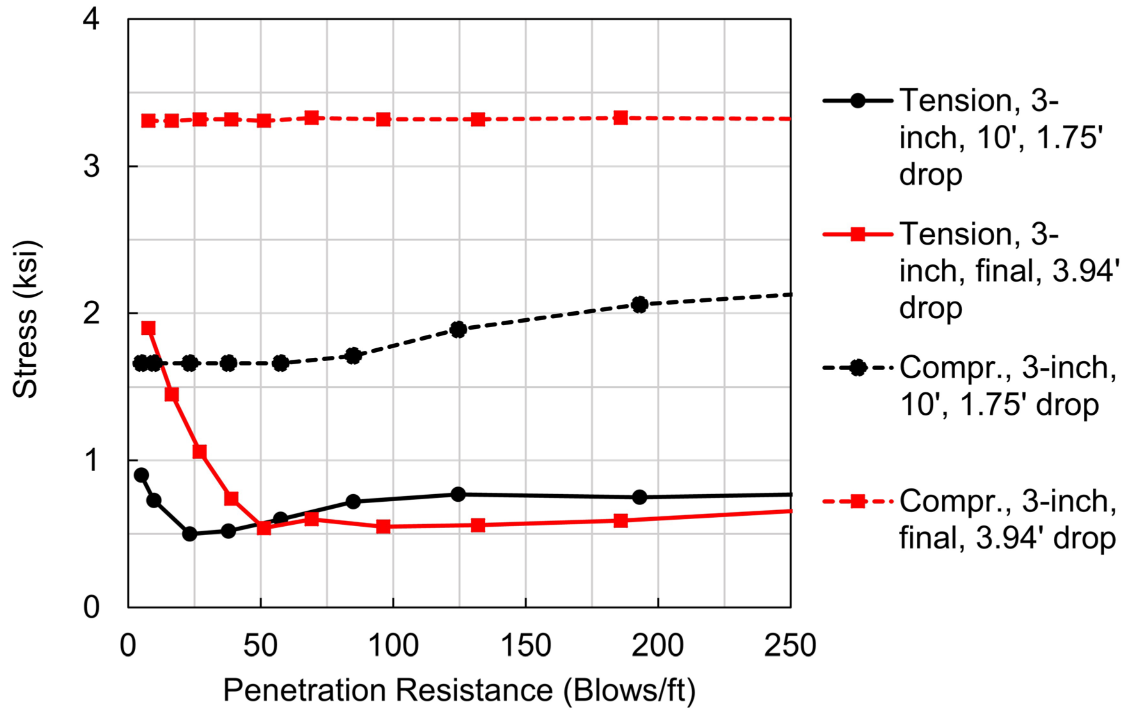
If the hammer would only be operated at the reduced 1.75 foot stroke, then the blow count would be at refusal (greater than 240 blow/foot) for the required 400 kip nominal resistance. Therefore, an additional analysis was performed for the final penetration depth of 58 feet using the full hammer stroke and rated energy and a used 3 inch thick pile cushion. The cushion was modeled with an elastic modulus of 75 ksi which considers that the cushion thickness is also reduced. Obviously, in this case it is not necessary to analyze the 20 kip nominal resistance value.

The wave equation analysis results for the final driving condition are shown in Figure 12-11. They indicate a penetration resistance of 56 blows/foot for the 400 kip required nominal resistance. The associated tension and compression stresses are 0.58 and 3.32 ksi which are less than the maximum recommended driving stresses of 0.92 and 4.15 ksi, respectively.

Figure 12-11 also indicates that the tension stresses for the full stroke driving would be excessive, greater than 0.92 ksi, if full stroke driving were used with the 3-inch cushion at a nominal resistance of 250 kips or less. For this resistance value, on the other hand, the reduced stroke analysis indicates a penetration resistance of 85 blows/foot. Based on these findings the following recommendation would be made to the contractor:

- Use a fresh 3-inch thick plywood cushion for every pile.
- Operate the hammer at a reduced equivalent stroke of 1.75 ft.
- Once the penetration resistance reaches 85 blow/foot, increase the equivalent stroke to the full rated stroke of 3.94 feet.
- Drive the pile to a minimum driving resistance of 56 blows/foot at the full rated stroke condition.

Note that it may be quicker, simpler, and possibly more cost effective to use the 6 inch pile cushion and full hammer energy for the complete driving sequence.



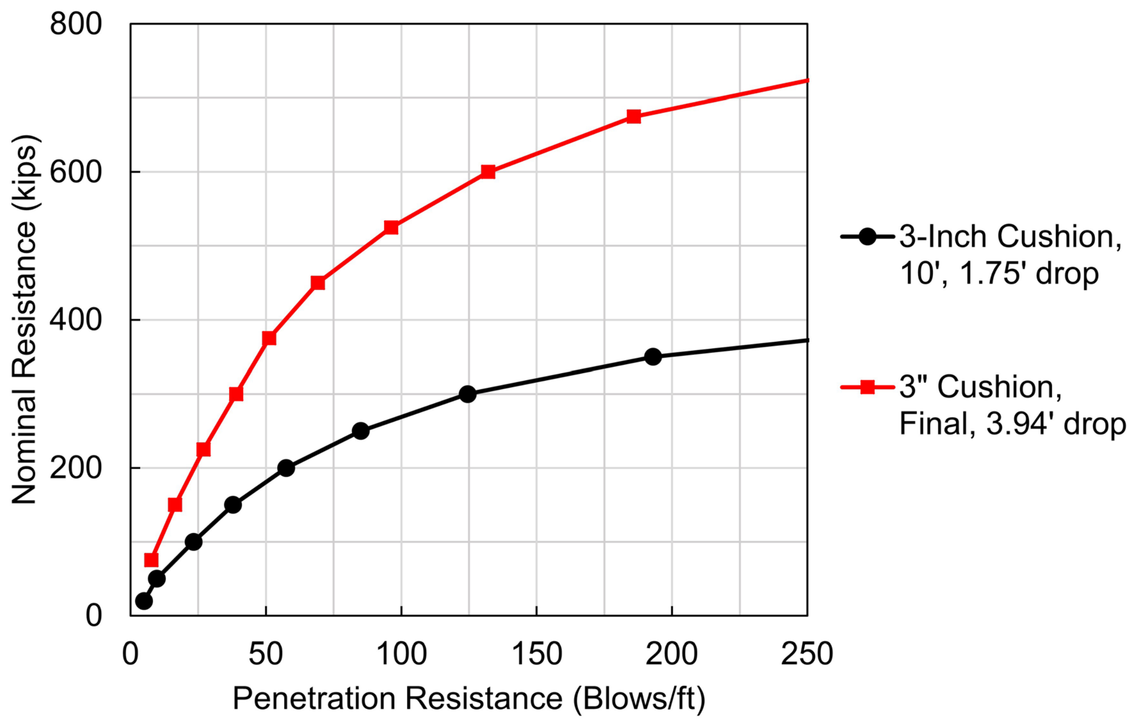


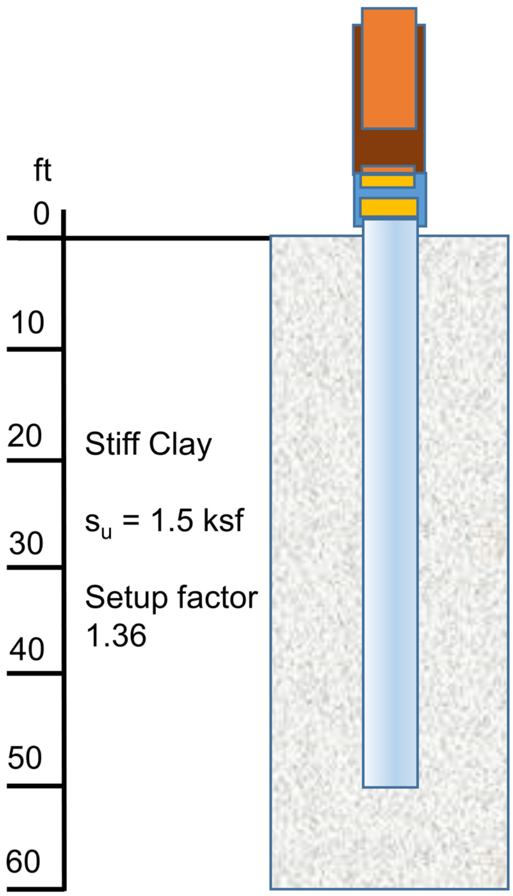
Figure 12-11 Bearing graphs for early low energy and end of driving high energy driving condition.

12.5.5 Example 5 – Use of Soil Setup

Consider the soil profile in Figure 12-12. In this example, a 12 inch square, prestressed concrete pile is to be driven into a thick deposit of stiff clay. The stiff clay has an average shear strength of 1.5 ksf. Based on field vane shear tests, it is estimated that the remolded shear strength at the time of driving will be 1.1 ksf, resulting in an expected soil setup factor of 1.36. A static analysis indicates that a nominal resistance of 300 kips after setup can be obtained for the proposed pile type at a penetration depth of 50 feet. The static analysis also indicates that the nominal resistance is distributed as 92% uniform shaft resistance and 8% toe resistance.

The contractor selected a Vulcan 08 single acting air hammer for driving the prestressed concrete piles. The contractor's hammer submittal indicates that the hammer cushion will consist of 8.5 inch of Hammortex with a cross sectional area of 148 in^2. The pile cushion will consist of plywood with a total thickness of 6 inches. It is anticipated that the pile cushion will compress and stiffen during driving similar to that described in Example 4. For an easy driving analysis, the assumption of a new pile cushion with the elastic modulus of 30 ksi would apply. For the late driving scenario, a compressed cushion modulus of 75 ksi should be considered. As mentioned earlier, using this 2.5 times higher modulus will not only account for the stiffening of the material but also for its thickness decrease. The contractor's submittal indicates that the helmet weighs 2.6 kips.

Based upon the reported soil type and setup behavior, a 36% increase in nominal resistance with time is expected at this site. Therefore, piles could be driven to a reduced nominal resistance, or static resistance to driving (SRD), of 225 kips instead of the required value of 300 kips with the remaining 75 kips expected from soil setup. As noted earlier in this chapter, a static load test will be performed on the project to confirm the expected resistance gain.



Hammer model:	Vulcan 08
Hammer cushion thickn:	8.5 inches
Hammer cushion type:	Hamortex
Helmet weight:	2.6 kips
Pile cushion thickness	6 inches
Pile cushion type:	Plywood
Pile type	PSC concrete
Pile size	14 inch square
Pile length	52 ft
Final pile penetration	50 ft
Nominal resistance	300 kips
Nominal shaft resistance	92% (276 kips)
Nominal toe resistance	8% (24 kips)

Figure 12-12 Example 5 –- Problem profile.

The wave equation results presented in Figure 12-13 indicate a final penetration resistance of 43 blows/foot could be used as the driving criteria for a 225 kip SRD or end of driving nominal resistance. This is significantly less than the 89 blows/foot required for a nominal resistance of 300 kips. Hence, significant pile length and driving effort may be saved by driving the piles to the lower resistance. However, this approach requires a restrike test or a static load test sometime after pile installation; the waiting time period is soil type dependent as discussed in Section 7.2.4.

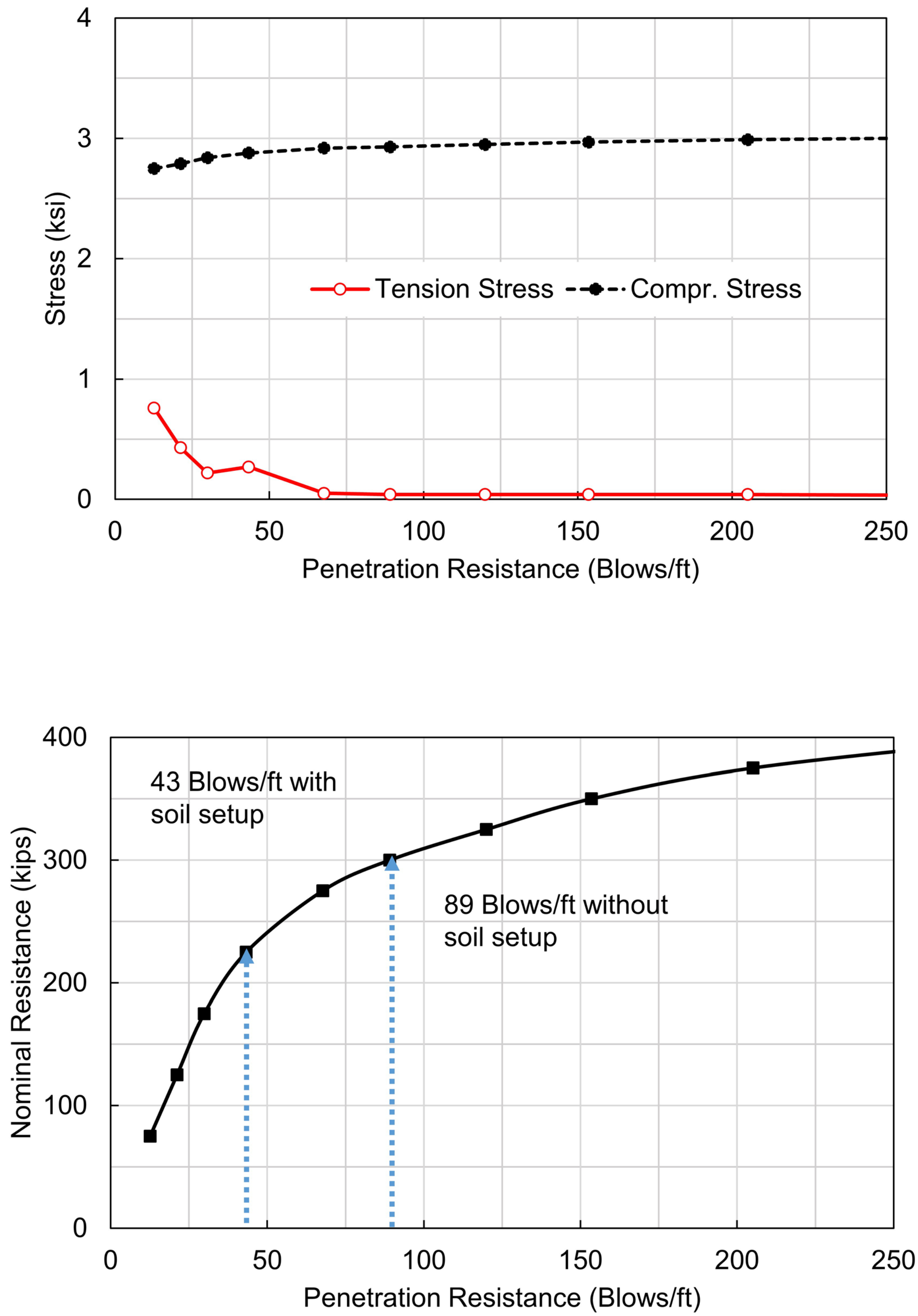


Figure 12-13 Using a bearing graph with soil setup.

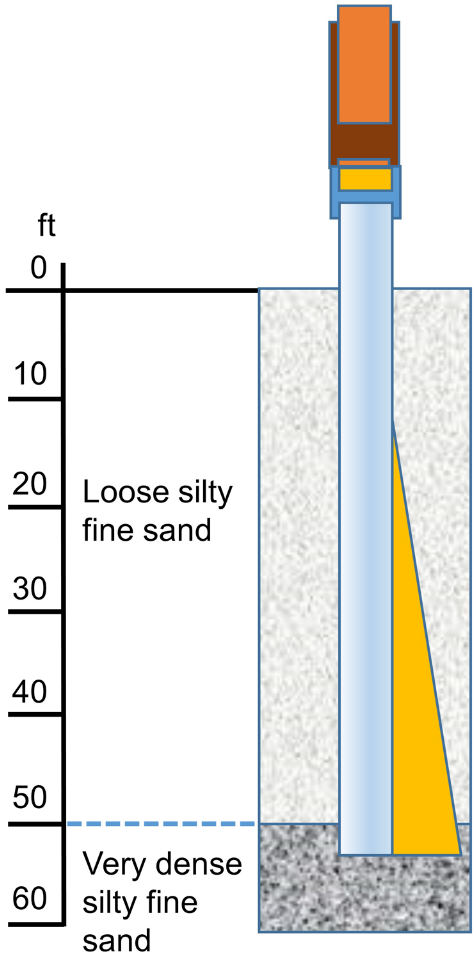
12.5.6 Example 6 – Driving System Characteristics

Example 6 presents a wave equation comparison of two hammers having the same potential energy. Dynamic formulas such as the Modified Gates formula consider only the potential energy of the driving system. Therefore, the penetration resistance required for a specific nominal resistance by this and most other dynamic formulas would be the same for two hammers provided that the hammers had the same potential energy. In this example, the penetration resistances predicted by the wave equation for these two hammers in the same pile-soil condition is, however, quite different.

In this example, a 14 inch O.D. x 0.375 inch wall, closed end pipe pile is to be driven to a nominal resistance of 400 kips. The pile has a furnished length of 66 feet and an embedded length of 52.5 feet. 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. The example problem's soil profile is presented in Figure 12-14. With a very dense, dry soil at the pile toe, the normal program recommendation for the quake at the pile toe is D/120 or 0.12 inch. However, experience with the bearing layer from previous dynamic measurements showed that the silty fine sand at this site is highly elastic and has a larger than normal toe quake of 0.40 inch.

The contractor is considering using either a Vulcan 014 air hammer or an ICE 42-S open end diesel hammer to drive the piles. Both hammers have the same manufacturer's rated hammer energy of 42 ft-kips. However, the ram of the Vulcan 014 is roughly 3.5 times heavier than the ram of the ICE hammer. Details of these hammers and their associated proposed driving systems are summarized in Table 12-2.

Wave equation results for the two hammers are plotted on the same bearing graph in Figure 12-15. For the high toe quake case (0.40 inches), wave equation analysis calculates a penetration resistance of 94 blows/ft to achieve a 400 kip nominal resistance with the heavy ram air hammer whereas the lighter ram diesel hammer requires a penetration resistance of 209 blows/ft. For the standard toe quake case (0.12 inches), the heavy ram air hammer requires a penetration resistance of 56 blows/ft while the light ram diesel hammer requires 99 blows/ft. Hence, even though both hammers have the same potential energy, the required penetration resistance for the 400 kip nominal resistance is quite different.



First hammer model:	ICE 42-S
Rated energy:	42 ft-kips
Second hammer model:	Vulcan 014
Rated energy:	42 ft-kips
Hammer cushion	varies
Helmet weight:	varies
Pile type:	Closed End Pipe
Pile diameter:	14 inches
Pile wall thickness:	0.375 inches
Pile length:	66 ft
Final pile penetration:	52.5 ft
Nominal resistance:	400 kips
Nominal shaft resistance:	30% (120 kips)
Nominal toe resistance:	70% (280 kips)
Toe quake:	0.4 inches

Figure 12-14 Example 6 – Problem profile.

Table 12-2 Example 6: Proposed Hammer and Driving Systems

Hammer Model	Vulcan 014	ICE 42S
Ram Weight, kips	14	4.09
Rated Energy, ft-kips	42	42
Rated Stroke, ft	3.0	10.3
Helmet Weight, kips	1.67	2.05
H. Cushion Material	Nycast	Blue Nylon
H. Cushion E-Mod, ksi	208	175
H. Cushion Area, in^2	234	398
H. Cushion Thickness, in	6.0	2.0
H. Cushion COR	0.91	0.92

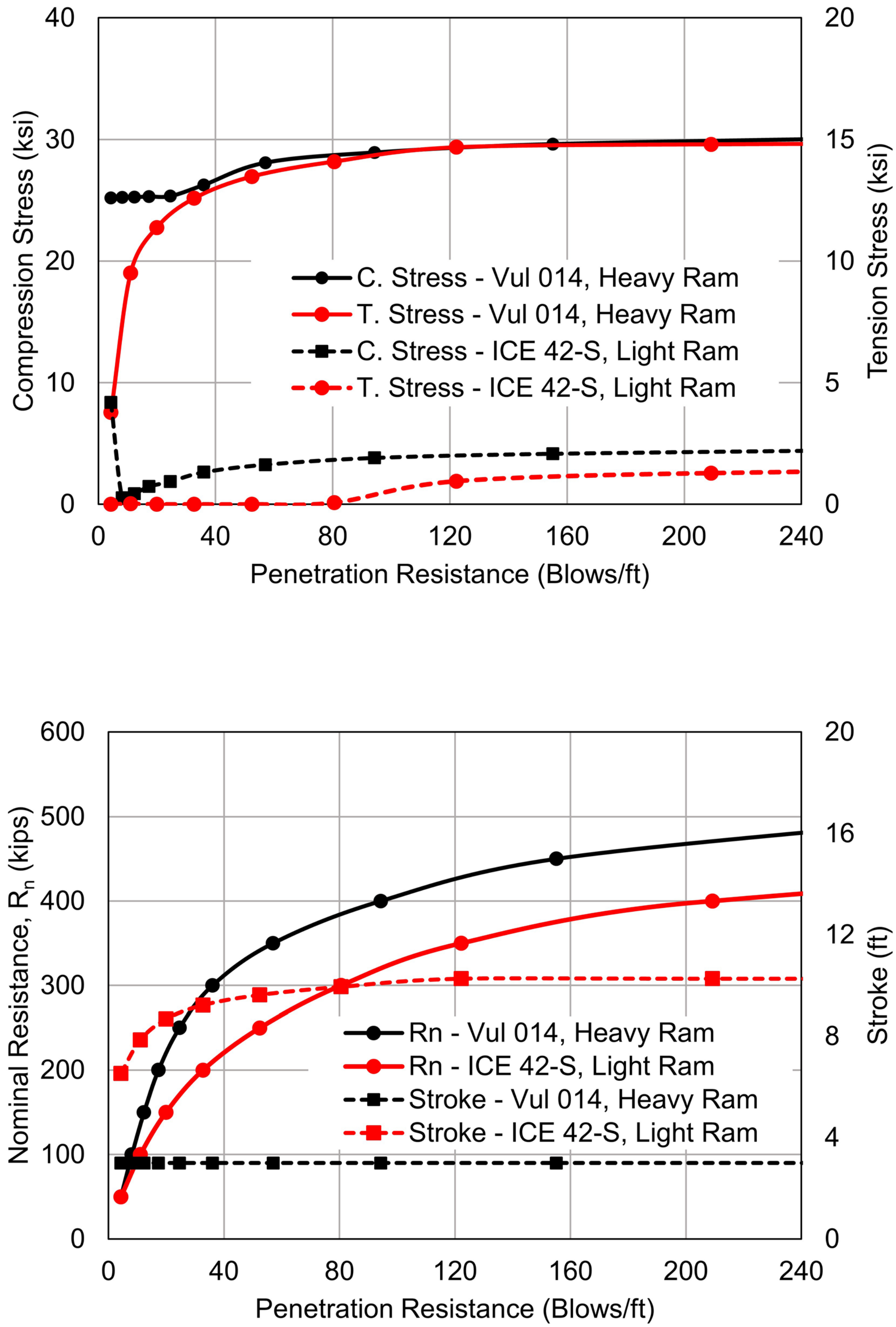


Figure 12-15 Bearing graph – for two hammers with equivalent potential energy and large toe quake.

Even though the Vulcan 014 requires a lower penetration resistance (blow count) for the same nominal resistance and has a lower efficiency (0.67 vs. 0.80 for the diesel hammer), it transfers roughly 20% more energy into the pile. This is because, first, the diesel hammer uses part of its energy to compress the gasses prior to impact. Second, the lower impact velocity of the heavy hammer is associated with lower energy losses. And lastly, the duration of the air hammer's impact is longer and consequently more effective at driving into a highly elastic soil with a large quake.

It is, however, interesting to note that for the smaller normal quake case in Figure 12-16, the lighter ram's blow counts improve relative to the heavier hammer at high nominal resistances and blow counts. This phenomenon can be explained with the diesel hammer's higher stroke and, therefore, higher impact force during harder driving. At the higher penetration resistance levels, energy is not as important as force to overcome the soil resistance.

This example illustrates the dynamic complexities of hammer-pile-soil interaction. Clearly, the potential energy alone, which is the sole hammer input in dynamic formulas, does not adequately assess pile drivability.

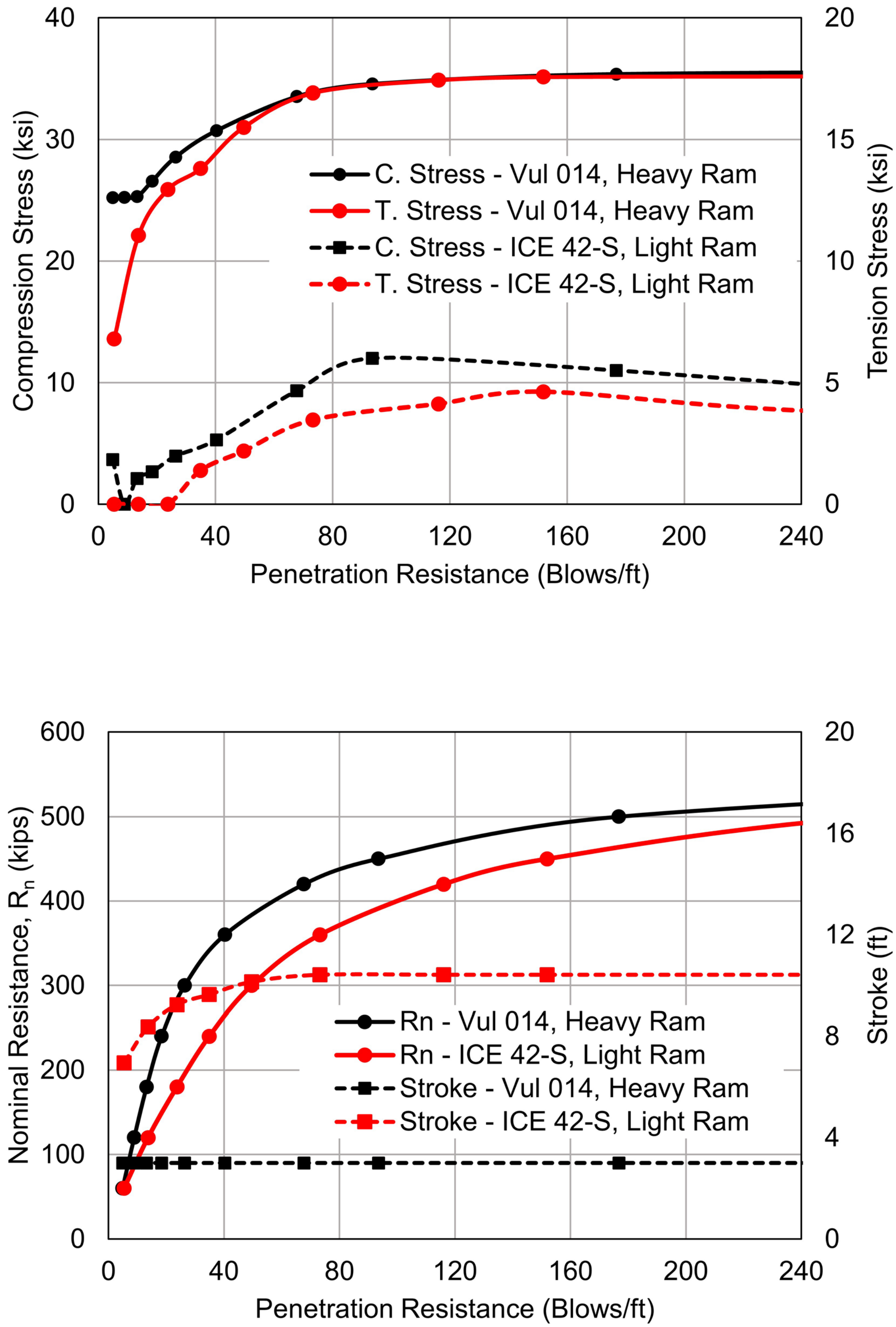
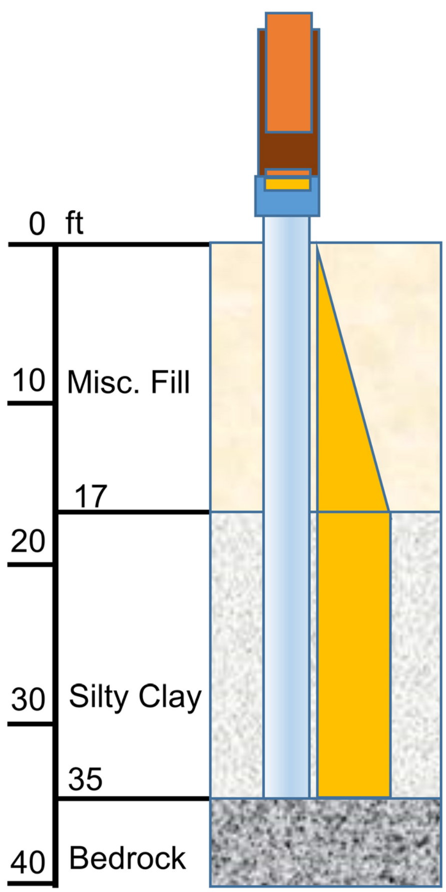


Figure 12-16 Bearing graph – for two hammers with equivalent potential energy and normal toe quake.

12.5.7 Example 7 – Assessment of Pile Damage

Another pile driving construction concern is pile damage. Although it is frequently assumed that steel H-piles can be driven through boulders and fill materials containing numerous obstructions, pile installation reviews reveal that this assumption is invalid. H-piles without commercially manufactured pile toe reinforcement present one of the most commonly damaged pile types. The damage occurs because of the ease with which flanges can be curled, rolled, and torn. Because deforming a pile plastically or otherwise non-elastically requires a significant amount of energy, pile damage has a detrimental effects on both penetration resistance and, therefore, nominal resistance activation (the blow count will be higher at the same resistance).



Hammer model:	Pileco D12-42
Rated energy:	29.9 ft-kips
Hammer cushion:	
Type:	Aluminum-Micarta
Thickness:	2 inches
Helmet Weight:	1.3 kips
Pile type:	Steel H-pile
Pile size:	HP 12x53
Pile length:	40 ft
Final pile penetration:	35 ft
Nominal resistance:	380 kips
Nominal shaft resistance:	20% (76 kips)
Nominal toe resistance:	80% (304 kips)

Figure 12-17 Example 7 –- Problem profile.

This example illustrates how the wave equation can be used to obtain insight into a driving situation involving pile damage. The project conditions are shown in Figure 12-17. The HP 12x53 H-piles are 40 feet in length with a factored load of 304 kips and a nominal resistance of 380 kips based on confirmation with a static load test.

The soil profile consisted of 15 to 17 feet of miscellaneous fill, including some bricks and concrete. Below the fill, 15 feet of silty clay overlay bedrock which was encountered at a depth of about 35 feet. The design called for driving the piles to bedrock.

The contractor selected a Pileco D12-42 single acting diesel hammer with a rated energy of 29.9 ft-kips to drive the piles. Using the FHWA modified Gates formula specified in the contract documents, the required penetration resistance was 46 blows/ft with this hammer for a nominal resistance of 380 kips.

Figure 12-18 presents the wave equation results indicating a nominal resistance of 245 kips at the Gates blow count of 46 blows/ft, well below the required 380 kip nominal resistance. On the other hand, the wave equation also showed that the maximum compression stresses at the pile toe would reach 40 ksi when the required nominal resistance was reached at 104 blows/foot and even 43 ksi at refusal. Most H-pile sections are now made of steel with a 50 ksi yield strength. However, in this example case, the yield strength was only 36 ksi and the compression driving stress limit only 32.4 ksi (90% of the yield strength).

In accordance with the contract requirement, several static load tests were conducted. In all cases, the piles failed to carry the 380 kip nominal resistance in spite of the fact that several of the piles were eventually driven to a penetration resistance exceeding 240 blows/ft with no indication of damage at the pile head. Because of the high penetration resistances to which several piles were driven, it was apparent that even harder driving would not result in a higher nominal resistance. 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 at the pile toe. The flanges were separated and rolled up from the web. While the damage probably occurred as the unprotected piles were driven through the miscellaneous rubble fill, it is also obvious from Figure 12-18 that the refusal blow count would generate dynamic steel pile stresses in excess of 40 ksi, and therefore in excess of the driving stress limit. The highest stresses would occur at the pile toe according to the numerical wave equation results while the stresses at the pile top were significantly lower (thus no damage at the pile head). Tables 12-3 and 12-4 show the maximum values of forces, stresses, and other variables calculated by wave equation analysis for both the required nominal resistance of 380 kips and the refusal driving situation (penetration at or in excess of 240 blows/ft) respectively.

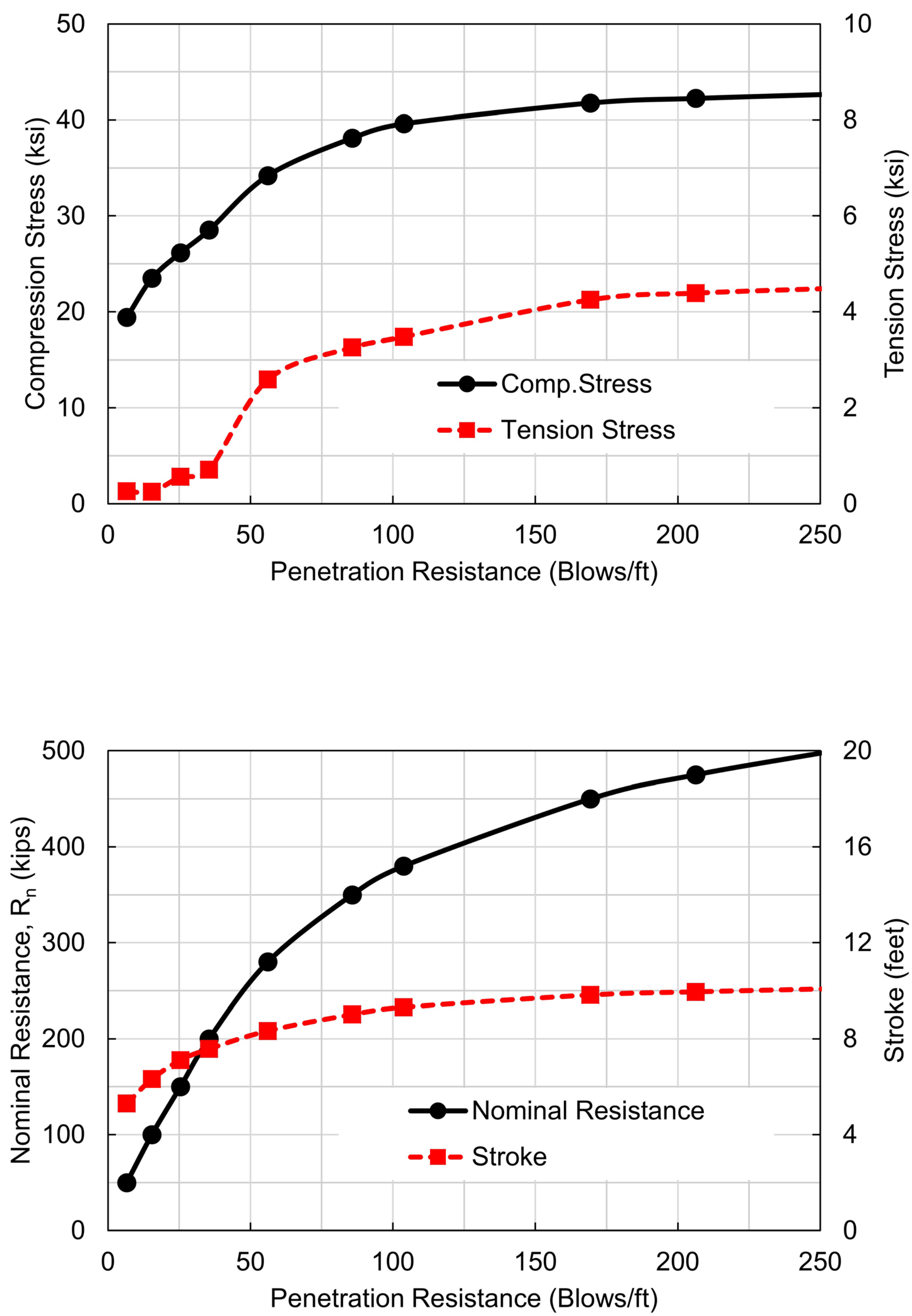


Figure 12-18 Wave equation bearing graph for proposed driving system.

Table 12-3 Example 7 – Maxima of Forces, Stresses and Other Variables for Required Nominal Resistance of 380 kips

Segment No.	Max Tension Force kips	Max Compr. Force kips	Max Tension Stress ksi	Max Compr. Stress ksi	Max Velocity ft/s	Max Displacement inches	Max Transfer Energy kip-ft
Top = 1	0.0	502.2	0.00	32.40	16.49	0.584	15.99
2	-18.9	492.1	-1.22	31.75	16.60	0.556	15.61
3	-35.7	498.7	-2.31	32.17	16.56	0.524	15.07
4	-49.1	501.2	-3.17	32.34	16.49	0.490	14.35
5	-54.0	502.4	-3.48	32.41	16.29	0.454	13.46
6	-53.9	499.6	-3.48	32.23	16.06	0.415	12.42
7	-50.7	495.6	-3.27	31.98	15.77	0.374	11.23
8	-39.6	502.7	-2.56	32.43	15.43	0.335	10.13
9	-31.3	523.0	-2.02	33.74	15.17	0.301	9.24
10	-27.6	537.3	-1.78	34.66	15.02	0.272	8.57
11	-21.7	556.4	-1.40	35.90	14.01	0.244	7.96
Toe = 12	-12.2	613.9	-0.79	39.61	9.58	0.216	7.46

Table 12-4 Example 7 – Maxima of Forces, Stresses and Other Variables for at Refusal Driving Condition (Nominal Resistance of 500 kips)

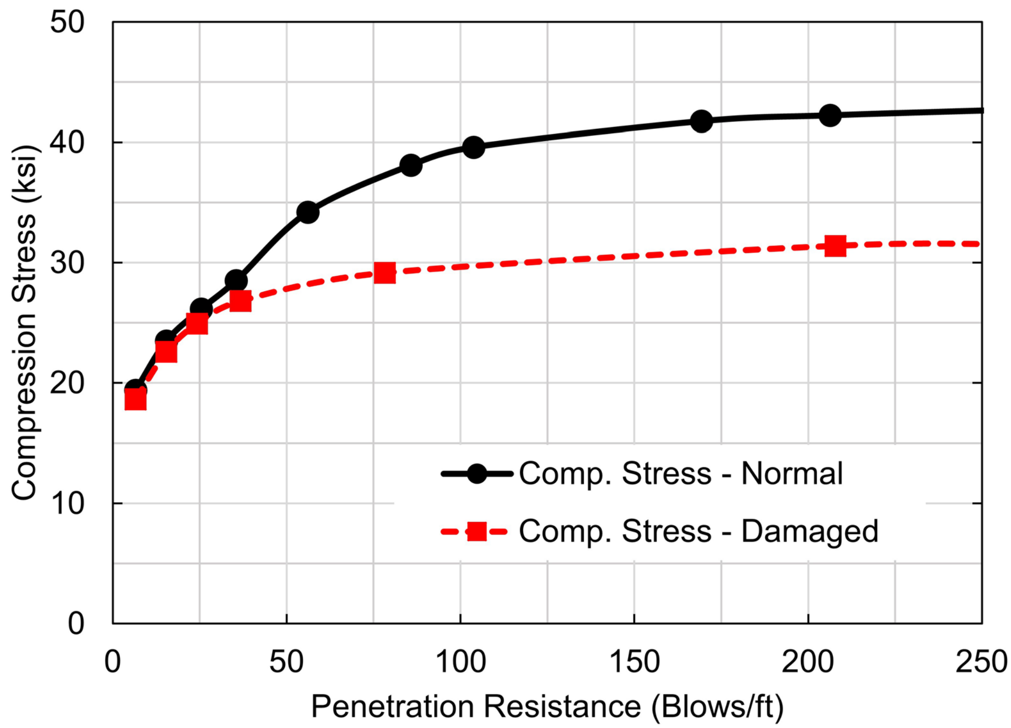
Segment No.	Max Tension Force kips	Max Compr. Force kips	Max Tension Stress ksi	Max Compr. Stress ksi	Max Velocity ft/s	Max Displacement inches	Max Transfer Energy kip-ft
Top = 1	0.0	586	0.00	37.80	17.51	0.590	17.39
2	-23.6	539.2	-1.52	34.79	17.63	0.559	16.94
3	-40.7	530.5	-2.63	34.23	17.61	0.526	16.32
4	-59.3	532.2	-3.82	34.33	17.46	0.490	15.48
5	-69.1	532.5	-4.46	34.36	17.22	0.452	14.43
6	-69.6	540.2	-4.49	34.85	16.90	0.411	13.19
7	-57.0	552.2	-3.68	35.62	16.50	0.367	11.77
8	-53.0	566.0	-3.42	36.52	16.09	0.320	10.23
9	-50.2	582.3	-3.24	37.57	15.69	0.271	8.65
10	-40.8	591.6	-2.63	38.17	15.33	0.224	7.27
11	-23.6	613.5	-1.52	39.58	13.79	0.184	6.23
Toe = 12	-13.3	661.6	-0.86	42.68	8.57	0.147	5.48

The effect of the damage on the pile drivability can also be evaluated with a wave equation analysis. Since static load tests indicate that piles driven as hard as 240 blows/foot did not support the 380-kip load, one pair of nominal resistance and penetration resistance values is available as a reference point on the wave equation bearing graph. For the damaged pile scenario, a bearing graph may be calculated for the pile with damaged toe by a simply reducing the elastic modulus of the lowest pile segment until results agree with the penetration resistance and nominal resistance observations. In the present case, the reduced toe segment stiffness was found to be approximately 10% of the normal value.

Figure 12-19 presents wave equation results for both the undamaged and the damaged pile toe scenarios. The results indicate that the nominal resistance of 380 kips could not be obtained for the damaged pile, regardless of the penetration resistance. Essentially, the damaged pile section "cushioned" the hammer blow at the pile toe and attenuated the hammer energy. Once damaged, the soil resistance at the pile toe could not be overcome, and therefore, the pile toe would not advance. The above illustrates that not only blow counts but also driving stresses also may limit the drivability of a pile to the required nominal resistance.

The potential for pile damage 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. As pointed out earlier, the wave equation bearing graph in Figure 12-19 illustrates that the nominal resistance of 380 kips could only be obtained by the contractor's driving system at a penetration resistance of 104 blows/foot or more with an associated pile toe stress of 40 ksi, a stress in excess of the steel yield strength of 36 ksi and recommended limit driving stress of 32.4 ksi.

Considering that the stresses calculated by the wave equation are averages over the cross section, a non-uniform distribution of the soil or rock resistance could have added significant additional bending stresses in the steel pile near its toe. Hence, the potential damage would have been clearly apparent at the time of the contractor's hammer submittal had a wave equation analysis been performed. Additional wave equation analyses of the contractor's driving system could have been performed at the same time to determine if driving stress levels could be acceptably reduced by using reduced fuel settings or shorter hammer strokes. If driving stresses could not be controlled in this manner, as is most likely in the present case, approval of the proposed driving system should not have been obtained, and either alternate hammers should have been evaluated or a higher steel yield strength required.



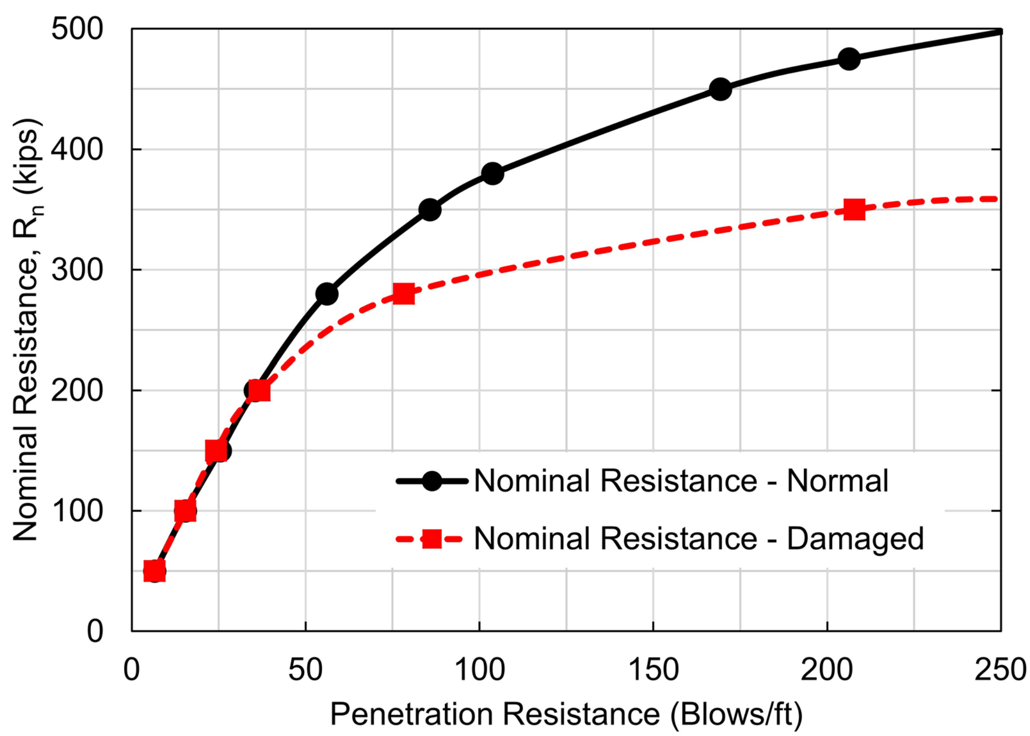


Figure 12-19 Wave equation bearing graphs for damaged and undamaged pile.

In any event, where H-piles have to be driven through materials that could include obstructions or where piles have to be driven to hard rock, it is always strongly recommended to protect the pile toe with a so-called driving shoe. Driving shoes are discussed in Chapter 16 and consist of steel castings which can be welded to the pile toe. Driving shoes tend to centralize the toe resistance force and/or reinforce the flanges.

12.5.8 Example 8 – Selection of Wall Thickness

This wave equation example demonstrates the selection process for the required wall thickness of a pipe pile. Consider the soil and problem profile presented in Figure 12-20. The foundation details are based on a bridge construction situation where a major bridge was built over a lake with a water depth of 85 feet. The piles extended 25 feet above the water level. Based upon static soils analysis the piles were to be driven through 25 feet of granular overburden, 5 feet of overconsolidated glacial till, and then to a hard bedrock. Thus total pile length was 140 feet. The piles had to be designed for a factored load of 1600 kips; a resistance factor of 0.80 (static load testing was required) yielded a required nominal resistance of 2000 kips. Structural considerations called for a minimum wall thickness of 0.50 inches. However, as a drivability check, wave equation analyses were performed for the 48 inch outside diameter pipe pile with four different wall thicknesses of 0.50, 0.75, 1.00, and 1.25 inches. The analyses were based on the following assumptions:

1. A Berminghammer 6005 open end diesel hammer would be available. This hammer has a ram weight of 13.6 kips and a rated energy of 161 ft-kips.

2. The pile is driven open ended and because of its relatively shallow penetration, it will not plug.

3. The toe resistance will develop against only the steel toe area on the hard rock. The toe resistance is estimated to be 1600 kips or 80% of nominal resistance for all four pile types regardless of wall thickness.

4. Because the pile is a low-displacement type driven to hard rock, the program recommended toe quake is 0.04 inches. (For moderately hard rock the standard 0.10 inch quake could be assumed.)

5. Average shaft damping was assumed to be 0.10 s/ft.

6. During driving, open end pipe piles with diameters greater than 30 inches usually do not plug and for moderate penetrations (say less than 10 diameters) it may be assumed for the dynamic analysis that at least partial internal friction occurs. The shaft resistance in the present case was assumed to act equally on the inside and outside surfaces of the pile and amount to 20% of the total nominal resistance. For greater pile penetration depths, it would be less likely that so much internal friction occurs during driving because the effective stresses acting inside the pipe and, therefore, the unit shaft resistance inside the pipe would be less than for the outside shaft resistance. For that reason, the internal friction is often only assumed to be either 0 or 50% of the outside friction. In wave equation analysis modeling, a 50% inside friction could be simulated by a 50% increase in the pile perimeter while the unit shaft resistance would remain as calculated for the outside pile wall.

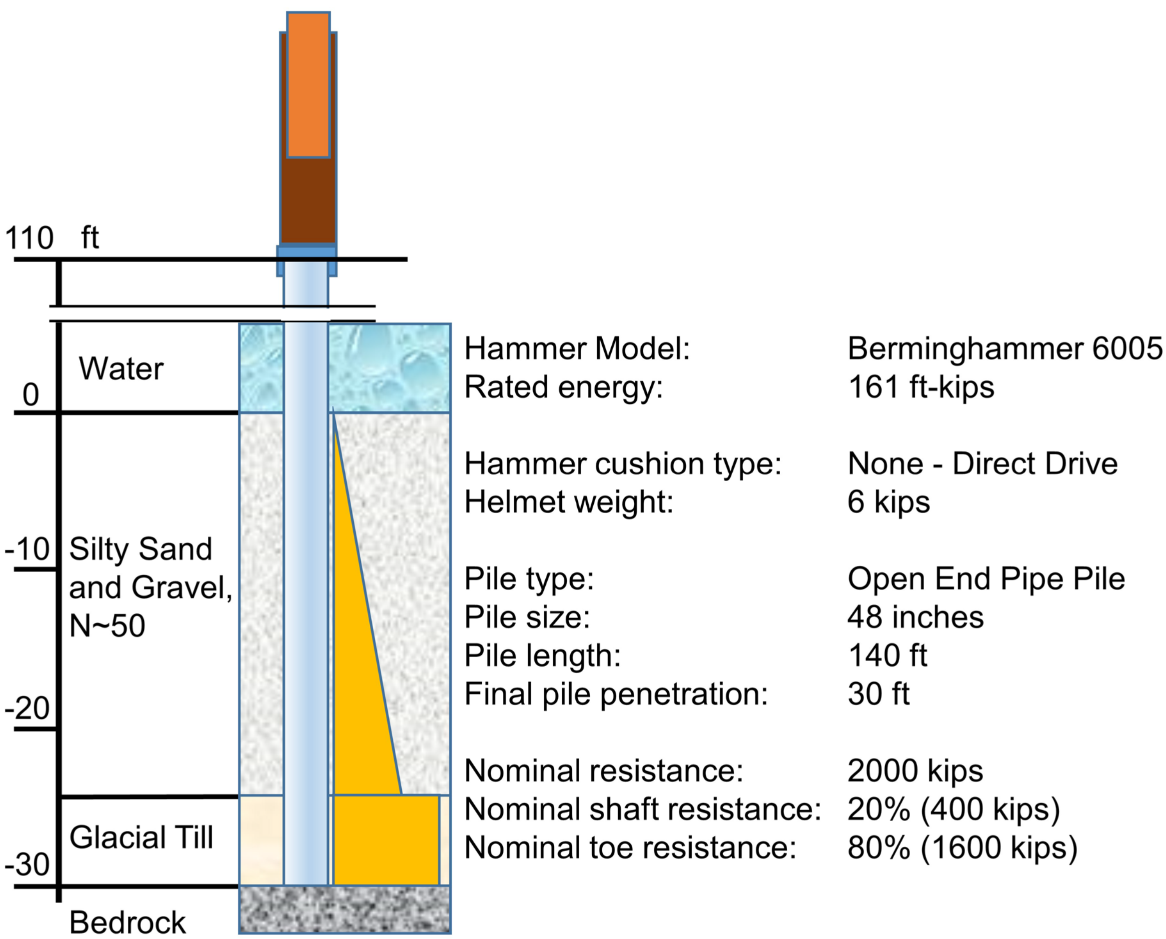


Figure 12-20 Example 8 – Problem profile.

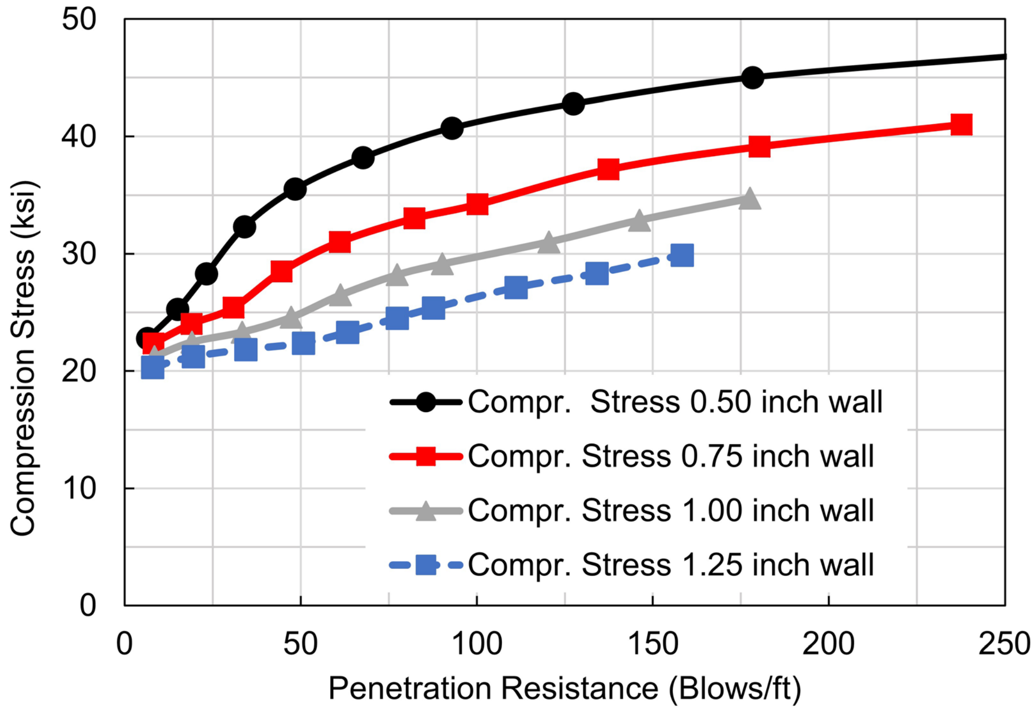
Figure 12-21 presents the results of these analyses in the form of bearing graphs and Table 12-5 summarizes calculated stresses, blow counts, stroke and transferred energy for the required 2000 kip nominal resistance.

Calculated maximum driving stresses are not more than 43 ksi for the 0.50 inch wall pipe and less than 35 ksi for the other thicker wall pile sections. Assuming that the pipe piles would be manufactured from Grade 3 pipe with yield strength of 45 ksi, the driving stress limit would be 40.5 ksi. Therefore, all except the 0.5 inch wall piles should withstand the driving stresses. However, uneven or sloping rock would cause higher localized stress concentrations. It would, therefore, be wise to choose a greater wall thickness for which the driving stresses allow for an additional margin of safety.

Table 12-5 Example 8 – Wave Equation Results for Four Potential Pipe Pile Wall Thicknesses at 2000 kip Nominal Resistance

Wall Thickness inches	Stroke feet	Transferred Energy ft-kips	Compression Stress ksi	Blow Count blows/ft
0.50	9.1	93	42.8	127
0.75	8.7	79	34.2	100
1.00	8.5	71	29.1	90
1.25	8.6	68	25.4	88

For hammer approval, AASHTO (2010) LRFD Bridge Construction Specification require the wave equation penetration resistance to be between 24 and 120 blows/foot. The wave equation analysis results indicate that the 0.50 inch thick wall pipe would require too high a blow count while the 0.75 inch wall thickness would have an acceptable penetration resistance. Use of the 1.00 or 1.25 inch wall thickness would help little with either drivability or driving stresses and be much more costly. Table 12-5 also shows that while hammer strokes were nearly the same, the transferred energies varied significantly with the more flexible piles accepting more energy. However, this energy was needed to elastically compress the very long piles and was therefore not available for actual work to overcome the soil resistance.



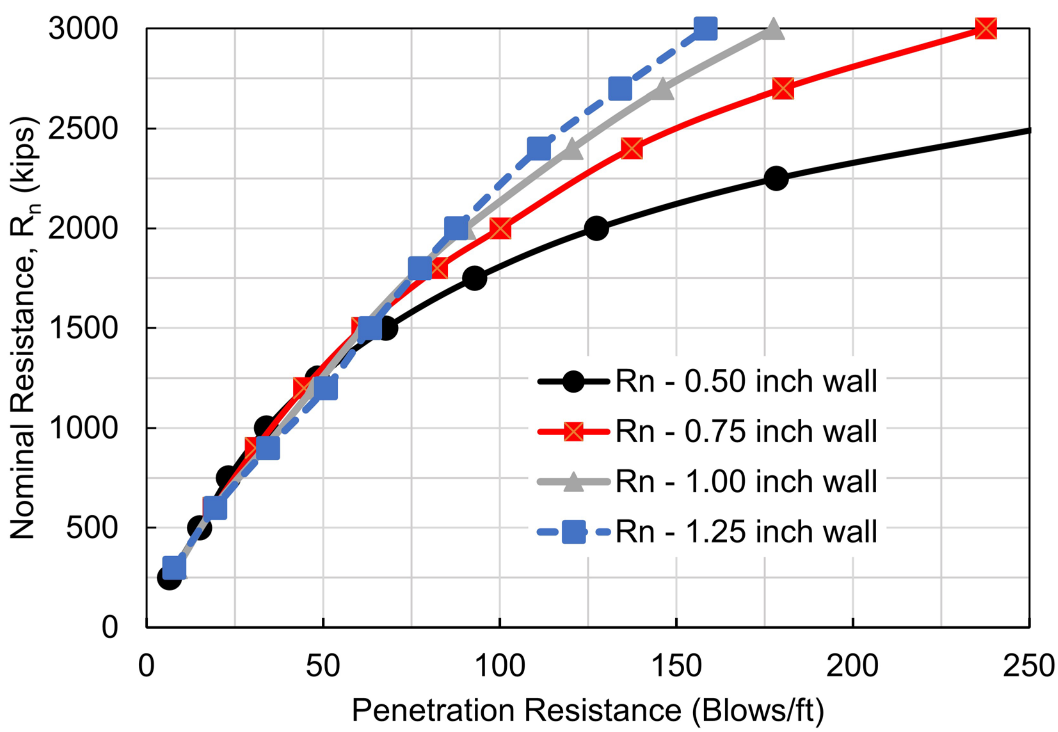


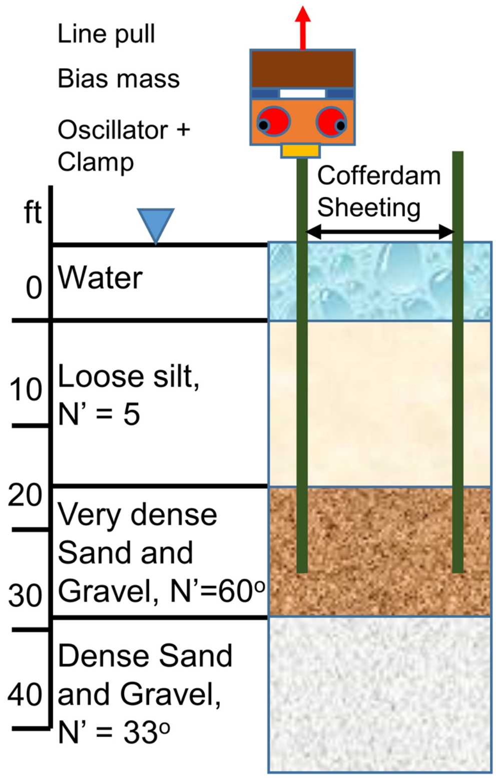
Figure 12-21 Bearing graphs for open end pipe piles with wall thicknesses of 0.50, 0.75, 1.00, and 1.25 inches.

The actual project which was used as a guide in setting up this example was successfully completed with the 0.75 inch wall thickness. Reportedly, the static load test did not fail under a 2000-kip proof test. While this project was successfully completed, a caution about driving and analyzing large diameter open end pipe piles should be added. When driving to dense soil layers, large pipes may not plug during driving and they may plug under static loading conditions. Pile driving may therefore be much easier than predicted by the wave equation if the analysis were made on the assumption of a plugged pile which behaves like a displacement pile with high toe resistance and a large toe quake. The situation may also be different at the end of driving and during a restrike. It is recommended to perform wave equation analyses with upper and lower bound soil resistance values to assess hammer sizing.

It is also important to remember that a driving criterion for a pile driven to hard rock should not be specified in terms of blows per unit penetration, but rather as a maximum penetration for a certain number of hammer blows. In the present case, for the pile with the ¾ inch wall (a required blow count of 100 blows per foot was calculated for the ¾ inch wall pile) it would be reasonable to require that the pile be accepted when its penetration under 10 consecutive hammer blows is less than 0.5 inches while the hammer stroke is between 8.5 and 9.0 ft. Specifying that the pile would have to penetrate a certain distance into a hard rock to qualify for having achieved the nominal resistance would most probably lead to pile toe damage. Even when driving into moderately hard rock, overdriving the piles should be done cautiously. This is sometimes problematic, since some shales or weathered rock materials exhibit relaxation after driving and it is desirable to drive piles in those formations to a capacity in excess of the required nominal resistance. Wave equation results should indicate that the higher resistance can be achieved within material stress limits if overdriving is required.

12.5.9 Example 9 – Evaluation of Vibratory Driving

This example illustrates the use of a wave equation analysis for evaluating vibratory hammer installation of the sheet piles required for the cofferdam construction. The situation is depicted in Figure 12-22 and is similar to Example 3. The sheet piles must be installed using a vibratory hammer. The contractor has an ICE 815 hammer available and intends to drive pairs of AZ18 sheet piles whose combined cross sectional area is 29.5 in^2. These are Z-section sheets, each with a width of 24.8 inches, a depth of 15 inches and a thickness of 0.37 inches. At the time of sheet pile installation, the soil within the cofferdam is not excavated. The 50 foot long sheet piles are vibrated from mudline to an estimated depth of 35 feet below mudline.



Hammer:	ICE 815
Eccentric moment:	4.40 inch-kips
Rated power:	375 kW
Line Pull:	5.6 kips
Weight of Bias Mass:	5.5 kips
Weight if oscillator mass:	8.1 kips
Weight of Clamp:	2.2 kips
Sheet pile type:	AZ18 – double sheets
Cross sectional area:	29.5 inch^2
Sheet perimeter:	11.2 feet
Pile length:	50 feet
Penetration:	35 feet below mudline

Figure 12-22 Example 9 – Problem profile.

For the non-excavated condition, the sheet piles must first penetrate a 16.5 foot thick layer of soft silt, followed by very dense sand and gravel, and then dense sand and gravel layers. The static resistance values were calculated based on corrected SPT values of 5 for the silt, 110 for the very dense sand and gravel, and 33 for the dense sand and gravel. As is reasonable for submerged coarse grained soils subjected to vibratory driving, a shaft resistance loss of 75% of the long term resistance was assumed while no loss of resistance was assumed for the silt, which was considered cohesive. This situation is modeled with respective setup factors for silt and sand layers of 1.0 and 4.0. The associated gain/loss factor was, therefore, set to 0.25 which produces an SRD in the sands which is 25% of the long term resistance while the silt is assumed having an SRD equal to the long term resistance. (Note: it is not recommended to use these setup and gain/loss factors for drivability analyses of impact pile driving). For the toe resistance it was conservatively assumed that the full long term resistance would be present during the vibratory driving (gain/loss factor equal to 1.0).

As per program recommendations, damping factors were set to twice the values assumed for impact driving, i.e., a shaft damping of 0.30 s/ft in the silt (assumed to behave cohesive) and 0.10 s/ft in the sand and gravel layers. Toe damping was input as 0.30 s/ft for all layers. Also the damping type was switched from Smith to Smith-viscous per program recommendations. In addition, program recommendations are that only quakes should be doubled for cohesive soils but not non-cohesive soils. This guidance was followed for the silt layer while quakes were left at their 0.10 inch default values in the sands and gravels. The calculated unit soil resistance values and the associated dynamic soil parameters are shown in Table 12-6.

Table 12-6 Soil Information for Vibratory Sheet Pile Driving in Example 9

Depth feet	Unit Shaft Resist ksf	Unit Toe Resist. ksf	Skin Quake inch	Toe Quake inch	Skin Damping s/ft	Toe Damping s/ft
0.0	0.0	0.0	0.20	0.20	0.30	0.30
16.5	0.283	22.7	0.20	0.20	0.30	0.30
16.5	0.465	250.6	0.10	0.10	0.10	0.30
29.0	1.272	250.6	0.10	0.10	0.10	0.30
29.0	0.274	137.8	0.10	0.10	0.10	0.30
43.0	0.453	137.8	0.10	0.10	0.10	0.30

The pertinent information needed for the hammer model is shown in Figure 12-22, i.e., bias mass of 5.5 kips and an oscillator mass of 8.1 kips of the ICE 815 hammer. The clamp weight of 2.2 kips is added to the oscillator mass in the analysis. The two main masses are connected by elastomers which are modeled with very soft springs so as to isolate the crane line from the oscillator vibrations while at the same time adding crowd force to the driving system. The eccentric moment, represented in the program by a mass and a radius, is 4.4 inch-kips for the ICE 815 hammer. With a rated frequency of 26.7 Hz (1600 rpm) the centrifugal force of this unit is 320 kips. However, for conservatism, it is assumed that the hammer will only be run at 20 Hz which yields a centrifugal force of 180 kips. Efficiency and start-up time (the time necessary for the hammer to reach full frequency) are left at their respective 1.0 and 0.0 default values. Another input is a 7-kip line pull, or upward directed crane force, which may be needed to maintain hammer-pile system stability. Oftentimes after the pile has sufficient embedment for stability, the operator will let the line slacken which will allow for a greater downward force and therefore an increase in the speed of pile penetration. An upward directed (positive) line force is therefore a conservative input.

Figure 12-23 shows the drivability results in graphic form and Table 12-7 shows the results in numerical form. For the first analyzed depth of 6 feet, the SRD is less than all of the applied weights (hammer weight plus clamp weight plus pile weight minus line pull) causing the sheet pile to “run” as indicated in the final results by the zero (0) penetration time. After the pile penetrates into the very dense sand layer, the required penetration time sharply increases to values up to almost 20 s/ft, but reduces to much more comfortable values as the sheet pile toe enters the dense sand and gravel. The final foot of penetration requires less than 5 s/ft. At that point, the SRD ranges between 85 and 90 kips with 28 kips acting at the sheet pile toe (the steel area of the pile). The calculated compression and tension stress maxima are rather small varying between 2 and 5 ksi. These stresses are typical for vibratory pile driving, by it should be remembered that these stresses are averaged over the pile cross sectional area. Local stress concentrations of much higher magnitudes must be expected, for example in the areas surrounding the pile clamp.

Vibratory hammer refusal has occasionally been specified as low as 1 inch/min corresponding to a very high penetration time of 720 s/ft, and the results, therefore, suggest that the sheet pile installation should be possible with the 815 hammer. However, the accuracy of the wave equation prediction strongly depends on the realism of the relatively crudely estimated static resistance to driving (SRD). Furthermore, a good alignment of the sheet piles and thus no excessive interlock friction is another condition for a successful installation.

It should be emphasized that the drivability analysis of vibratory pile driving is a reasonably reliable tool for equipment selection. On the other hand driving time estimates can widely fluctuate (even from pile to pile on the same) since they are extremely sensitive to soil resistance variations, particularly at the pile toe. Even greater uncertainty exists when attempting to relate SRD and even more so, long term resistance to penetration time. While research in this area has been and is occasionally being done, no really reliable methods have yet been established that would allow for a nominal resistance calculation from vibratory driving observations.

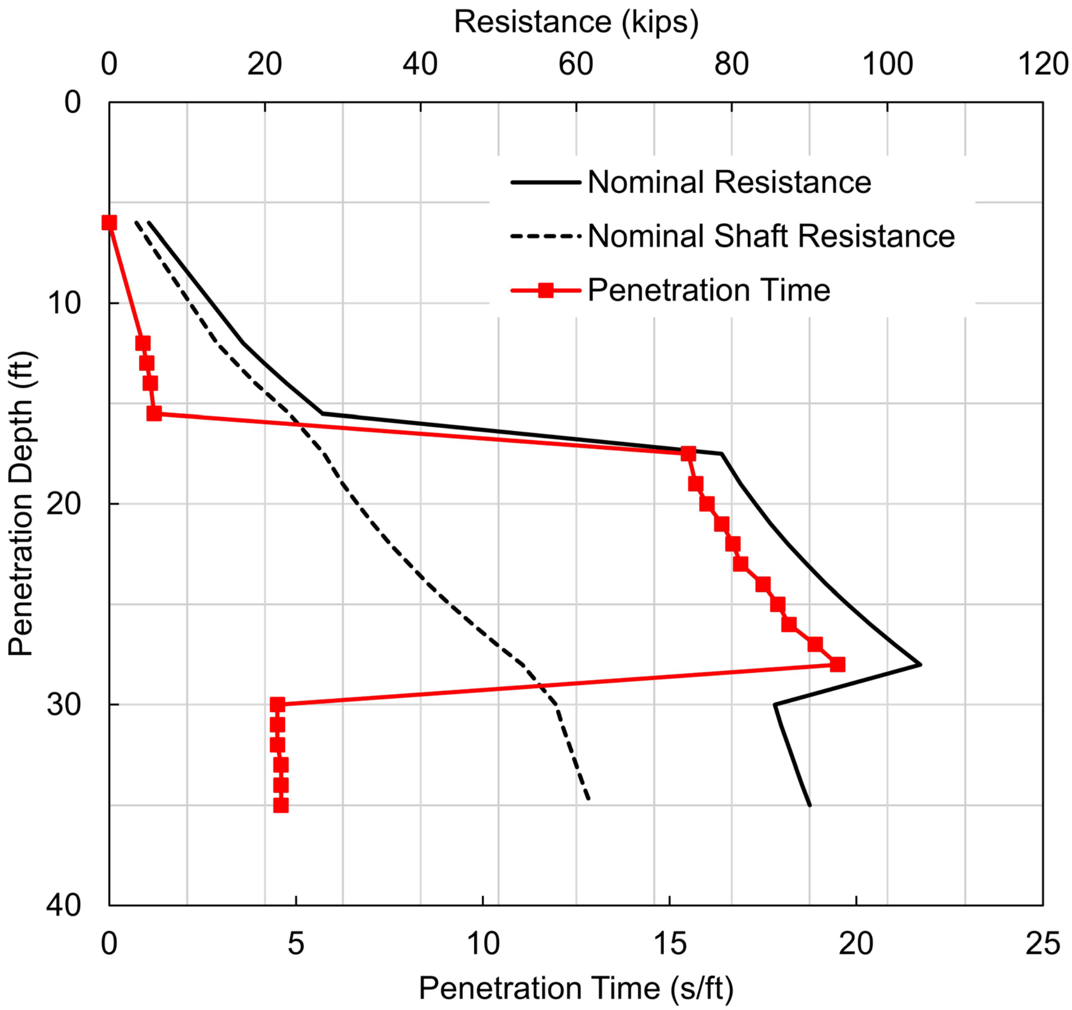


Figure 12-23 Nominal resistance and wave equation calculated penetration time for vibratory drivability analysis.

Table 12-7 Nominal Resistance and Wave Equation Calculated Results for Vibratory Drivability Analysis. Total Drive Time – 4 Minutes

Depth feet	Nominal Resist. kips	Shaft Resist. kips	Toe Resist. kips	Pen. Time s/ft	Comp. Stress ksi	Tension Stress ksi
6.0	5.1	3.5	1.7	0.0	0.0	0.0
12.0	17.2	13.8	3.4	0.9	2.4	-1.6
13.0	19.9	16.3	3.6	1.0	2.5	-1.5
14.0	22.8	18.8	3.9	1.1	1.9	-1.4
15.5	27.4	23.1	4.3	1.2	1.9	-1.3
17.5	78.7	27.6	51.1	15.5	3.1	-3.4
19.0	81.1	30.0	51.1	15.7	3.2	-3.5
20.0	83.0	31.9	51.1	16.0	3.3	-3.6
21.0	85.0	33.9	51.1	16.4	3.4	-3.7
22.0	87.2	36.1	51.1	16.7	3.5	-3.8
23.0	89.6	38.5	51.1	16.9	3.6	-3.9
24.0	92.1	41.0	51.1	17.5	3.8	-4.0
25.0	94.9	43.8	51.1	17.9	4.0	-4.2
26.0	97.8	46.7	51.1	18.2	4.1	-4.3
27.0	100.9	49.8	51.1	18.9	4.3	-4.5
28.0	104.2	53.1	51.1	19.5	4.5	-4.7
30.0	85.5	57.4	28.1	4.5	4.3	-4.0
31.0	86.3	58.2	28.1	4.5	4.3	-4.1
32.0	87.2	59.1	28.1	4.5	4.4	-4.2
33.0	88.1	60.0	28.1	4.6	4.4	-4.2
34.0	89.0	60.9	28.1	4.6	4.5	-4.3
35.0	90.0	61.9	28.1	4.6	4.6	-4.4

12.6 ANALYSIS DECISIONS FOR WAVE EQUATION MODELING

A wave equation analysis offers the engineer a variety of input and analysis options. Choosing just one of these options may limit what can be learned from the simulations. For example, it may be helpful to calculate both a bearing graph and drivability plot or investigate pile stress extrema and blow counts for both a diesel hammer and a hydraulic one. The two hammer types would be different as far as energy transfer and driving resistance even if the hammers are identically rated. The drivability analysis would show certain potential driving problems such as start-up problems for the diesel in easy driving or refusal in dense intermediate soil layers. The following sections aim at helping the analyst to understand why certain options were made available and how they can help achieve an optimal pile installation.

12.6.1 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. Some wave equation programs, provide a simplified static analysis for resistance distribution purposes. However, the basic wave equation approach does not determine the nominal resistance of a pile based on soil boring data. The wave equation calculates a penetration resistance for an assumed nominal resistance (ultimate capacity), or conversely, it assigns an estimated nominal resistance (ultimate capacity) to a pile based on a field observed penetration resistance. It is one thing to perform a wave equation bearing graph for an expected nominal resistance at a particular pile penetration and a totally different matter to actually realize that nominal resistance at that depth. Significant cost overruns can occur when pile lengths required during construction vary significantly from those estimated during design by a static analysis. To avoid such occurrences, it is imperative that a static analysis, as described in Chapter 7, precede the wave equation analysis. The static analysis will yield an approximate pile penetration for a desired nominal resistance or a nominal resistance for a certain depth. The static analysis can also generate a plot of estimated pile nominal resistance as a function of depth. As a preparation for the wave equation analysis, it is important that the static analysis evaluates the soil resistance in the driving situation (e.g., remolded soil strength, before excavation, before scour, before fill placement, etc.). For the assessment of long term static conditions, the static analysis must consider the influence of soil setup or relaxation, additional change due to excavation, water table variations, and scour, etc.

After completion of the static analysis, a wave equation analysis should be performed, leading to either a bearing graph as illustrated in Example 1, or a drivability analysis of penetration resistances and stresses versus depth as presented in Example 3. Sometimes both analyses are performed. The validity of the bearing graph depends on the proximity of the analyzed soil profile and the site variability of the soil properties. The drivability analysis calculates penetration 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 the nominal resistance of a pile given a penetration resistance at a certain depth and to formulate a driving criterion for a required nominal resistance. The drivability analysis points out certain problems that might occur during driving prior to reaching the target penetration. If the pile actually drives differently from the wave equation predictions, a reanalysis with different soil resistance parameters would be needed to match the observed behavior.

Even if 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 penetration resistance during pile installation were to differ substantially from the predicted one. Most likely, the observed penetration resistance would be lower than calculated. As an example, suppose that a pile had to be driven into a clay for a factored load of 182 kips. With a resistance factor of 0.65 (dynamic testing of at least 2% of the piles would be specified), the required nominal resistance would be 280 kips. The static soil analysis indicates that the pile should penetrate to a depth of 82 feet to meet this nominal resistance requirement. 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 (soil setup factor = 2). It is therefore only necessary to drive the pile to a nominal resistance of 140 kips, which should be achieved at the 82 foot depth. The expected end of installation penetration resistance would then correspond to 140 kips. A restrike test, performed 7 days after installation, would include soil setup effects and might show the required 280-kip nominal resistance and a much higher penetration resistance than 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.

12.6.2 External Combustion Hammer Consideration

For external combustion hammers (e.g., air or hydraulically powered hammers) one of the most important wave equation 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 proceeding the time of impact. Many sources of energy loss are usually lumped into this one number. If the hammer efficiency is set too high, an optimistically low penetration resistance would be predicted. This in turn could lead to a dangerous overprediction of nominal resistance. If the efficiency is set very low, for conservative pile nominal resistance assessments, the stresses may be underpredicted, leading to possible pile damage during installation.

Hammer efficiency and hammer stroke should be reduced for inclined (battered) pile driving. These reductions depend on the hammer type and batter angle. For hammers with internal ram energy measurements, no reductions are required to cover losses due to inclined pile driving. 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 need not 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 penetration resistances by trying various energy (equivalent stroke) values that 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 help to find the penetration resistance at which the stroke can be safely increased to maximum. It is important, however, to realize that in fact the reduced stroke is often exceeded and the maximum stroke not fully reached. Corresponding increases and decreases of efficiency to cover the uncertainty of the actual equivalent stroke, may, therefore, be investigated.

12.6.3 Diesel Hammer Considerations

The diesel hammer stroke increases when the soil resistance, and therefore penetration resistance, increases. Certain wave equation programs simulate this behavior by trying a down stroke and, when the calculated up stroke is different, repeating the analysis with the new value for the down stroke until the strokes converge. The accuracy of the resulting stroke is therefore dependent on the realism of the complete hammer-pile-soil model and should 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, non-conservative predictions of nominal resistance 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 not always mean higher transferred energy values. When a diesel hammer preignites, probably because of poor maintenance, 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, the calculated transferred energy would be much higher than the measured one, and the calculated penetration resistances (blow counts) would be non-conservatively low. It is, therefore, recommended that hammer problems are corrected as soon as detected on the construction site. If this is not possible, several diesel stroke or pressure options should be tried when matching wave equation results with field observations and the most conservative results should be selected. Section 12.8 discusses the available diesel hammer stroke options in greater detail.

Generally the hammer data file of wave equation programs contain 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 also have decreasing numbers. (For example, Delmag, APE D-Series, Pileco, ICE I-Series and other makes have fuel pump settings with 4 (maximum), 3, 2, and 1 (minimum), roughly correspond to combustion pressures of 100, 90, 81 and 73 percent of that associated with the hammer's rated energy.) Other diesel hammers may have continuously adjustable fuel pumps; for stroke control of such diesel hammers, a reduced combustion pressure may be chosen as a percentage of the data file value which corresponds to the hammer's rated energy. However, for construction control, the hammer stroke has to be measured, e.g., calculating it from the hammer's speed of operation in blows per minute using a so-called Saximeter, and adjustments of the fuel amount have to be made by the operator until the desired, analyzed stroke is achieved.

12.6.4 Vibratory Hammer Considerations

Vibratory hammers come in a variety of models, among them high frequency hammers and resonance free hammers. For the high frequency hammers, as they are approaching the natural frequency of the piles, it will be difficult to make accurate predictions of drivability and or stresses because of the uncertainty of the hammer, pile and soil response. Resonance free hammers have a variable eccentric moment which would require different hammer data file values for eccentric moments that differ from the rated one, should the hammer be run on a reduced setting. However, in general, the reduced eccentric moments are only used during the starting and stopping cycle so as to reduce the danger of low frequency soil resonance while the maximum one is used for the production driving situation.

Vibratory hammers usually perform at 100% efficiency, with the greatest uncertainty the actual frequency. Note that often reduced driving times can be achieved with lower frequencies than maximum depending on the mass and stiffness of the combined soil-pile system.

As pointed out in Example 9, there is a great difference in how the soil responds to a vibratory hammer and an impact hammer. In granular, submerged soils the shaft resistance may almost complete vanish due to a liquefaction type effect. In cohesive soils, if they are sensitive to vibration a similar very pronounced loss of resistance may occur during driving. However, in soils where the bond between pile surface and soil is not broken, possibly because the hammer amplitude is too small, the resistance of the soil attached to pile may lead to refusal. In any event, where the vibratory excitation of the soil-pile interface leads to great losses of resistance, the relationship between speed of pile penetration rate and soil resistance is virtually meaningless while for those soil where the soil-pile bond is not broken, the full soil resistance is also not mobilized.

12.6.5 Batter Pile Considerations

Pile top bending due to poor hammer alignment is more likely to occur in battered than in vertical driving because of difficulties with maintaining a good hammer–pile alignment. The problem is aggravated during restrike, because realigning hammer and pile is then even more difficult than maintaining alignment during initial driving.

Inclined pile driving also causes, sometimes unexpected, bending stress problems if the piles are not properly supported and guided by the hammer leads. In fact, for offshore leads where the pile supports the non-axial weight component of pile, leads and hammer, these bending stresses are predictable. Offshore versions of wave equation programs have been developed for those situations that include a routine for calculating these bending stresses which are then superimposed to the dynamic stresses. However, for piles supported by fixed, swinging or semi-fixed leads it is assumed that the piles are supported in such a way that no significant bending stresses result along the length of the pile that is supported by the leads. This must be confirmed on site by a thorough inspection.

It has been mentioned that external combustion hammers (primarily air or hydraulic hammers) possibly do not reach the full effective stroke during inclined or batter pile driving, because of limitations of the effective (vertical) ram travel. This should be modeled by a reduced stroke. It must be expected that the full rated hammer energy will not be available even if the hammer efficiency is satisfactory. The increased friction in the hammer due to the pile batter can be estimated under consideration of a friction coefficient and batter angle. The wave equation software's Help section should provide some helpful values. For instrumented hammers, the friction loss is generally internally measured and/or compensated for. As the pile is driven, energy measurements whether in the hammer or by pile top measurements, will be able to assess the actual hammer energy transferred from the hammer to the pile.

For open end diesel hammers, also called internal combustion hammers, the ram travel is unlimited and it is likely that the actual, inclined ram travel is greater than the vertical ram travel under similar pile and soil conditions. However, friction effects should be accounted for by a reduced efficiency value as discussed for the external combustion hammers.

12.6.6 Hammer and Pile Cushion Considerations

Hammer cushions are primarily used to protect the hammer while pile cushions protect concrete pile tops. Direct Drive hammers use no cushioning material and are sometimes used for steel pile driving situations.

Cushion materials are subjected to destructive stresses during their service and, therefore, continuously change properties. For hammer cushions, a variety of man-made materials (e.g., Micarta, Conbest, and Nylon among others) are acceptable while wood chips as a hammer cushion are totally unpredictable and therefore should never be allowed. Frequently, hammer cushions are engineered as sandwiches of aluminum plates (to extract heat) and softer cushioning materials. The input values for the combined stiffness of two cushion materials can be easily calculated from the individual material properties by remembering that the inverse of the combined stiffness, k_c, of two springs in series is the sum of the inverse of the two individual springs ($1/k_c = 1/k_1 + 1/k_2$). However, for most commonly used cushions the Help section of the wave equation software contains extensive material property tables. Wood chips or softwood with the grain parallel to the pile axis as a hammer cushion have totally unpredictable material properties and therefore should not be allowed.

It has been explained in Example 4 that pile cushions experience a particularly pronounced increase in their stiffness during driving, 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 begin to burn and quickly deteriorate. This typically happens after approximately 1500 hammer blows. For conservative stress predictions, the harder, used cushion could be modeled by an increased elastic modulus and reduced thickness. In the United States two types of pile cushions are most frequently encountered. The most common pile cushion consists of plywood sheets and the second most common pile cushion is made of oak boards. Improved agreement with measurements can be achieved if new and used plywood cushions are analyzed with elastic moduli of 30 and 75 ksi, respectively (Rausche et al. 2004). For oak board pile cushions, the respective recommended values for new and used cushions are 60 and 90 ksi. These elastic modulus values should always be used with the nominal (uncompressed) thickness. For conservative nominal resistance predictions, a less effective pile cushion may be modeled using by a somewhat lower, maybe 50% lower than normally recommended, input for both elastic modulus and coefficient of restitution.

Uncushioned or direct-drive diesel and hydraulic hammers are also frequently encountered when steel piles are driven. The advantage of these hammers is obvious: their energy transfer to the pile is not subject to varying cushion properties. For the wave equation analysis, the stiffness of the spring between hammer and helmet is derived from the elastic properties of either ram or impact block (diesels) since there is no hammer cushion. (This stiffness is very high, much higher than the stiffness values of most other components within the system, and for numerical reasons, may lead to inaccurate stress predictions. Analyses with different numbers of pile segments (the automatically selected number of pile segments is the pile length divided by 3.3 feet) would show the sensitivity of the numerical solution. In general, the greater the number of pile segments, the more accurate the stress calculation). Choosing 1 foot long segments (the number of pile segments then equals the pile length in feet) would generate a much more detailed pile model than is standard.

12.6.7 Selection of Soil and Rock Parameters

The greatest errors in nominal resistance predictions are usually observed when the soil resistance has been improperly considered. A very common error is the confusion of factored design loads with the wave equation's nominal resistance (usually called ultimate capacity in the wave equation documentations). Note that the wave equation nominal resistance always must be multiplied by a resistance factor which depends on the nominal resistance verification method and this product has to be greater than the factored load. In the past, and that is still the way the wave equation documentation describes the approach, the ultimate capacity was divided by a factor of safety to yield the allowable design load. Resistance factors suggested by FHWA and AASHTO are discussed in Chapter 7.

Since the soil is disturbed at the end of driving, it then often has a lower nominal resistance (occasionally also a higher one) than at a later time. For this reason, a restrike test should be conducted to assess the nominal resistance 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 nominal resistance of the soil deteriorates. Depending on the sensitivity of the soil, the penetration resistance may be taken from the first 3 inches of pile penetration even though this may be conservative for some sensitive soils. For high penetration resistances (e.g., more than 10 blows/inch) smaller than 3-inch penetrations should be considered or the penetration for ten blows should be accurately measured. For sensitive soils with quickly lost soil setup resistance, measuring the penetrations of individual blows in the beginning of a restrike should be attempted if possible. Of course, this information then has to be used together with the energy transferred to the pile head for each blow during the early restrike.

For construction control, rather than restrike testing many piles, it is more reasonable to develop a site specific setup factor in a pre-construction 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 resistance is then the required nominal resistance divided by the setup factor. From the wave equation calculated bearing graph and with the reduced end of driving nominal resistance, the required end of driving penetration resistance is found.

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. In general, shaft damping factors on the order of 0.05 s/ft for non-cohesive soils, 0.10 s/ft for silty sands, clayey sands, and sandy silts, 0.15 s/ft for cohesive silts and sandy clays, and 0.20 s/ft for cohesive soils are typical. For most soils, the toe damping factor is about 0.15 s/ft. The above values are general recommendations for initial driving conditions. Damping factors have been observed to vary with waiting times after driving. Thus, damping factors higher than recommended above and 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, damping factors may be much higher than recommended, and on hard rock they may be much lower. However, choosing too low a damping factor may produce non-conservative nominal resistance predictions. The program recommendations for damping factors are averages which work reasonable well as a first assumption (see input description in Section 12.7). After piles have been driven and dynamically tested, appropriate site specific values should be available.

Shaft quakes are usually satisfactory as recommended at 0.10 inch. However, for displacement piles, toe quakes can vary widely and reach values well in excess of the program recommended range of 1/60 and 1/120 of pile diameter or pile width, particularly when the soil is saturated and/or rather sensitive to dynamic effects. Only dynamic measurements can reveal more accurate soil quakes. However, short of such measurements, conservative assumptions must sometimes 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 piles achieving their nominal resistance primarily through toe resistance, large toe quakes often develop during driving in saturated soils causing the toe resistance to build up only very slowly during the hammer blow. As a consequence, at the first instant of stress wave arrival at the pile toe, little resistance exists and damaging tension stress reflections can develop in concrete piles even if the penetration resistance is high. At the same time, large toe quakes dissipate an unusually large amount of energy and therefore cause high penetration resistances. 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 should be chosen to reduce the detrimental effects of large toe quakes. Example 6 in Section 12.5.6 illustrates the effect of a large toe quake.

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

Soft rock is generally modeled like a soil with quakes and damping values chosen as for the underlying material (e.g. clay for a claystone or shale). However, pile toe compression stresses can be damaging when the piles are driven to a hard rock. Since piles generally do not penetrate into a hard rock and since the plastic stage cannot be reached, the main concern is not with the nominal resistance but with the pile toe stresses. The resistance mobilized is usually a lower bound conservative value. Hence, to be conservative it is recommended to analyze these hard rock cases with a toe quake of 0.04 inches. A lower than the normal 0.15 s/ft damping value may also be used (e.g., 0.05 s/ft).

Residual stress wave equation analyses are superior to normal analyses in basic concept and probably also in results. Unfortunately, because of lack of correlation work, no empirically determined dynamic soil constants (quake and damping values) can be recommended for use with residual stress analyses. Experience exists for fluted and tapered piles which supports using the residual stress analysis with commonly used damping factors. Residual stress analyses should be performed (maybe in addition to standard analyses for long slender piles with significant shaft resistance components,) to assess potentially damaging stress conditions and the possibility of nominal resistance values which could be much higher than indicated by the standard wave equation analysis. Note that residual stress analyses may not be meaningful for representation of early restrike situations in which energies increase from blow to blow while, in sensitive soils, capacities successively decrease. The residual stress analysis assumes that hammer energy and pile nominal resistance are constant under several hammer blows. For further information please see also Section 12.7.1 and the background information of the wave equation computer program.

12.6.8 Pile Modeling Considerations

In general, the pile is represented more accurately and reliably than other parts of the wave equation model. Of course, unit weight and elastic modulus have to be well known which is sometimes challenging for concrete and timber. Note that for concrete and timber there is a significant difference between the static elastic modulus and the dynamic one, the latter usually being significantly higher than the static one. Since approximately 2012, it has also been observed that even the steel elastic modulus can be higher by up to 7% for large diameter pipe piles than normally assumed. Plastic piles are also some times analyzed and because of the large variety of materials and their basically non-elastic behavior, dynamic testing is needed to determine the material properties prior to performing the analysis.

Modern wave equation programs automatically generate a pile model which is generally satisfactory for most commonly encountered situations. However, there are differences in the basic approach between different software products and the user should be aware of any limitations of these products in certain situations.

For the Smith-type programs which use mass and stiffness to represent the elastic properties of the hammer, driving system and pile, non-linear system components like cushions or splices with slacks are relatively easily and quite realistically represented. In addition, the default 3.3 foot long pile sections are generally adequate although smaller segments can lead to an improved program performance. Situations where smaller pile segments or also smaller computational time increments are helpful may be highly non-uniform piles or uncushioned driving systems. Like the length of the pile segments, computational time increments are normally set by the computer program, but can be modified with a factor which E.A.L. Smith called a safety factor against numerical instability. This factor is normally 1.6 in GRLWEAP and may be increased to 3 or even 5 if unusual situations cause the program to give a numerical instability warning.

For programs which use the characteristics approach to solving the underlying differential equation (e.g. Middendorp 2004) the computational time increment must be equal to the wave travel time of every segment. This leads to segments of different lengths when the pile consists of different materials such as a concrete pile with steel stinger. While the characteristics solution has the advantage of truthfully modeling ideal elastic situations, such as a square input pulse without degradation, there is a natural tendency for waves propagating along materials to maintain lower frequency components and shed higher frequency ones. This leads to a smoothing of the pulse traveling along the pile which is similar to the degradation observed when analyzing with a lumped mass model. Modeling piles with slacks in splices is also more difficult with the characteristics method than using the more forgiving Smith approach.

Unusual situations which require special attention are: installations which include followers, particularly when a soft cushion separates a steel follower from a concrete pile, mandrel driven piles, and composite piles such as a concrete pile with a steel stinger. These conditions require modeling using splices with compression and tension slacks, parallel piles and more than one toe model. Examples of these more complex situations are available in the help section of the GRLWEAP software.

Another problem frequently encountered is the uncertainty on how to model open end pipe piles. Very large diameters (say greater than 120 inches diameter) will never plug during driving and may experience some internal friction. This can be modeled by using an increased perimeter as discussed in Section 12.5.8. Smaller diameter (say 30 inches or less) open ended pipe piles, will generally plug once driven a sufficient, diameter dependent distance into a dense or very dense material. Intermediate pile sizes may or may not plug and then behave either like the piles with larger or the smaller diameters. The pile model may have to include a consideration of the mass effect and soil model needs to address the effective toe area over which toe resistance acts. While this is primarily a soil model problem, it potentially affects the mass properties of the pile model over the plug length. As noted in Chapter 7, Holloway and Beddard (1995) also reported that hammer blow intensity (impact force and energy) influenced plug formation and slippage. Brown and Thompson (2015) published a Synthesis of the current practice of design and analysis of large open ended pipe piles which provides further insight on this issue.

When the pile model to be analyzed is unusual in its complexity, it is recommended to perform sensitivity studies by comparing results with different numbers of pile segments and/or reduced time increments and possible maximum and minimum values of pile properties.

12.6.9 Comparison with Dynamic Measurements

Often, wave equation predicted stresses and nominal resistance values initially appear to agree quite well with results from field dynamic measurements, described in Chapter 10. However, there are additional observations and measurements that should be compared, such as stroke or 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 nominal resistance agreement. Thus, instead of adjusting hammer efficiency, the cushion stiffness or coefficient of restitution may require reduction. 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.

Note: The wave equation maximum stresses in the final summary table can occur anywhere along the length of the pile and therefore at a location different from where the field measurements were taken. It is therefore important to check the maximum driving stresses in the Extrema Tables for the pile segment that corresponds to the measurement location when comparing wave equation calculation and field measurement results.

Matching wave equation results to field observations and measurements is often referred to as a Refined Wave Equation Analysis (Rausche et al. 2009). The following procedure requires that wave equation input parameters for hammer, driving system, and soil resistance are adjusted and then wave equation analyses are performed for the field verified nominal resistance. The following data preparation steps and successive input parameter adjustments generally lead to an acceptable solution. As noted in Section 12.6.8, multiple wave equation software programs exist and some use different models. The correlation procedure below is specifically based on the hammer, pile, and soil models used in GRLWEAP and CAPWAP. Procedures for other wave equation and/or signal matching programs may differ depending upon the hammer, pile, and soil models used in those programs.

a. Set up a table with the observed stroke or bounce chamber pressure for diesel hammers, and measured values of compression stresses and transferred energy, both at the measurement location. Include in this table for concrete piles the PDA calculated maximum tension stresses. These values should be averages over several consistent blows of pile installation or the earliest consistent blows of restrike testing. Additional matching quantities are the CAPWAP calculated nominal resistance and penetration resistance.

b. Set up the GRLWEAP wave equation model to run bearing graphs for the actual hammer, pile, and driving system with total nominal resistance, resistance distribution, quake, and damping obtained from dynamic field measurements and by signal matching.

c. Perform wave equation analyses and compare results with table values from step a. Adjust hammer efficiency (for diesel hammers, also maximum combustion pressure) until agreement between measured and wave equation computed compression stress and transferred energy (for diesel hammers, also stroke) is within 10%. For steel piles modifications of the hammer cushion stiffness and/or its coefficient of restitution and for concrete piles adjustments of the pile cushion stiffness and/or its coefficient of restitution may also be needed.

d. After an initial agreement has been achieved for transferred energy and pile top compression stress, compare calculated penetration resistance for CAPWAP nominal resistance and associated maximum tension stresses. For steel piles, adjust hammer cushion stiffness and coefficient of restitution, and for concrete piles, adjust the equivalent pile cushion parameters, together with efficiency, to improve agreement of penetration resistance and tension stresses within the 10% tolerance.

e. Adjust the hammer efficiency to values not greater than 0.95 and not less than 50% of the standard recommended hammer efficiency values for that hammer type. The exceptions are hammers whose stroke input is based on measured impact velocity, therefore efficiency values greater than 0.95 are possible. Adjust cushion coefficients of restitution between 0.10 and 1.0.

f. If penetration resistance and stresses cannot be simultaneously matched by adjusting hammer and driving system parameters, change the shaft and toe damping and the toe quake simultaneously and proportionately to achieve agreement between measured and computed penetration resistance. Under certain conditions, it may also be necessary to change the wave equation damping model from Smith to Smith-Viscous.

Perfect agreement should not be expected between wave equation results and quantities derived from field measurements. The reason is primarily a difference between the measured pile top force and velocity and the corresponding quantities obtained by the wave equation driving system model. Also, there may be some differences in the pile model and soil models between the different analyses programs used for wave equation and signal matching. Plots of wave equation calculated and dynamic test measurements such as force and velocity can be easily generated and can sometimes explain the differences between observed or measured and calculated values.

12.7 WAVE EQUATION ANALYSIS INPUT

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 12-24), soil boring, and a static pile nominal resistance analysis.   In a case where the contractor proposes using a follower as part of the driving system, detailed drawings of the follower should also be obtained. Helpful information can also be found in the "Help" display of the wave equation program. 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 the equipment that the contractor has actually included in the driving system on the project.

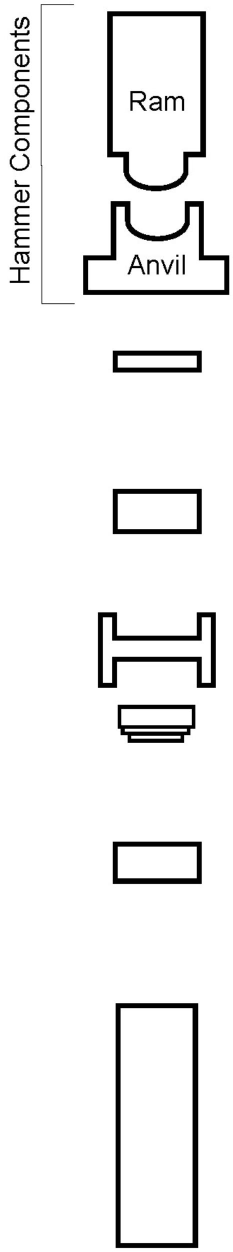
Contract No.:______________________ Structure Name and/or No.:______________________________

Project:______________________ ______________________________

______________________ Pile Driving Contractor or Subcontractor:______________________

County:______________________ ______________________________

(Piles driven by)



Hammer

Manufacturer:____________________ Model No.:____________

Hammer Type:____________________ Serial No.: ____________

Manufacturers Maximum Rated Energy:____________________(ft-lbs)

Stroke at Maximum Rated Energy:____________________________(ft)

Range in Operating Energy:________________to_____________(ft-lbs)

Range in Operating Stroke:________________to________________(ft)

Ram Weight:__________________(kips)

Modifications:____

____

____

Striker Plate

Weight:__________________(kips) Diameter:________________(in)

Thickness:________________(in)

Hammer Cushion

Material #1 Material #2 (Composite Cushion)

Name:___________________ Name:______________________

Area:____________________(in^2) Area:____________________(in^2)

Thickness/Plate:___________(in) Thickness/Plate:___________(in)

No. of Plates:_____________ No. of Plates:_________________

Total Thickness of Hammer Cushion:__________________________(in)

Helmet

Weight:__________________(kips)

Insert (If Any)

Weight:__________________(kips)

Total Weight of Helmet and Insert:__________________________(kips)

Pile Cushion

Material:____

Area:____________________(in^2) Thickness/Sheet:___________(in^2)

No. of Sheets:______________

Total Thickness of Pile Cushion:________________(in)

Maximum Thickness Accommodated by Helmet :________________(in)

Pile

Pile Type:____

Wall Thickness:____________(in) Taper:______________________

Cross Sectional Area:_______(in^2) Weight/ft:___________________

Ordered Length:_________________(ft)

Factored Resistance:_______________(kips)

Nominal Resistance:_______________(kips)

Description of Splice:____

____

Driving Shoe/Closure Plate Description:_________________________

____

Submitted By:___________________ Date:______________________

Telephone No.:__________________ Email:______________________

Figure 12-24 Pile driving and equipment data form.

The following sections explain the most important input quantities needed to run the GRLWEAP program. For a more detailed explanation of input quantities, reference is made to the program's Help Section (function key F1 or F3 or click on Help).

The (second topic) of the Help Menu (F1 or click on Help) explains the Main Input screen and all of its menus, data entry fields, and information indicators. Figure 12-25 shows this Help Window as it first appears and subdivides the Main Input screen into 10 major sections:

A Standard Window Menus
B Icons for standard Windows Operations
C Icons for GRLWEAP displays and operations
D GRLWEAP Drop-Down Menus
E Input fields for Title and Hammer Selection
F Hammer Parameters and Pile Material Selection
G Hammer and Pile Cushion Input
H Pile Data Input
I Ultimate Capacity (Nominal Resistance) Input or Gain/Loss Factors
J Soil Parameters

Although a simple bearing graph analysis only requires input in the GRLWEAP Main Input Screen, it is recommended to utilize the step-by-step input requests generated after clicking on the “New Document” icon (or New in the File Menu).

The Job Information window shown in Figure 12-26 will display first, accepting input of a title of up to 40 characters and the assignment of a file name and directory. Browse may be used to navigate the user’s computer and assign the desired directory.

Clicking on Next will open up the Select Hammer window shown in Figure 12-27. The GRLWEAP program includes a hammer data file in which the major mechanical properties of approximately 1000 hammers are stored. By selecting an identification number (ID) and/or corresponding hammer name in the List of Hammers window, the user prompts the program to automatically input the selected hammer’s properties. Note that the automatic hammer input assumes use of a well maintained and unmodified hammer.

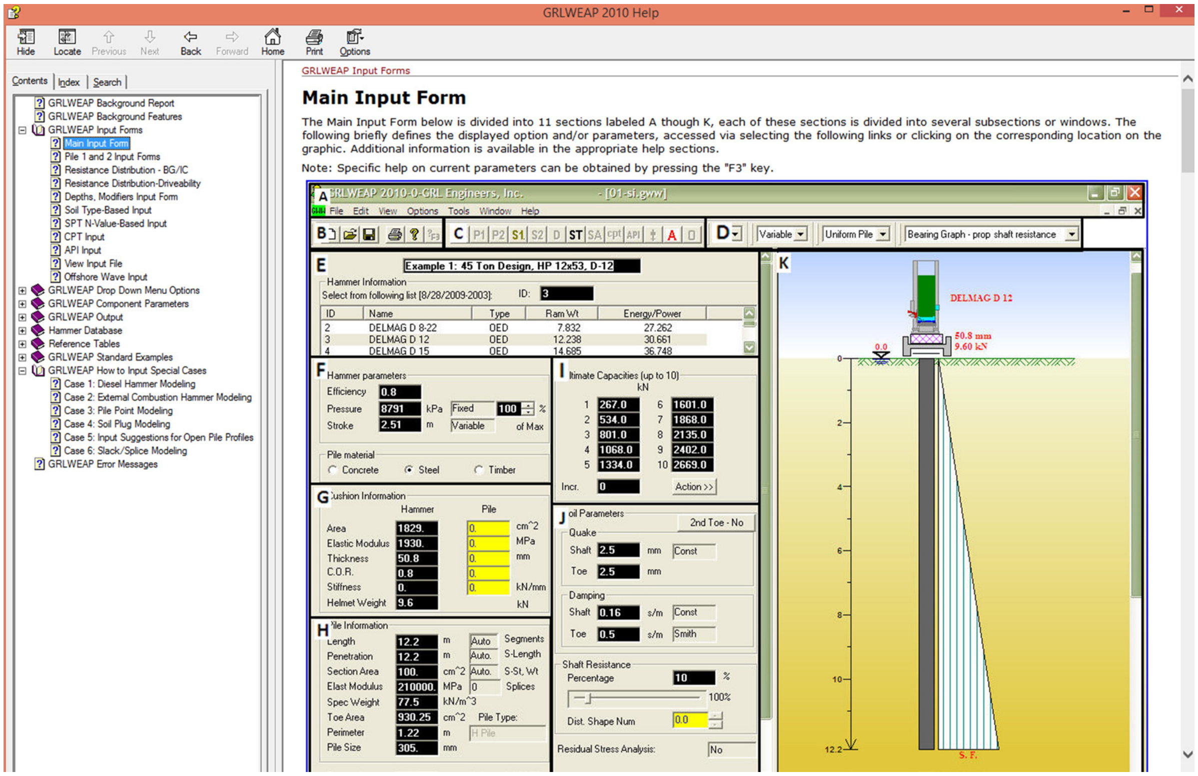


Figure 12-25 GRLWEAP help window for main input form.

While initially all hammers are displayed in the order of hammer **ID** number, the display may be reorganized by certain hammer types or manufacturers. Hammer types are OED (Open End Diesels), CED (Closed End Diesels), ECH (External Combustion Hammers, including the air, steam, hydraulic and drop hammer categories), and VIB (Vibratory Hammers). The user can also organize the contents in the List of Hammers window by hammer **Name**, **Type**, **Ram Weight** or **Rated Energy** by clicking on the column heading.

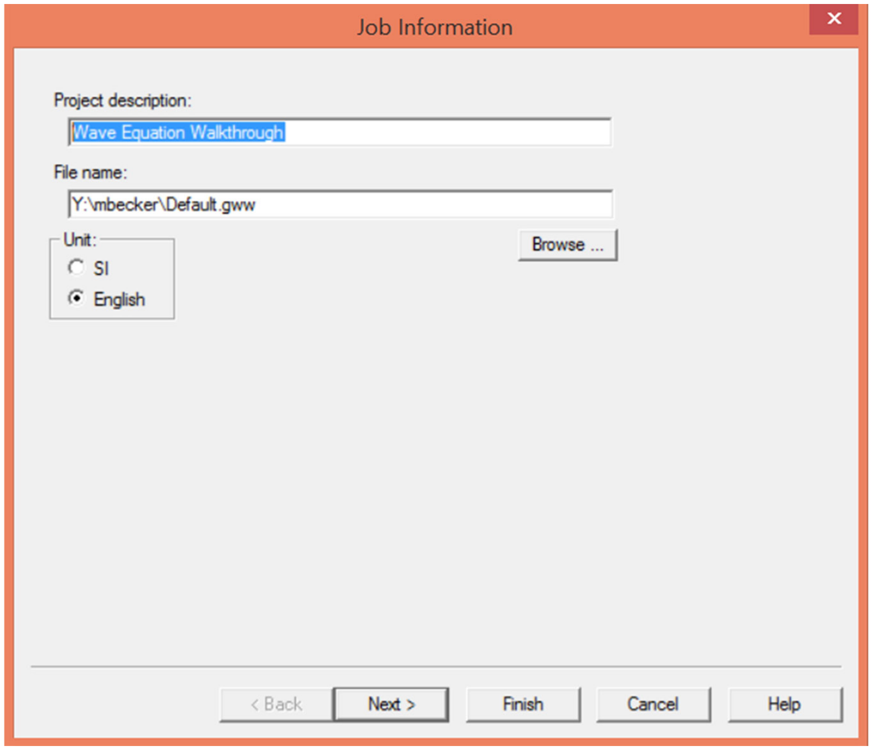


Figure 12-26 Job information window.

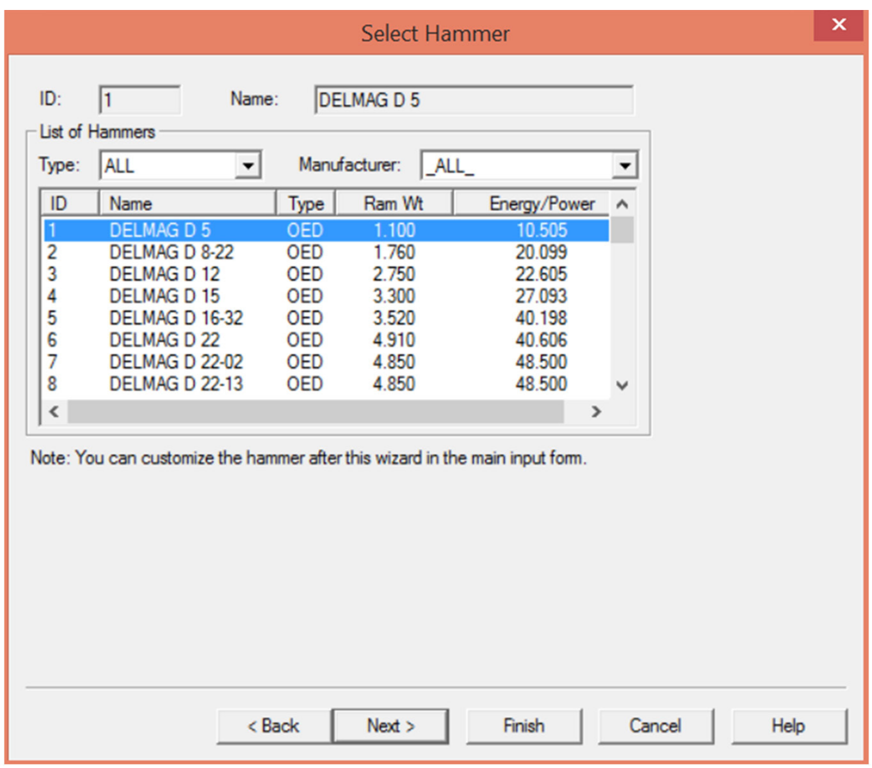


Figure 12-27 Select hammer window.

The next section involves the **Analysis Type** window, displayed in Figure 12-28. For a simple **Bearing Graph**, the **Proportional Shaft Resistance** option is the default. It assumes constant percentages of shaft resistance and end bearing for all nominal resistance values to be analyzed. The alternate bearing graph options analyze the various nominal resistance values either assuming a **Constant Shaft Resistance** or a **Constant End Bearing**.

A modified Bearing Graph approach, the **Inspector's Chart** provides the possibility of analysis with an increasing stroke (or hammer energy values) for a single nominal resistance value. This option is useful for diesel hammers, whose stroke can vary and/or be adjusted by different fuel settings, and for hydraulic hammers, whose energy level can be selected on the hammers' control panel.

The user may also choose the **Drivability** option. It requires as an input the unit shaft resistance and unit toe resistance as a function of pile penetration and, therefore, requires an accurate static soil analysis. The resulting output will show the corresponding nominal resistance values together with calculated penetration resistance (blow count), pile stress maxima, and other quantities and, thus, indicates the complete, expected driving behavior.

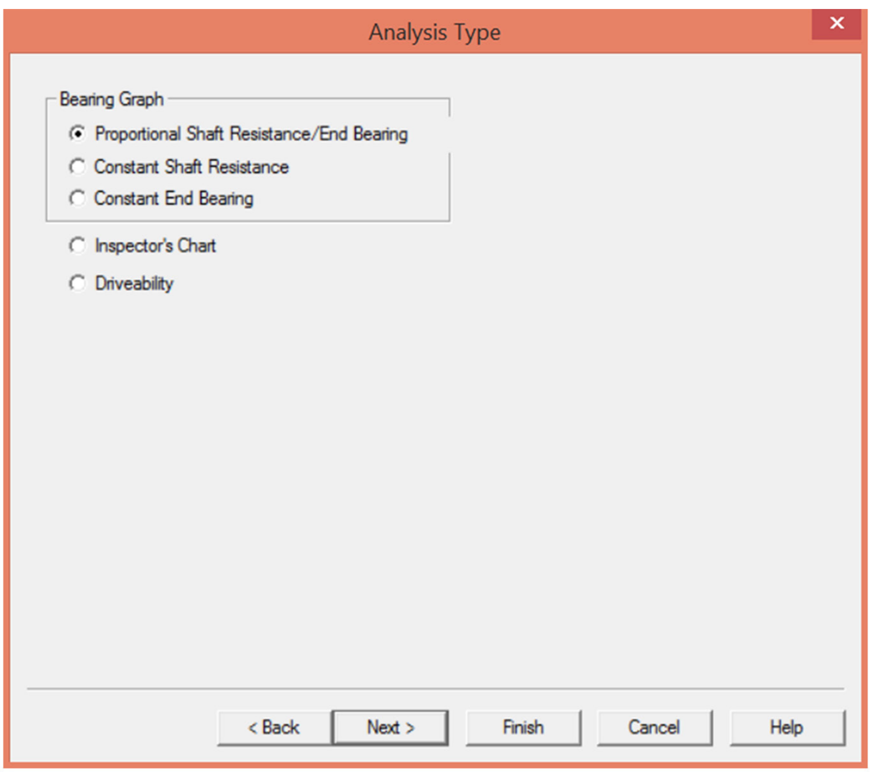


Figure 12-28 Analysis type window.

Clicking on **Next** brings up the **Pile Input** window, illustrated in Figure 12-29. After selection of the pile material, i.e., **Concrete**, **Steel**, or **Timber**, the program inputs default values for pile top elastic modulus, coefficient of restitution, and specific weight in the corresponding fields and also, for concrete pile material, activates the pile cushion input section. As with the hammer cushion, described below, the user may utilize the **Area Calculator** and the **Cushion Material Properties** Help by pressing the **F3** function key or directly input a stiffness. Additionally, selection of the pile material will automatically select the pile damping parameter which is accessible through **Options**, **General Options**, **Damping**. The user may adjust the aforementioned defaults but must enter the initial inputs for the Pile Length and cross Section Area. For the latter, the user may again employ the **Area Calculator**, shown in Figure 12-30, which also provides the Pile Size, Perimeter and Toe Area based on pile type and pile dimensional information. These quantities are particularly important for the drivability analysis when calculating the nominal resistance from the unit resistance values.

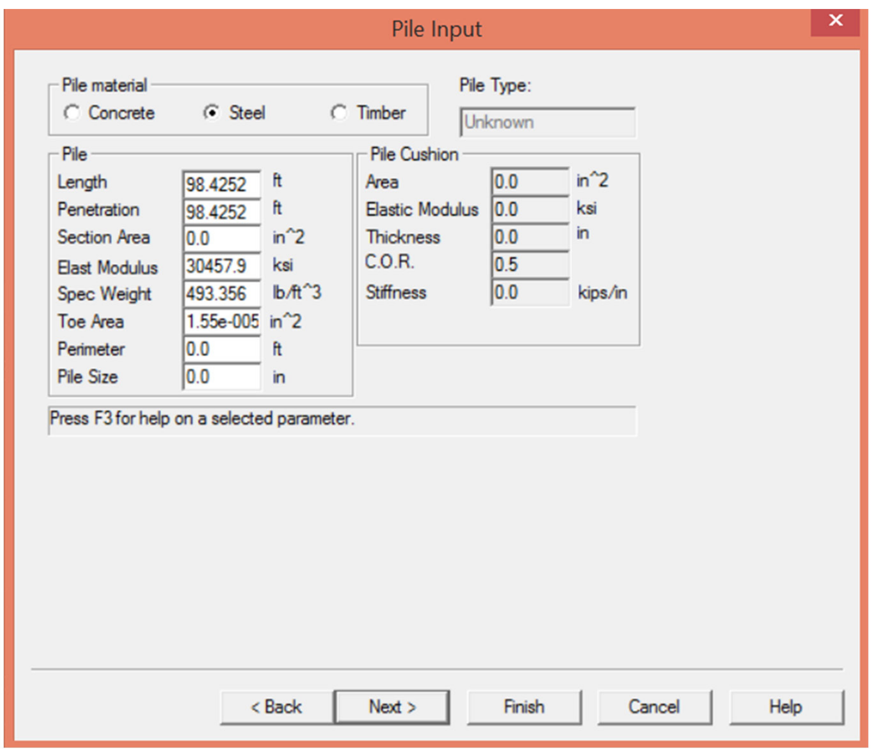


Figure 12-29 Pile input window.

It is important to note that for non-uniform piles the input quantities of Cross **Sectional Area**, **Elastic Modulus**, **Spec**ific **Weight** and **Perimeter**, in this window only refer to the pile top. Also **Toe Area** and **Pile Size** may need correction for non-uniform piles, the latter because it is used for calculating a recommended toe quake.

Once the data entry wizard has been finished, the non-uniform quantities must be entered in the P1 window, accessible after clicking on the pile type drop-down menu. As form most default values, those automatically chosen for pile elastic modulus and specific weight may or may not be correct and must be reviewed by the program user. For example, for concrete or timber piles, measurements could indicate other values. Pressing **F3** with the cursor on the Elastic Modulus or Specific Weight input field brings up added Help information. The following information is required information.

Length is the total pile length in the leads in feet. For example, if plans require a pile of 50 feet in length but the contractor is driving 60 long piles, the proper analysis length would be the full 60 feet. If pile sections are spliced together to form a longer pile, an analysis before and after splicing may be of interest. In such cases, the Length may be either the length of a single section before splicing or the combined length after splicing.

Penetration refers to the analyzed pile toe penetration below grade in feet for Bearing Graph or Inspector's Chart and final penetration for Drivability Analyses. This measurement must use the same soil grade reference as that of the soil resistance distribution.

Section Area is the pile cross section area at the pile head in $inch^2$.

Elastic Modulus is the elastic modulus of the pile material at the pile head in ksi.

Spec Weight is the weight per unit volume of the pile material at the pile head in lbs/ft^3. This value is used for calculating the mass of the pile material by division with 32.17 ft/s^2. The program provides for a modification of the pile's weight component by modification of the gravity acceleration value in Options/General Options/Numeric to reflect, for example, the effect of buoyancy or pile inclination on the pile weight on the soil.

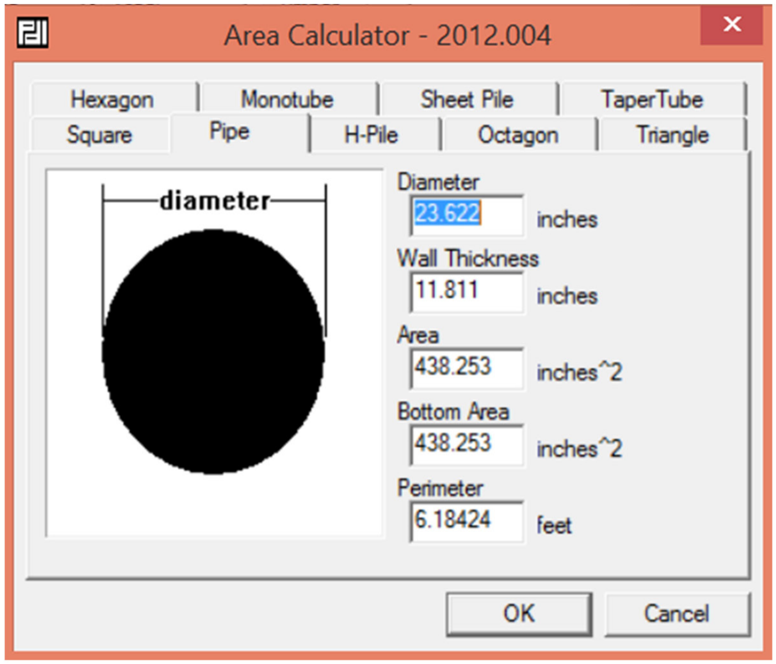


Figure 12-30 Area calculator window

After the pile input is done, clicking on **Next** will open up the Hammer Cushion window, as shown in Figure 12-31. GRLWEAP offers an extensive data file for those situations in which the contractor's available equipment is unknown. The data file has been made possible courtesy of the various manufacturers and dealers whose products are listed. Please note that this file is neither complete nor necessarily appropriate for all situations, as the contractor may not follow the manufacturer's recommendations. The required information consists of:

Area is the area of the hammer cushion perpendicular to the load in inch^2.

Elastic Modulus is the elastic modulus of the hammer cushion material in ksi.

Thickness of the hammer cushion. For sandwiched cushions, this is the thickness of the cushion material that corresponds to the elastic modulus in inches. If the entire stack thickness is entered, the combined elastic modulus of the sandwich and the striker plate is not included. If no hammer cushion exists, leave this value and the stiffness value at zero.

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

Stiffness of the hammer cushion in kips/inch. Use of this optional input will override the inputs for area, elastic modulus, and thickness.

Helmet Weight, consisting of the combined weight of the helmet, hammer cushion, striker plate, inserts, and all other components located between the hammer and pile in kips. The input may be zero if there is no helmet mass.

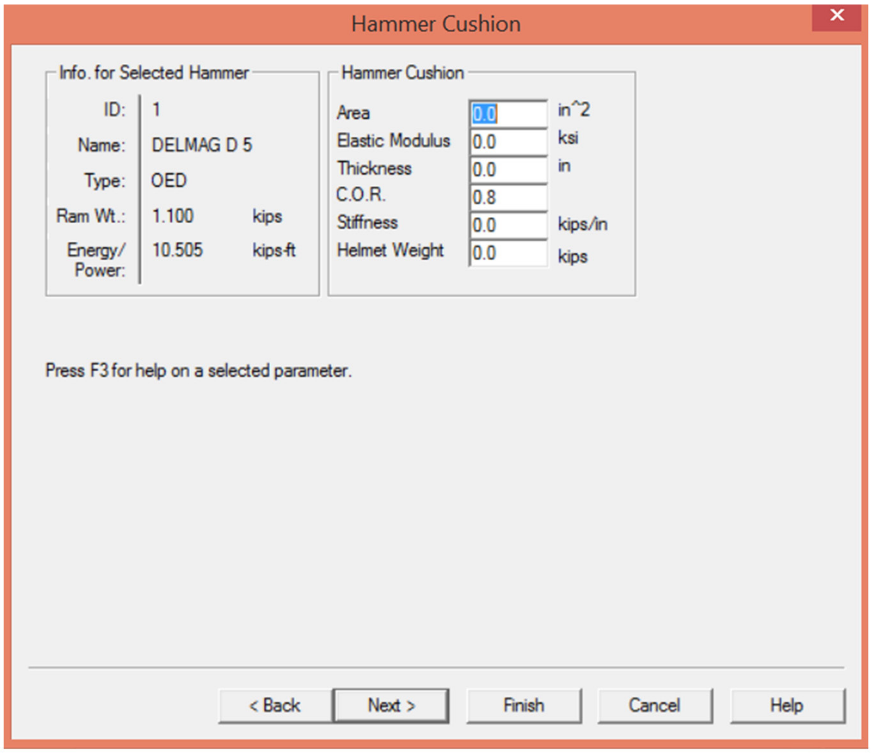


Figure 12-31 Hammer cushion window.

Ideally, the contractor would provide the above drive system data for his actual hammer system. However, if not available, the required hammer cushion data may be selected using one of three different methods:

1. The hammer cushion Stiffness and Coefficient of Restitution may be known from other analyses and can be input directly into the appropriate fields. In such cases, hammer cushion area, elastic modulus, and thickness are not needed.

2. If some or all of the driving system data is to be retrieved from the program data file, merely pressing F3 while the cursor is on one of the associated input fields and then clicking on Manufacturer's Recommended Driving System opens a listing of the recommended input. The user may transfer the suggestions in whole or part to the input sheet.

3. If the cushion material area, thickness, and type are known but modulus of elasticity and Coefficient of Restitution are not, pressing F3 while the cursor is

on the elastic modulus field and then selecting Cushion Material Properties brings up a list of frequently used cushion materials and their properties. These values can be transferred directly to the hammer cushion data input fields.

The **Next** input sections for bearing graph or inspector's chart analyses may be done in the dynamic soil parameters window on the **Main Input Form** or **Soil Profile Input** window, displayed in Figure 12-32. (For Drivability analyses, the **S1 Form** is opened as later discussed.) The most convenient input is through the **ST** analysis in the **Soil Profile Input** window. There, the user first specifies the:

Number of Soil Layers. It is recommended to divide the soil into layers of not more than 10 feet in thickness for improved accuracy.

Final Penetration Depth is the distance from grade to that depth to which data is to be given in feet. The window will at first display the value entered under the pile information. However, it may be changed here with the exception that it cannot be greater than the pile length.

Water Table is the distance from grade in feet where the water table begins. If grade is underwater, enter zero or a negative value; the later will show on the Main Screen graphics the water depth. As far as buoyancy is concerned both zero and negative values give the same result.

Effective Overburden at Grade is the intensity of any overburden pressure in ksf. For example, in the case of an excavation of limited extent (trench), the depth of excavation times the soil unit weight equals the effective overburden.

For each layer, the analyst then enters:

Either the **Layer Bottom Depth** or the **Layer Thickness** in feet.

The layer soil type as either **Granular** (non-cohesive soil for primarily sandy or other coarse grained soils) or **Cohesive** (for clays and silts) and selects as sub types the density or consistency of the layer. For intermediate soil types or non-cohesive silts, it may be conservative to choose "cohesive", since soil damping is then assigned a higher value. However, under all circumstances, the analyst should review the results obtained from this very simplified analysis.

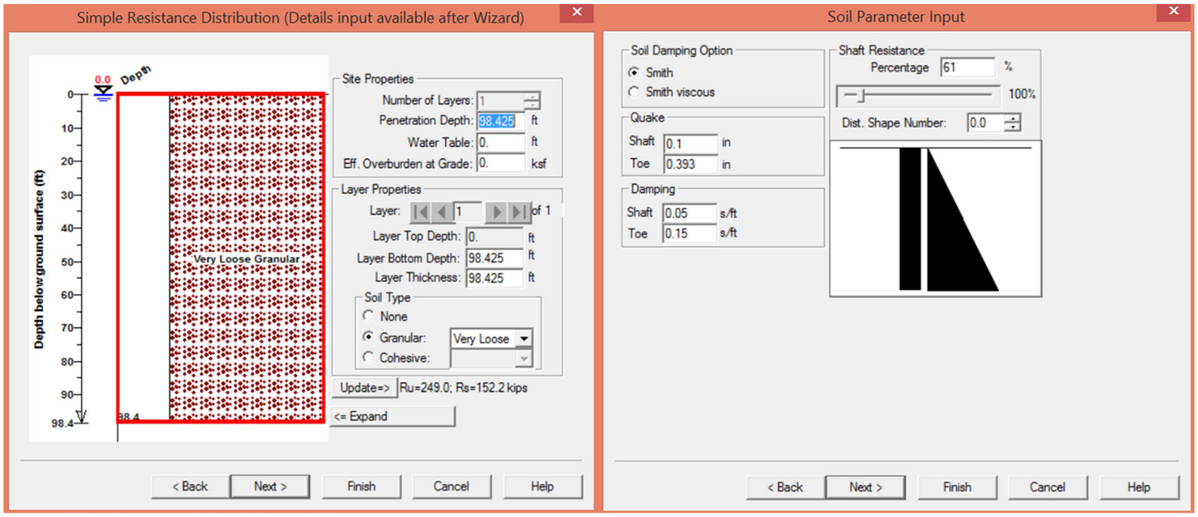


Figure 12-32 (a) Soil profile input window for soil type based static soil analysis and (b) Soil parameter input window for bearing graph analysis.

After clicking **Update**, the program will display a nominal resistance (**R_u**) and a nominal shaft resistance (**R_s**) value. These two results pertain to the **Final** penetration **Depth**, where the ratio **R_s/R_u** is the percentage of shaft resistance and one of the soil resistance inputs generated by the routine. Under no circumstances should these values be used for pile design purposes. The results are based on the following two methods:

For Non-Cohesive Soils

Using the Effective Stress Method, the unit shaft resistance is $f_s = \beta\, \sigma'_v$, with β being the Bjerrum-Burland beta coefficient as tabulated in Table 12-8 and σ'_v being the effective vertical stress at the midpoint of a given soil layer. The unit toe resistance is $q_p = N_t\, \sigma'_{vt}$, where N_t is a toe bearing capacity coefficient (see Table 12-8) and σ_{vt} is the effective overburden pressure at the pile toe. Both **f_s** and **q_p** are subjected to certain specified limits.

For Cohesive Soils

For cohesive soils, **ST** applies a modified α-method, also called the total stress method, and relies on the unconfined compression strength (q_u) of the soil layer. The q_u-value and, based on it, the unit shaft and unit toe resistance values are shown as a function of both soil type and a representative N-value in Table 12-9.

Table 12-8 Soil Parameters in ST Analysis for Granular Soil Types

Soil Type	SPT N	Friction Angle ϕ	Unit Weight pcf	β	N_t	Max. Unit Shaft Res. ksf	Max. Unit End Bearing ksf
Very Loose	2	25 - 30	86	0.203	12.1	0.5	50
Loose	7	27 - 32	102	0.242	18.1	1.0	100
Medium	20	30 - 35	118	0.313	33.2	1.5	150
Dense	40	35 - 40	125	0.483	86.0	2.0	200
Very Dense	50+	38 - 43	141	0.627	147.0	4.o	400

Table 12-9 Soil Parameters in ST Analysis for Cohesive Soil Types

Soil Type	SPT N	Unconfined Compression Strength ksf	Unit Weight pcf	Max. Unit Shaft Res. ksf	Max. Unit End Bearing ksf
Very Soft	1	0.25	111	0.07	1.1
Soft	3	0.75	111	0.23	3.3
Medium	6	1.50	118	0.40	6.7
Stiff	12	3.00	131	0.81	14.0
Very Stiff	24	6.00	131	1.30	27.0
Hard	32+	8.00+	121-140	1.60	36.0

After the soil types of all layers have been entered, the program computes the percentage and distribution of shaft resistance, the average shaft damping parameter, and the toe quake. These wave equation input values are based on pile penetration, water table depth, pile size, pile perimeter, and pile toe area. Damping and quake values are adjusted considering soil and pile type. Again, the analyst is responsible for checking these values and should be aware that this analysis is not applicable to non-uniform piles.

It is very important that the user carefully reviews the wave equation input parameters resulting from this very simplified static soil analysis, possible in the **Soil Parameters Input** window (see Figure 12-32b). Particular attention should be paid to the pile toe area because the shaft resistance percentage and toe quake directly depend on its magnitude. Also, it is recommended to perform comparative analyses, for example, when the soil type does not clearly fall into either the cohesive or granular categories. In such cases, results for both soil types should be obtained and compared. The ST generated input parameters should be reviewed once the input wizard has been finished and the main screen is displayed.

Help pertaining to both soil type input and soil quakes and damping appears in the program Help Menu under GRLWEAP Input Forms, Soil Type-Based Input Form and GRLWEAP Component Parameters, Soil Parameters, respectively. It is also recommended that the user carefully review both the PDI (2010) Background Report and the program Help.

For Bearing Graphs or Inspector's Charts, the user must input between one and ten nominal resistance values in the **Ultimate Capacity** window shown in Figure 12-33. Several options are available including values spaced at constant increments (Incr.), generated by pressing "Interpolate" to interpolate between the first and last entries, and **Automatic Capacities**, based on the pile cross section properties. It is recommended to analyze nominal resistance values that will provide a meaningful bearing graph for both easy and hard driving conditions. The input wizard is now finished. The completed **Main Input** screen should resemble that shown in Figure 12-39. To perform a more complex analysis, additional inputs may be made by specifying a Non Uniform Pile or a more detailed soil resistance distribution in Variable Resistance Distribution.

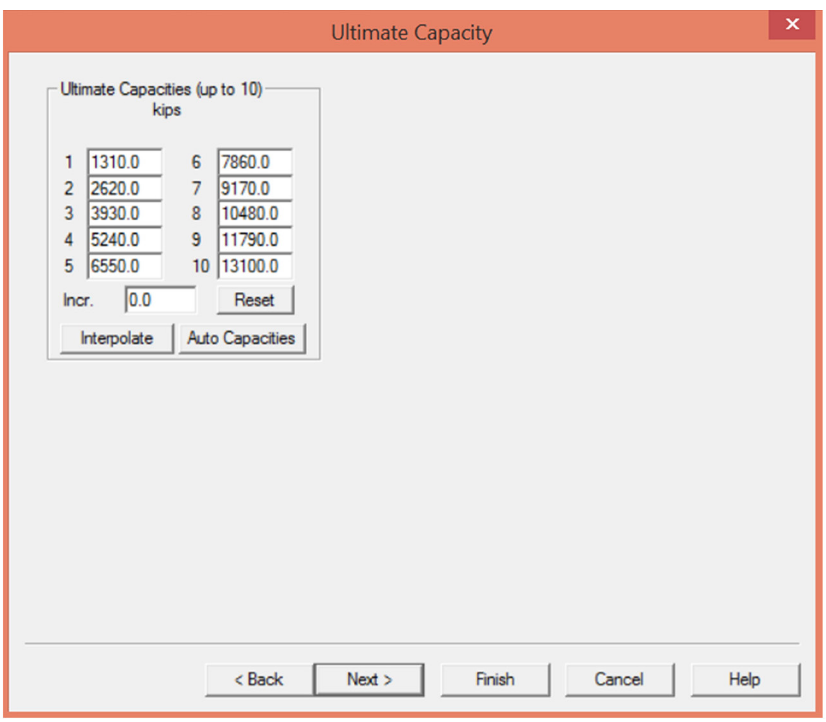


Figure 12-33 Window for entering up to 10 nominal resistance values for bearing graph and inspector's chart analyses.

For Drivability, instead of nominal resistance values, the analyst must input **Resistance Gain/Loss Factors**. Figure 12-34 shows the related window. The analyst may perform at most 5 analyses at each specified depth and provide at most five associated gain/loss factors for both the pile shaft and toe. These factors are related to the soil resistance parameters to be entered in the S1 Form (Figure 12-35) discussed below. A factor of 1.0 implies no change in soil strength during driving and thus that no resistance gain or loss will be analyzed. A factor less than 1.0 proportionally reduces the resistance values under consideration of their relative setup factors and thus reflects that the soil resistance is lower during driving and increases after pile installation, i.e., soil setup. A factor greater than 1.0 proportionally increases the resistance values and thus reflects the soil relaxation scenario, i.e., where the soil resistance is greatest during driving. In most cases, it is sufficient to enter two values for the shaft analysis. The first shaft value, marked 1, would be the inverse of the highest soil setup factor entered in the S1 Form and would represent the greatest resistance loss during driving along the shaft. The associated toe resistance factor, Toe 1, would be set to 1.0 to indicate neither gain nor loss of toe resistance during driving. For the second analysis, Shaft 2 and the associated factor Toe 2 would be set to 1.0. This latter input then reflects the absence of both gain and loss during driving at each depth analyzed (see Figure 12-34).

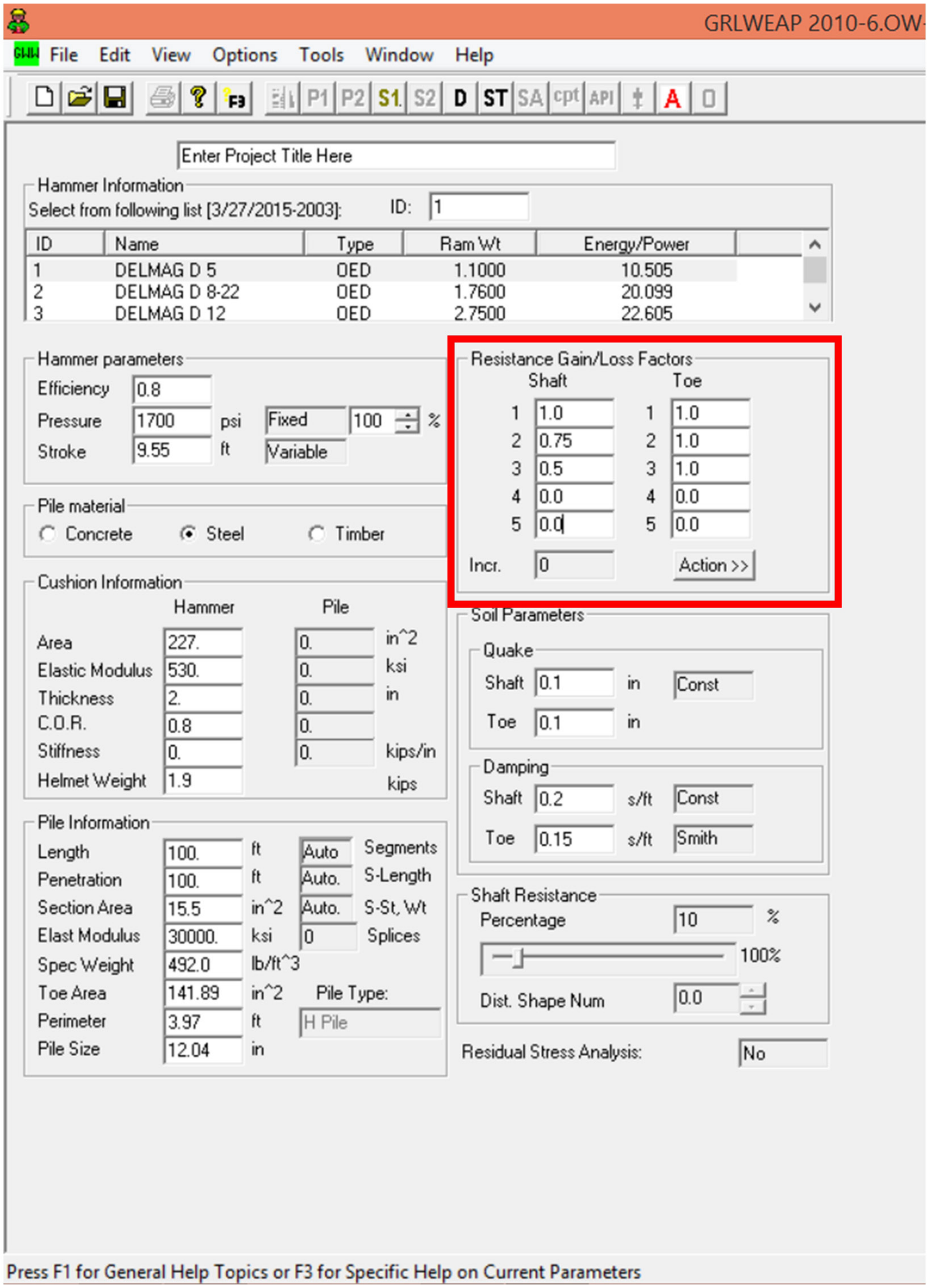


Figure 12-34 Resistance gain/loss factors in main input form for drivability analysis.

Next for Drivability analyses, the GRLWEAP program requires input in the S1 Form (Figure 12-35). Important inputs for each soil layer are (refer also to descriptions for the equivalent bearing graph inputs):

Depth is the soil layer distance below grade or mudline in feet.

Nominal Unit Shaft Resistance in ksf is determined by a static geotechnical analysis (e.g., the GRLWEAP SA routine). GRLWEAP multiplies this input by the pile perimeter, the segment length, and a soil layer specific gain/loss factor to yield the shaft resistance at the segment.

Nominal Unit Toe Resistance in ksf equals the unit toe resistance determined by a static geotechnical analysis.

Skin quake is the shaft quake in inch, usually left at the default value of 0.10 inch.

Toe quake in inch is per the Soil Parameter Help.

Skin damping is the shaft damping in s/ft as per the Soil Parameter Help.

Toe damping in s/ft, usually left at the default value of 0.15 s/ft.

Setup factor is based on site specific knowledge and, in conjunction with the resistance gain/loss factor, determines for each soil layer the soil resistance to driving.

The parameters of Limit Distance and Setup Time allow for a qualitative evaluation of soil strength change during driving interruptions, providing for more detailed analyses of splice time interruptions. These parameters have no influence on results as long as entered **Wait Times** in the **Depths, Modifiers Input Form** (see Figure 12-38) are zero.

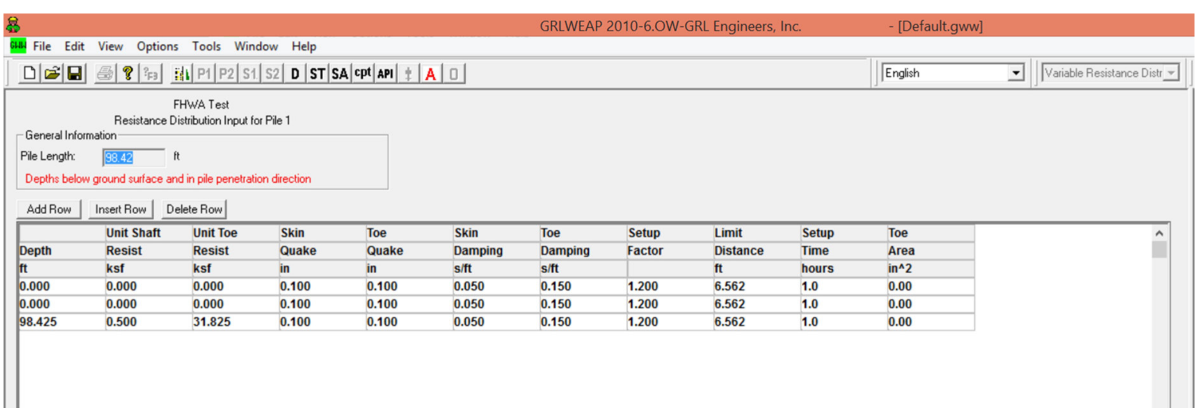


Figure 12-35 S1 form for soil resistance vs. depth input.

Should the user utilize the static analysis (SA) routine that is built into the GRLWEAP program to complete the S1 Form, clicking the SA icon will open the Figure 12-36 window. First, selection of Profile and New allows specification of the following quantities in the Static Analysis General Information window (refer to Figure 12-36):

The **Total Number of Soil Layers** to be included between grade and a depth equal to the total pile length in feet.

The depth of the **Water Table** in feet.

Overburden Pressure in ksf; see also ST above for an explanation of these inputs.

After closing the **Profile** window, the soil layer specific input can be made in the **SA** window. The following information should be provided (see Figure 12-37):

Layer Bottom Depth or **Layer Thickness** is in ft.

If the SPT N-value is known, choose from Gravel, Sand (with sub types indicating Grading and Grain Size), Silt, Clay, or Rock. Then enter the SPT N-value (not greater than 60), and the program will calculate a unit resistance and a unit weight for the soil layer.

If the SPT N-value is unknown, choose Other and either Cohesive or Cohesionless and then provide the Unit Weight in kips/ft^3 and the Unit Shaft Resistance and Unit Toe Resistance, both in ksf. The program will reduce the input unit weight value below the water table to yield an effective overburden.

Upon user request, the SA routine will also fill in the Other Parameters, i.e., the input values for quake, damping, setup factor, limit distance and setup time columns. Oftentimes these values do not differ from the default values specified earlier and then can be left at zero. Again, the user should carefully review the automatically generated input values prior to performing the actual wave equation analysis.

The SA routine is basically an effective stress method with different approaches for sand, silt and clay. This method is described in detail in the Background Report of the GRLWEAP program (PDI, 2010) and that description should be reviewed prior to using this approach. It applies very conservative limits on the unit resistance values which may be non-conservative as far as drivability analyses is concerned (calculated driving resistances and stresses may be low). Also the method is only applicable to piles with straight sections (not applicable to tapered piles) and should never serve as the sole static soil analysis method for a pile design. In fact, it is always prudent to compare several static analysis methods for an assessment of the range of possible results.

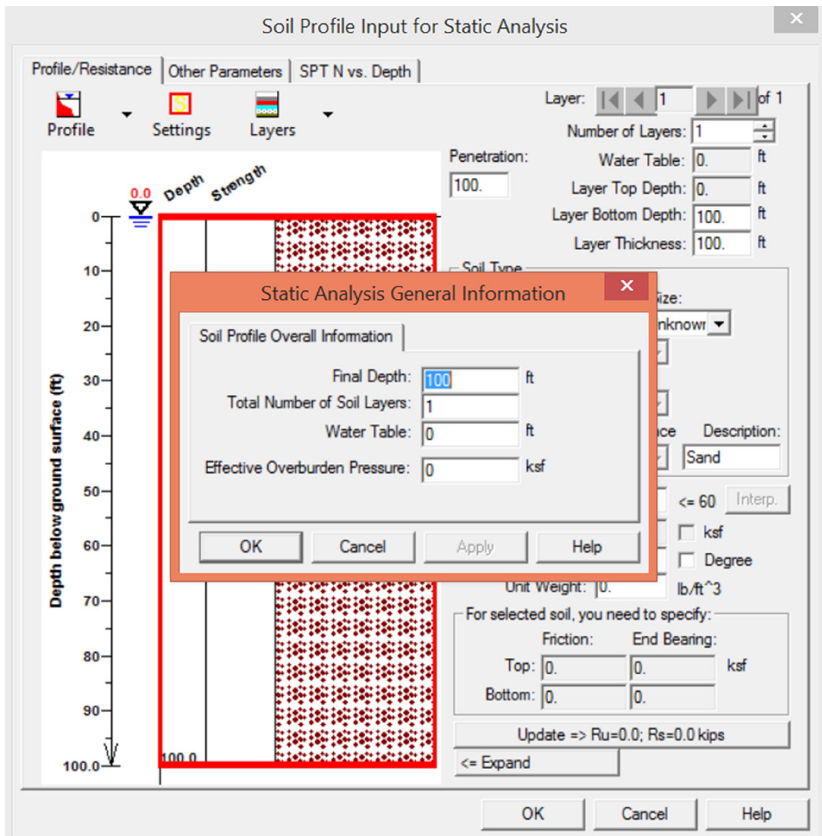


Figure 12-36 New profile window for the SA static analysis.

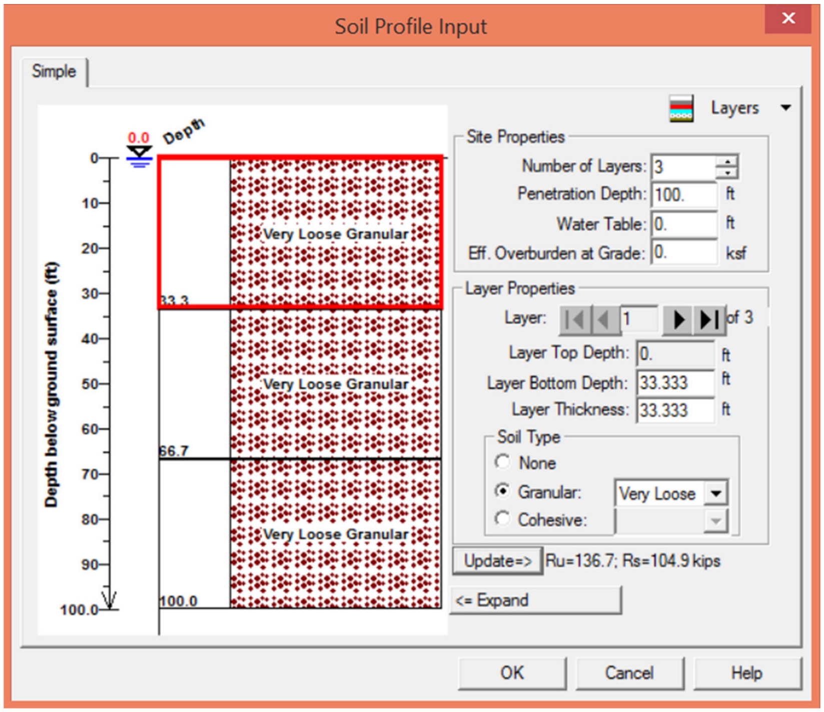


Figure 12-37 Soil property input window for the SA static analysis.

In a Drivability analysis, another required input is found in the **Depths**, **Modifiers Input Form**, accessible after clicking the **D** icon (see Figure 12-38). This form provides for the input of:

Depth to be analyzed in feet is a required input for at least two different depth values.

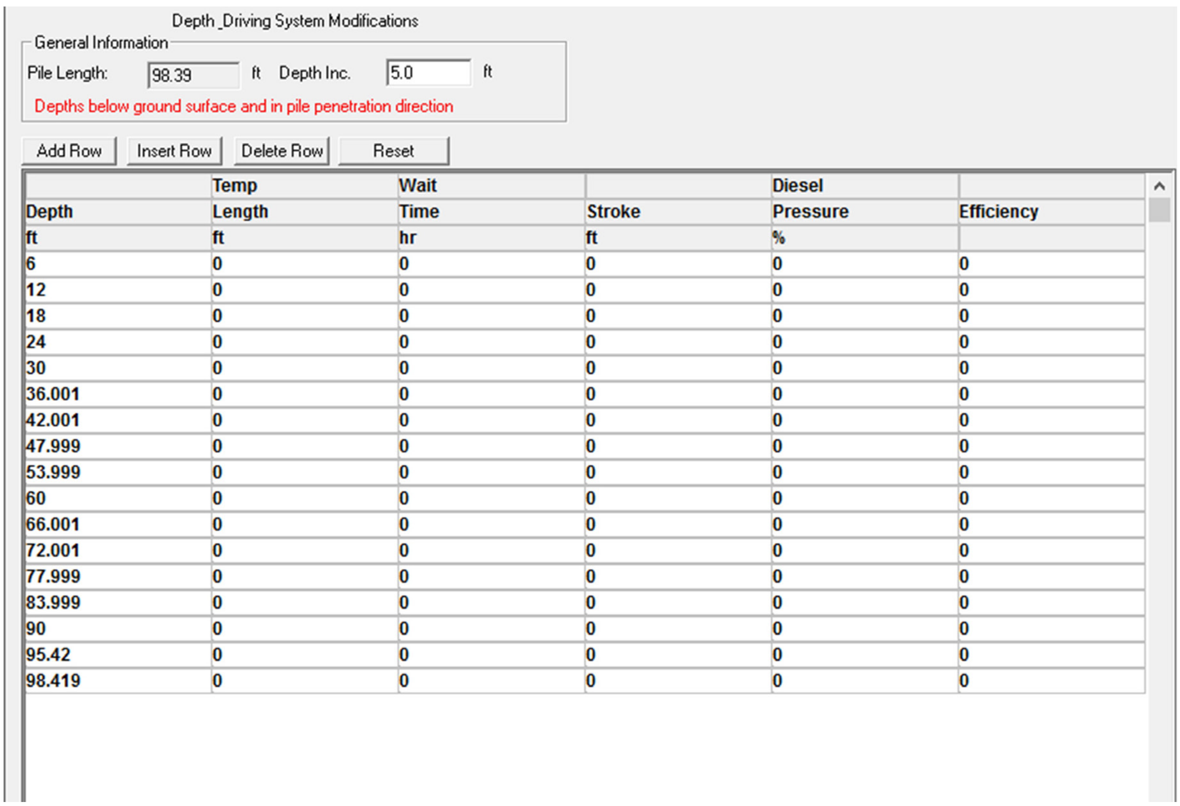
The pile **Temporary Length** in feet may be less than or equal to the length value given as the final length input and allows for consideration of a reduced pile length prior to splicing.

Wait Time in hours, which would be applicable if driving were to be interrupted, for example, for splicing operations. This input is only useful for a qualitative assessment of setup effects during the driving interruption, which, in turn, is a function of the Limit Distance and the Setup Time. Also it only applies to the first gain/loss factor. (As mentioned, this is rarely needed for highway construction projects.)

Stroke and **Efficiency** allow for variation of these hammer parameters as a function of depth. If not specified, the values input previously will be considered.

Diesel Pressure input allows for a modification of the hammer setting and/or stroke.

Pile Cushion Coefficient Of Restitution (**COR**) or **Stiffness Factor** can also be varied as a function of depth. For example, and considering the recommendations for new and used pile cushion parameters, the Stiffness Factor may be gradually increased from 1.0 to 2.5 if the cushion elastic modulus was specified earlier with the "New" elastic modulus.


Depth_Driving System Modifications

General Information

Pile Length: 98.39 ft Depth Inc. 5.0 ft

Depths below ground surface and in pile penetration direction

Add Row | Insert Row | Delete Row | Reset

Depth	Temp Length	Wait Time	Stroke	Diesel Pressure	Efficiency
ft	ft	hr	ft	%	
6	0	0	0	0	0
12	0	0	0	0	0
18	0	0	0	0	0
24	0	0	0	0	0
30	0	0	0	0	0
36.001	0	0	0	0	0
42.001	0	0	0	0	0
47.999	0	0	0	0	0
53.999	0	0	0	0	0
60	0	0	0	0	0
66.001	0	0	0	0	0
72.001	0	0	0	0	0
77.999	0	0	0	0	0
83.999	0	0	0	0	0
90	0	0	0	0	0
95.42	0	0	0	0	0
98.419	0	0	0	0	0

Figure 12-38 Depths, modifiers input form.

12.7.1 Other Analysis Options

A variety of options exist in the GRLWEAP program for non-standard input, analyses, and output. The setting of important options is indicated on the Main Input screen (see Figure 12-39). Please refer to the program Help Menu for additional, less frequently used options.

Important options pertain to the modification of certain hammer parameter, pile model, and soil resistance input. These options are generally accessible by clicking

Options/General Options, or Hammer Parameters, or Pile Parameters, or Soil Parameters. For proper hammer modeling, the Efficiency and Stroke values contained in the hammer data file must often be modified. This modification can be done on the Main Input screen below the Hammer Information window. Alternatively, these and other hammer details may be modified by clicking on Options, Hammer Parameters. Relevant quantities are explained in the Help Menu and will not be further discussed here.

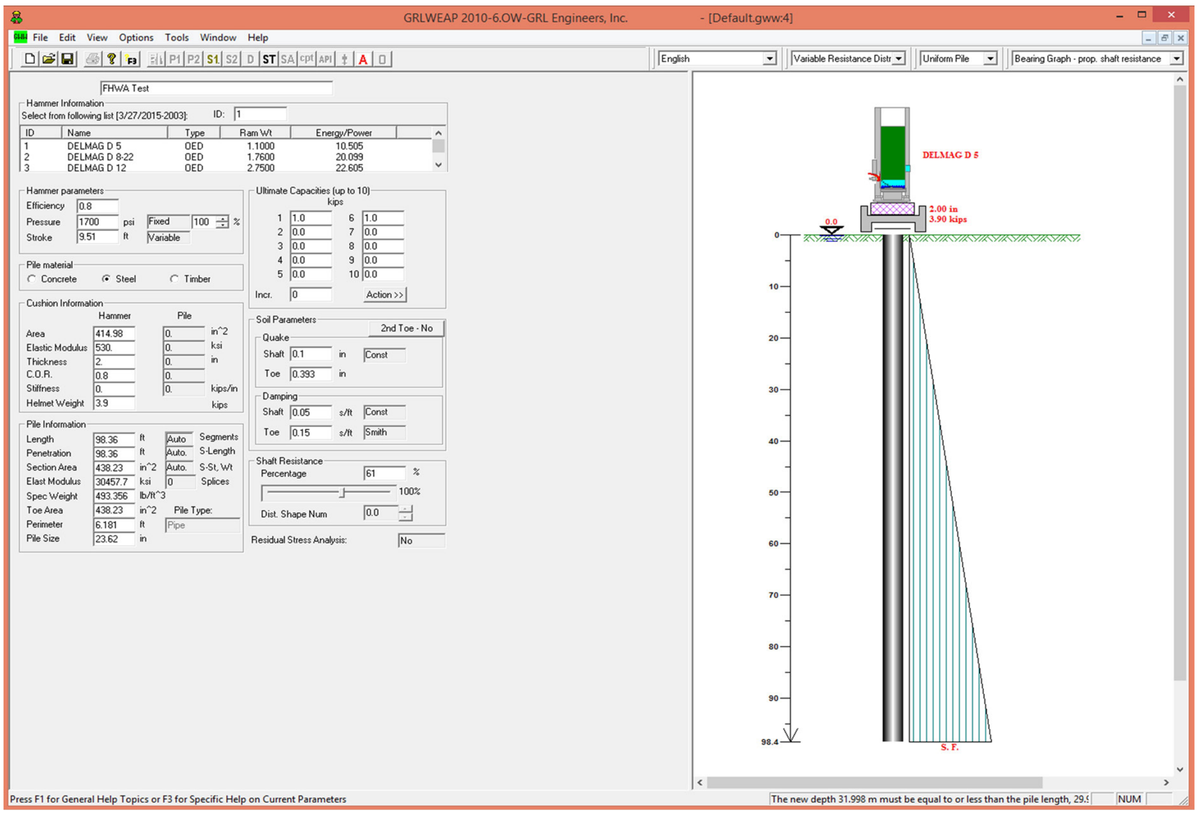


Figure 12-39 Completed main input for a simple bearing graph.

As mentioned, efficiency is a very important hammer parameter. The efficiency values in the hammer data file were selected according to the observed average behavior of all hammers of the same type. However, depending on a particular hammer's make or state of maintenance, the hammer may perform differently than assumed, and its parameters should be adjusted accordingly. Furthermore, because of uncertainties in actual hammer performance, greater and lesser efficiency values should be analyzed for conservative stresses and blow counts, respectively. Finally, efficiency should be adjusted for an inclined or batter pile which is simplified in the Options/Pile Parameters/Batter Inclination Input Window shown in Figure 12-40. Refer to the Help Menu for recommended efficiency reductions and further guidance for inclined pile driving.

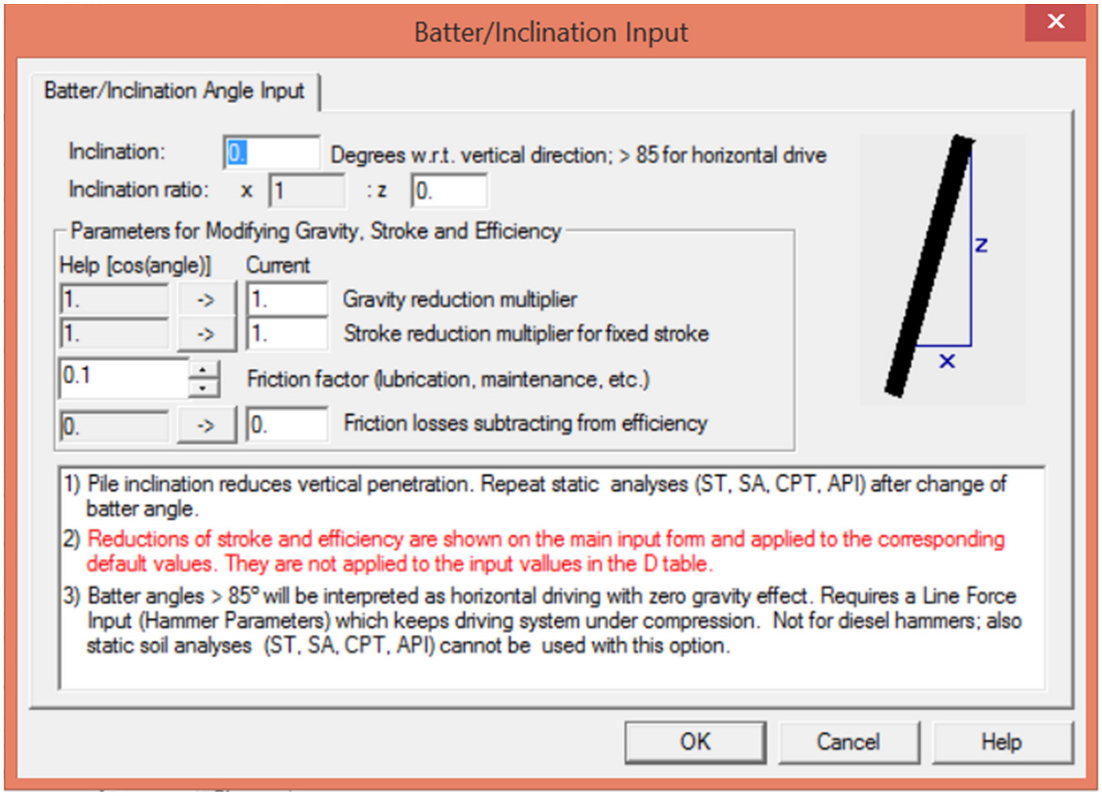


Figure 12-40 Batter/Inclination input window.

Stroke in feet is a useful performance parameter for single acting diesel hammers whose ram is visible and whose stroke is not equal to the rated value. For other hammers, energy level may be known, but since stroke is equal to energy divided by ram weight, stroke serves as an input for an adjusted hammer energy setting.

Pressure in psi is important for diesel hammers when the calculated and observed hammer strokes differ. A new pressure value may be tried for better agreement. Also, if the hammer is physically run at a reduced fuel setting, the pressure value should be reduced in the program accordingly.

Several options for **Pile Parameters** are available in GRLWEAP primarily for the purpose of flexibility in pile model generation. The status of many of these options is indicated under the Pile top Information on the Main Input Form.

The **Number of Pile Segments** may either be automatically set based on segment lengths of 3.3 feet, or the user may use a different number by clicking on **Options**, **Pile Parameters**, **Pile Segment Option**. Usually the program default of 3.3 feet segment lengths yields satisfactory accuracy. To avoid loss of this computational accuracy, only segments smaller than the default value should be entered. In the Pile Segment Option, the user can also modify the relative length of the segments (the information field marked S-Length would then be set to Man. for manual) and enter the segment stiffness and mass values. In the latter case, the information field marked S-ST, W_t would be set to manual.

Some piles are spliced with devices that allow for slippage during extension or compression. In Options/Pile Parameters/Splices, the user can choose the Number of Splices to be modeled and, after clicking on Update, edit the entry fields shown in Figure 12-41. For each splice, the user enters the Distance in feet of the splice location referenced from the pile head, the Tension Slack in feet, i.e. the distance that the splice can extend without transmitting a tension force, the C.O.R. or coefficient of restitution for the spring representing the spliced section, and the Compression Slack in feet, i.e., the distance that the splice can compress with a spring stiffness which increases linearly from zero to its loading value. The Main Input Form indicates the selected number of splices; the graphic of the hammer, driving system, pile, and soil model on the Main Input Form also indicates a splice with a slight gap in the pile representation. Note that neither an uncracked welded splice of a steel pile nor a well performing epoxy splice of a concrete pile requires slack modeling. These splices do not allow for slippage and, therefore, should be modeled as a uniform pile section and not as a splice with a slack.

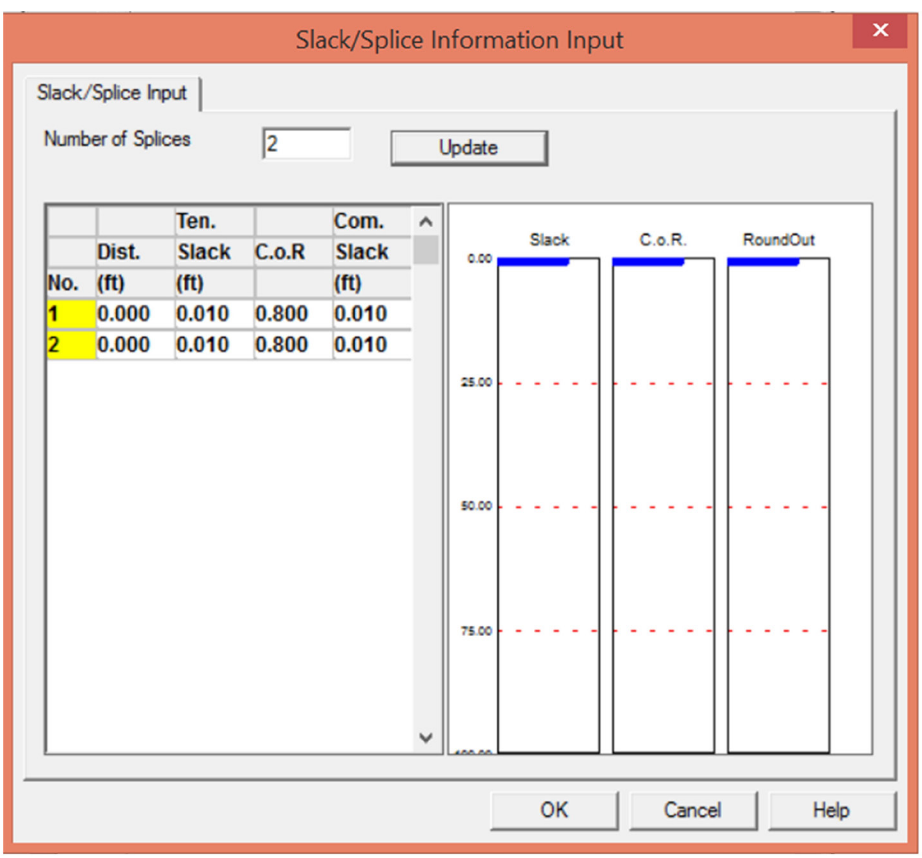


Figure 12-41 Slack/Splice information input window.

Non-uniform pile properties are modeled by selecting Non-Uniform Pile from the Pile Type drop-down menu which activates the P1 window. The user should complete the necessary information by specifying pile variations and adding the necessary number of rows immediately above and below a change of cross sectional area (**X-Area**), elastic modulus (**E. Modulus**), specific weight (**Spec. Wt.**), **Perimeter**, and Critical Index (**Crtcl. Index**) (see Figure 12-42). Note that Perimeter is needed for the computation of total shaft resistance in Drivability analyses. The Critical Index input is needed for the listing of critical rather than absolute maximum stresses in the result summary for piles consisting of materials of different strengths (for example, for a concrete pile with steel stinger, the concrete section may be more important to investigate for stresses then the steel stinger, even though the steel stinger may have stresses which are 10 times higher than the concrete stresses). Pile sections for which the Critical Index is set 1 are included in the search for a maximum tension and compression stresses for final output.

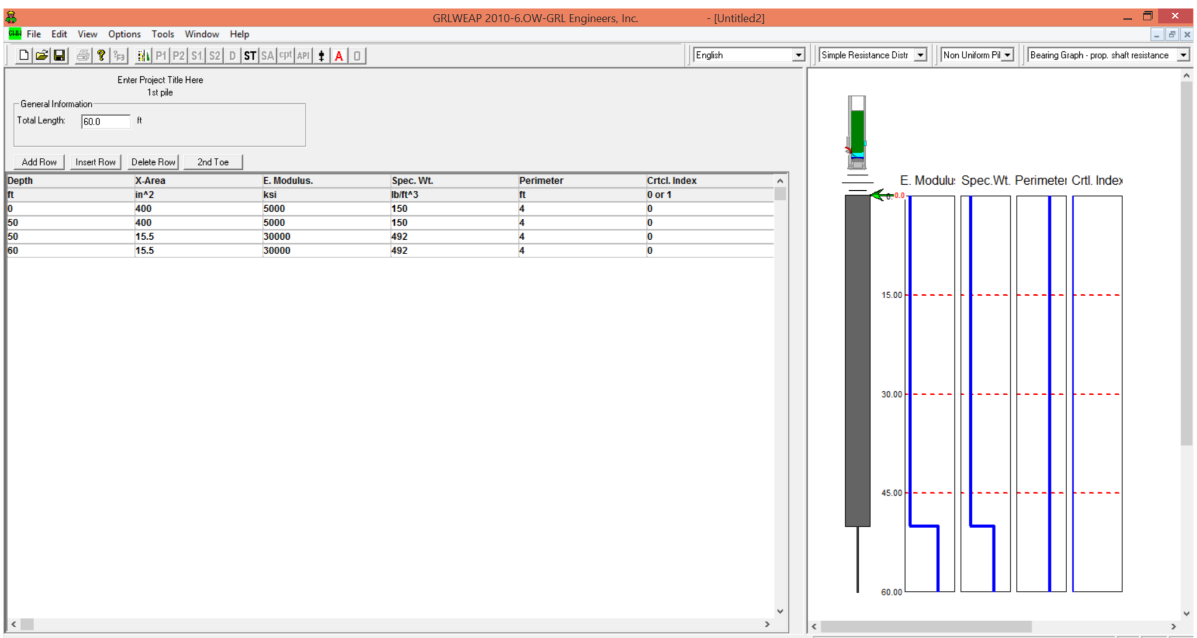


Figure 12-42 Data entry screen for non-uniform piles.

Several infrequently used options concerning primarily the pile model can be accessed in Options/General Options/Numeric (see Figure 12-43). An important but somewhat different pile option leading to an alternate type of analysis is the Residual Stress Analysis (RSA). The input number indicates the maximum number of repeat analyses (or blows) allowed, with a “1” representing the absolute limit of 100 cycles. Potentially important for large piles is the input of an adjusted Hammer and Pile Gravity. The default gravitational constant is 32.17 ft/s^2. The default value would represent a vertically driven pile above the water table. If the static weight is less due to either pile inclination or buoyancy, this value should be reduced using Options/General Options/Numeric. Note that the hammer and pile mass magnitudes will not be affected by this change of their gravitational constant.

Soil parameter options, like pile options, allow for increased input flexibility. Under Options/Soil Parameters, it is possible to enter individual nominal resistance values, damping, and quake values for each pile segment. Use of these options causes the corresponding field labels in the soil input section of the Main Input Form to read "Variable."

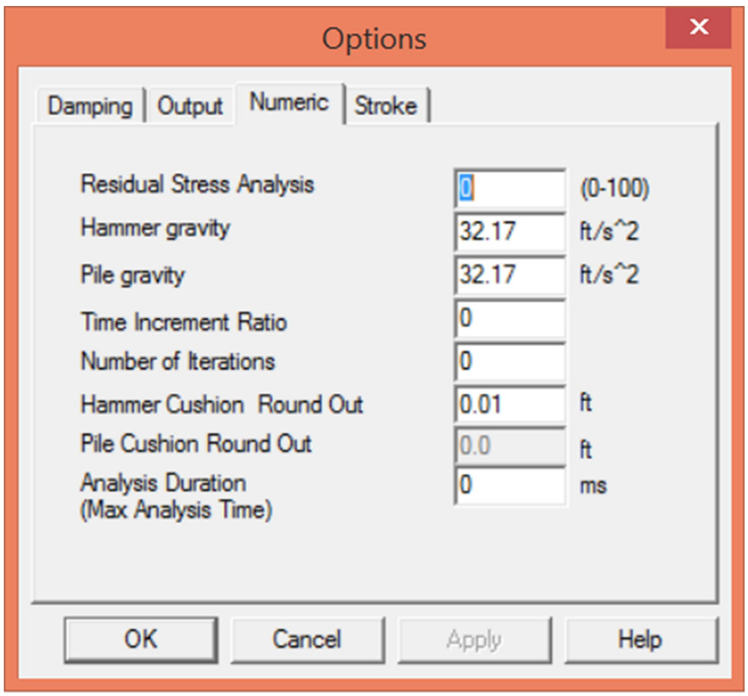


Figure 12-43 Numeric options window.

The **Damping Option**, accessible in Options/General Options/Damping (Figure 12-44), is rarely used for routine applications and is more useful for the researcher. In most instances, the Soil Damping is set to Smith damping. If the Residual Stress Analysis is performed or a vibratory hammer is analyzed then the Smith viscous damping option should be selected. Also Refined Wave Equation analyses sometimes have to done with Smith viscous damping for a good match. Hammer Damping and Pile Damping Options have been preset to a percentage of the impedance of the ram and hammer cushion and the pile, respectively, though the preset values may be replaced with small non-negative integers. Given a negative input, the program will read a zero value. For the pile material, the program automatically chooses values of 1, 2 and 5 for steel, concrete and wood, respectively. Another pile material (e.g., plastic piles) may require other, possibly higher inputs. While not enough is known about these parameters, their effect on the computed results is relatively insignificant.

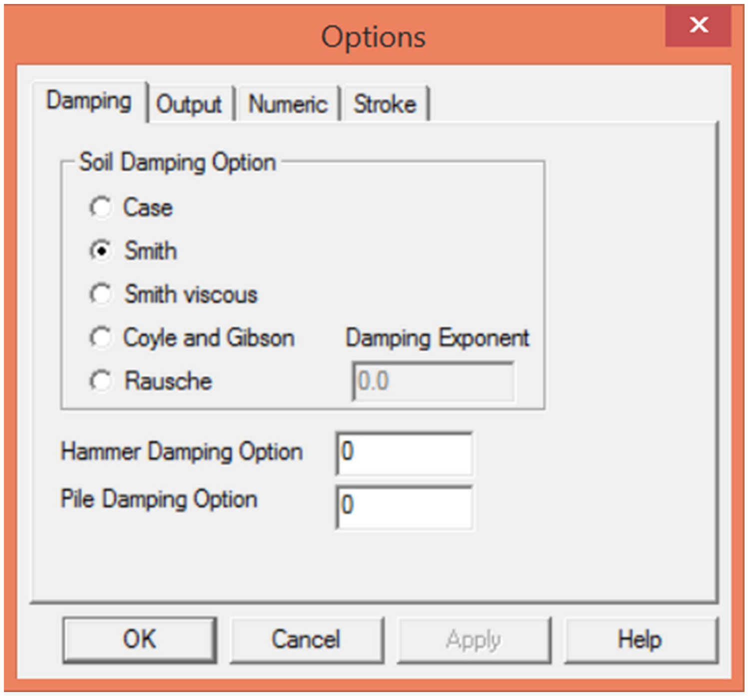


Figure 12-44 Damping options window.

The **Stroke Option**, important for diesel hammers, is accessible in Options/General Options/Stroke (see Figure 12-45). For any diesel hammer, the stroke is a function of fuel settings, pile mass and stiffness, and soil resistance. The stroke option allows the user to control whether the program will analyze a fixed stroke or calculate the stroke (default) based on the combustion pressure provided in either the hammer database or the user modified input. A fixed stroke can either be analyzed with an iteratively adjusted combustion pressure, such that upstroke equals down stroke, or with a single impact whose upstroke is then potentially different from its down stroke. On the Main Input Form below the Hammer Information window, this selected stroke option is identified. The selection of the Stroke Option is particularly important for Inspector's Chart analyses, and the reading of the associated Help is strongly recommended.

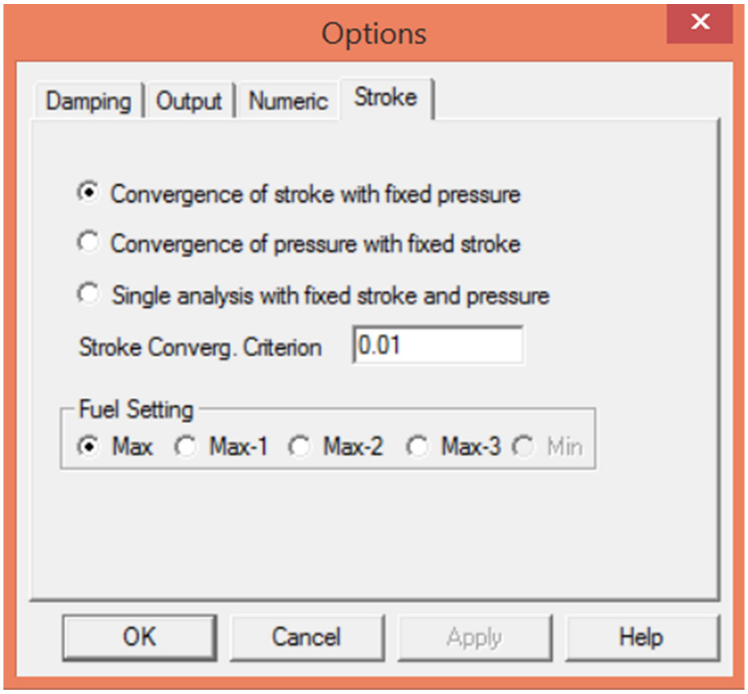


Figure 12-45 Stroke options window for diesel hammers.

The Stroke Options window also allows for a selection of Fuel Settings for those diesel hammers that have stepwise adjustable fuel pumps. Alternatively, the corresponding fractions of the maximum combustion pressures can be selected on the Main Input Form. As noted in Section 12.6.3, analyzing a hammer with a high combustion pressure, even if the high stroke is the result of pre-ignition, may lead to high calculated transferred energies and, therefore, non-conservative nominal resistance predictions and conservative stress predictions. On the other hand, if the observed hammer stroke is relatively low and friction (which should be modeled with a reduced hammer efficiency) has been eliminated as a reason for the low stroke, a reduced combustion pressure presents a reasonable analysis option. Because of the potential for an overprediction of nominal resistance due to excessive pressure adjustment, the Inspector's Chart option does not increase the combustion pressure above the value in the hammer data file despite the presence of any values for high stroke analyses in the "**Convergence of pressure with fixed stroke**."

12.8 WAVE EQUATION ANALYSIS OUTPUT AND INTERPRETATION

The GRLWEAP program offers several output options that may be invoked or modified using Options, General Options, Output as displayed in Figure 12-46. The box labeled Type allows for selection of certain variables (e.g., force, velocity, stress) at a number of different segments. The user may opt to plot these variables as a function of time or create a table for transfer to other programs. Of particular interest is the plotting of pile top force and proportional velocity vs. time for comparison with dynamic measurements.

The Numerical box underneath allows for the control of the numerical output in one of three means. Choosing Minimum (default for drivability analyses) will exclude the extrema tables that are included in the Normal output selection. The extrema tables are very helpful when investigating the location of maximum stress values, and even though they may make the output very long, it is often desirable to revert to the Normal option, even for drivability analyses. Another worthy candidate for the normal output is the multi-material pile. The final numerical option, Debug generates so much numerical output that it is rarely needed for real applications.

After an analysis has been run, clicking the **O** (**Output**) icon transfers control to the output program in which several output modes (depending on output and analysis options) of the Project Summary (containing several important parameters and title components) become available: **Bearing Graph** or **Drivability**, **Variables vs. Time** and **Numeric results**.

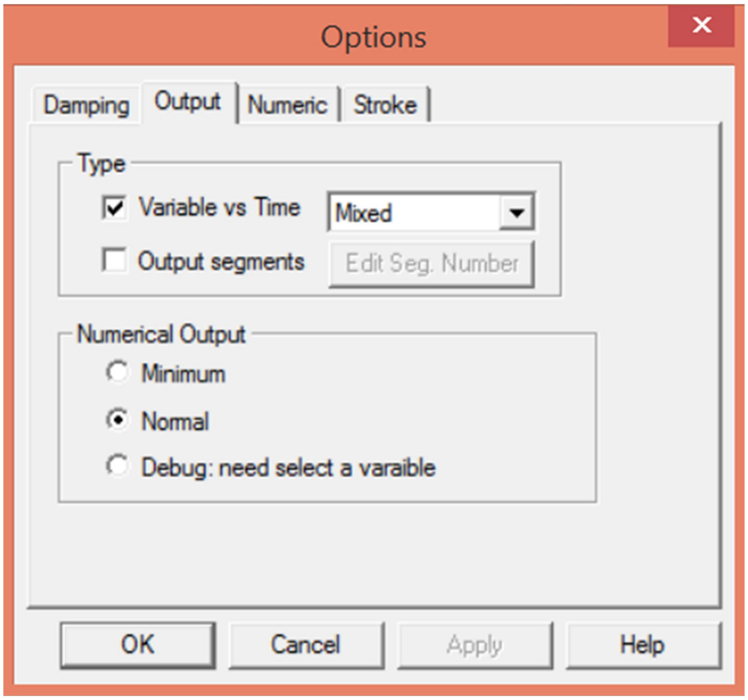


Figure 12-46 Output options window.

The Numerical GRLWEAP Output, or Numeric results, is the most important output. It begins with a listing of file names used for input and the input file (*.GWW). There follows a disclaimer pointing out some of the uncertainties associated with wave equation analyses. The user is urged to check that the correct data file is used and consider the disclaimer when drawing conclusions from analysis results.

The first page of output, shown in Figure 12-47, lists the hammer and drive system components used in the analysis. Hence hammer model, stroke, and efficiency, helmet weight, and 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.

```
               GRLWEAP: WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS
                                  Version 2010
                                 English Units

                       Example 1
                       ________________________________________

                Hammer Model:   D 12-42           Made by: DELMAG     DELMAG

                No.    Weight    Stiffn       CoR      C-Slk    Dampg
                         kips    k/inch                   ft    k/ft/s
                  1     0.940
                  2     0.940   92080.7     1.000     0.0100
                  3     0.940   92080.7     1.000     0.0100
          Imp Block     0.617   53843.8     0.900     0.0100
          Helmet        3.900  109975.0     0.800     0.0100       5.5
          Combined Pile Top     50000.0

      HAMMER OPTIONS:
        Hammer File ID No.              38  Hammer Type                OE Diesel
        Stroke Option             FxdP-VarS  Stroke Convergence Crit.      0.010
        Fuel Pump Setting           Maximum

      HAMMER DATA:
        Ram Weight          (kips)     2.82  Ram Length          (inch)    103.50
        Maximum Stroke        (ft)    11.81
        Rated Stroke          (ft)    11.81  Efficiency                     0.800

        Maximum Pressure     (psi)  1730.00  Actual Pressure      (psi)   1730.00
        Compression Exponent          1.350  Expansion Exponent              1.250
        Ram Diameter        (inch)    11.81
        Combustion Delay       (s)  0.00050  Ignition Duration      (s)   0.00200

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


        HAMMER CUSHION                       PILE CUSHION
        Cross Sect. Area     (in2)   415.00  Cross Sect. Area     (in2)      0.00
        Elastic-Modulus      (ksi)    530.0  Elastic-Modulus      (ksi)       0.0
        Thickness           (inch)     2.00  Thickness           (inch)      0.00
        Coeff of Restitution            0.8  Coeff of Restitution             0.0
        RoundOut              (ft)      0.0  RoundOut              (ft)       0.0
        Stiffness        (kips/in) 109975.0  Stiffness        (kips/in)       0.0
```

Figure 12-47 Hammer model, hammer options, and driving system output.

The second page of output, presented in Figure 12-48, 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, perimeter, material strength (normally 0 for uniform piles), wave speed, and pile impedance (EA/C). A detailed summary of the pile and soil model follows the pile profile. The detailed pile model includes the number of pile segments, their weight and stiffness, and any compression slacks (C-Slk) or tension slacks (T-Slk) with associated coefficient of restitution (C.O.R.). The listing also shows segment bottom depth (LbTop) and the averages of both segment circumference and cross sectional area. The summarized soil model 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 resistance, are summarized below in Figure 12-48.

On the third page, shown in Figure 12-49, an extrema table is printed for each pile segment number. This extrema output (Table 12-10) is printed for each analyzed nominal resistance and includes:

Table 12-10 GRLWEAP Extrema Output

Abbreviation	Description
No	Pile segment number
mxTForce	Maximum tension (negative) force in kips
mxCForce	Maximum compression force in kips
mxTStrss	Maximum tension (negative) stress in ksi
mxCStrss	Maximum compression stress in ksi
max V	Maximum velocity in ft/s
max D	Maximum displacement in inches
max Et	Maximum transferred energy in kip-ft

The "t" values following the extreme values are times in milliseconds relative to hammer impact. Note that tension is shown as a negative in these tables. For the analysis of diesel hammers, the iteration on hammer stroke is indicated beneath the extrema table information followed by the maximum combustion pressure analyzed in psi.

For bearing graph analyses, GRLWEAP concludes by printing a summary table for all input nominal resistances (ultimate capacities) after the extrema table listing. The summary table is illustrated in Figure 12-50 and includes the analyzed nominal resistance (ultimate capacity, R_{ut}) and corresponding penetration resistance (Bl Ct) for blow count, analyzed hammer stroke (for diesel hammers, both the down stroke and the rebound stroke), maximum tension stress (negative, Ten Str), maximum compression stress (Comp Str), maximum transferred energy (ENTHRU), and, for diesel hammers, hammer operating speed (Bl Rt). The indicators "i t" locate where (pile segment number) and when (time after impact in m/s) the extreme stress values occur.

Example 1 07/21/2015

GRLWEAP Version 2010

PILE PROFILE:

Toe Area	(in2)	438.250	Pile Type	Pipe
Pile Size	(inch)	23.620		

L b Top	Area	E-Mod	Spec Wt	Perim	C Index	Wave Sp	EA/c
ft	in2	ksi	lb/ft3	ft		ft/s	k/ft/s
0.0	400.00	5000.	150.0	6.7	1	12426.	160.9
50.0	400.00	5000.	150.0	6.7	1	12426.	160.9
50.0	53.00	30000.	492.0	4.0	0	16807.	94.6
60.0	53.00	30000.	492.0	4.0	0	16807.	94.6

Wave Travel Time 2L/c (ms) 9.237

Pile and Soil Model Total Capacity Rut (kips) 1310.0

No.	Weight	Stiffn	C-Slk	T-Slk	CoR	Soil-S	Soil-D	Quake	LbTop	Perim	Area
	kips	k/in	ft	ft		kips	s/ft	inch	ft	ft	in2
1	1.389	50000	0.010	0.000	0.85	2.5	0.050	0.100	3.33	6.7	400.0
2	1.389	50000	0.000	0.000	1.00	7.4	0.050	0.100	6.67	6.7	400.0
3	1.389	50000	0.000	0.000	1.00	12.3	0.050	0.100	10.00	6.7	400.0
4	1.389	50000	0.000	0.000	1.00	17.3	0.050	0.100	13.33	6.7	400.0
5	1.389	50000	0.000	0.000	1.00	22.2	0.050	0.100	16.67	6.7	400.0
6	1.389	50000	0.000	0.000	1.00	27.1	0.050	0.100	20.00	6.7	400.0
7	1.389	50000	0.000	0.000	1.00	32.1	0.050	0.100	23.33	6.7	400.0
8	1.389	50000	0.000	0.000	1.00	37.0	0.050	0.100	26.67	6.7	400.0
9	1.389	50000	0.000	0.000	1.00	41.9	0.050	0.100	30.00	6.7	400.0
10	1.389	50000	0.000	0.000	1.00	46.9	0.050	0.100	33.33	6.7	400.0
11	1.389	50000	0.000	0.000	1.00	51.8	0.050	0.100	36.67	6.7	400.0
12	1.389	50000	0.000	0.000	1.00	56.7	0.050	0.100	40.00	6.7	400.0
13	1.389	50000	0.000	0.000	1.00	61.7	0.050	0.100	43.33	6.7	400.0
14	1.389	50000	0.000	0.000	1.00	66.6	0.050	0.100	46.67	6.7	400.0
15	1.389	50000	0.000	0.000	1.00	71.5	0.050	0.100	50.00	6.7	400.0
16	0.604	39750	0.000	0.000	1.00	76.5	0.050	0.100	53.33	4.0	53.0
17	0.604	39750	0.000	0.000	1.00	81.4	0.050	0.100	56.67	4.0	53.0
18	0.604	39750	0.000	0.000	1.00	86.3	0.050	0.100	60.00	4.0	53.0
Toe						510.9	0.150	0.393			

22.644 kips total unreduced pile weight (g= 32.17 ft/s2)

22.644 kips total reduced pile weight (g= 32.17 ft/s2)

PILE, SOIL, ANALYSIS OPTIONS:

Non-uniform pile		Pile Segments: Automatic	
No. of Slacks/Splices	2	Pile Damping (%)	1
Pile Penetration (ft)	100.00	Pile Damping Fact.(k/ft/s)	3.219
% Shaft Resistance	61		
Soil Damping Option	Smith		
Max No Analysis Iterations	0	Time Increment/Critical	160
Output Time Interval	2	Analysis Time-Input (ms)	0
Output Level: Variable vs Time			

Figure 12-48 Pile, soil, and analysis options.

```
Example 1                                                    07/21/2015
                                                     GRLWEAP Version 2010

          Rut=  1310.0, Rtoe =    510.9    kips, Time Inc. =0.060 ms
 No mxTForce   mxCForce   mxTStrss    mxCStrss    max V     max D      max Et
      kips       kips        ksi         ksi       ft/s      inch      kip-ft
  1       0.0     1483.8       0.00        3.71      9.01     0.186       13.31
  2    -149.2     1477.7      -0.37        3.69      8.95     0.182       13.21
  3    -210.2     1473.1      -0.53        3.68      8.87     0.178       13.03
  4    -210.1     1466.5      -0.53        3.67      8.79     0.174       12.80
  5    -229.3     1454.5      -0.57        3.64      8.69     0.170       12.50
  6    -248.2     1440.8      -0.62        3.60      8.56     0.165       12.15
  7    -257.0     1424.4      -0.64        3.56      8.44     0.160       11.74
  8    -276.0     1402.0      -0.69        3.50      8.28     0.157       11.30
  9    -291.3     1381.2      -0.73        3.45      8.13     0.158       10.83
 10    -318.5     1354.3      -0.80        3.39      7.94     0.160       10.36
 11    -325.8     1327.0      -0.81        3.32      7.77     0.162        9.86
 12    -332.8     1295.1      -0.83        3.24      7.58     0.164        9.27
 13    -345.0     1256.7      -0.86        3.14      7.50     0.167        8.60
 14    -328.4     1179.9      -0.82        2.95      7.85     0.169        7.84
 15    -287.4     1010.2      -0.72        2.53      8.74     0.169        6.87
 16    -311.8      789.1      -5.88       14.89      9.36     0.166        5.82
 17    -248.1      629.8      -4.68       11.88     10.46     0.166        4.74
 18    -107.0      422.3      -2.02        7.97     10.78     0.164        4.19
Activated Capacity  1012.2 k
(Eq) Strokes Analyzed and Last Return  (ft):
11.81 10.43 10.42

Max. Combustion Pressure  1730.0 psi
```

Figure 12-49 Extrema table output.

```
Example 1                                                         07/21/2015
                                                          GRLWEAP Version 2010

    Rut   Bl Ct   Stroke  (ft)  Ten Str   i    t Comp Str   i    t ENTHRU   Bl Rt
   kips    b/ft    down     up      ksi              ksi             kip-ft   b/min
  655.0  1209.6    9.76   9.75    -1.21   4   22     3.51   1    2    12.4    38.2
 1310.0  9999.0   10.43  10.42    -0.86   0    0     3.71   0    0    13.3    36.9
```

Figure 12-50 Final summary for bearing graph analysis.

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

12.9 PLOTTING OF WAVE EQUATION RESULTS

The summary table results are usually presented in the form of a bearing graph relating the nominal resistance (ultimate capacity) to the pile penetration resistance. Plotting can be done by program built-in plotting routines or by saving the copying Numerical Output data to the clipboard and then pasting it in some other plotting program. The GRLWEAP program provides for the plotting of:

- one or two bearing graphs in the same plot,
- an Inspector's Chart,
- drivability results (e.g., blow count, nominal resistance, stresses, stroke, and transferred energy vs. depth) for one or two gain/loss factors, and
- the plotting of selected variables vs. time (e.g., forces, velocities, displacements etc.)

The wave equation bearing graph or inspector's chart should be provided for the resident construction engineer, pile inspector, and the contractor.

12.10 SUGGESTIONS FOR PROBLEM SOLVING

Table 12-11 summarizes some of the field problems that can be solved through use of wave equation analysis. Field problems may arise due to soil, hammer, driving system, and pile conditions that are not as anticipated or unknown. Of course, all possibilities cannot be treated in this summary. Sometimes, the performance of the wave equation program may produce an unexpected or apparently useless result and a corrective action may be required. A number of such problems together with suggested solutions are listed in Table 12-12. Further information may also be found in PDI (2010) and in the program's Help Menu.

Table 12-11 Suggested Use of the Wave Equation to Solve Field Problems

Problem	Solution
Concrete pile spalling or slabbing near pile head.	Perform wave equation analysis; find pile head stress for observed blow count and compare with allowable stresses. If high calculated stress, add pile cushioning. If low calculated stress, investigate pile quality, hammer performance, hammer-pile alignment.
Concrete piles develop complete horizontal cracks in easy driving.	Perform wave equation analysis; check tension stresses along pile (extrema tables) for observed blow counts. If high calculated tension stresses, add cushioning or reduce stroke. If low calculated tension stresses, check hammer performance and/or perform measurements.
Concrete piles develop complete horizontal cracks in hard driving.	Perform wave equation analysis; check tension stresses along pile (extrema table). If high calculated tension stresses, consider heavier ram. If low calculated tension stresses, take measurements and determine quakes, which may be higher than anticipated.
Concrete piles develop partial horizontal cracks in easy driving.	Check hammer-pile alignment since bending may be the problem. If alignment appears to be normal, tension and bending combined may be too high; solution as for complete cracks.
Steel pile head deforms or timber pile top mushrooms.	Check helmet size/shape; check steel strength; check evenness of pile head, banding of timber pile head. If okay, perform wave equation and determine pile head stress. If calculated stress is high, reduce hammer energy (stroke) for low blow counts; for high blow counts different hammer or pile type may be required.
Unexpectedly low blow counts during pile driving.	Investigate soil borings; if soil borings do not indicate soft layers, pile may be damaged below grade. Perform wave equation and investigate both tension stresses along pile and compression stresses at toe. If calculated stresses are acceptable, investigate possibility of obstructions / uneven toe contact on hard layer or other reasons for pile toe damage.

Table 12-11 Suggested Use of the Wave Equation to Solve Field Problems (Continued)

Problem	Solution
Higher blow count than expected.	Review wave equation analysis and check that all parameters were reasonably considered. Check hammer and driving system. If no obvious defects are found in driving system, field measurements should be taken. Problem could be pre-ignition, preadmission, low hammer efficiency, soft cushion, large quakes, high damping, greater soil strengths, or temporarily increased soil resistance with later relaxation (perform restrike tests to check).
Lower blow count than expected.	Probably soil resistance is lower than anticipated. Perform wave equation and assess soil resistance. Perform restrike testing (soil resistance may have been lost due to pile driving), establish setup factor and drive to lower nominal resistance. Hammer performance may also be better than anticipated, check by measurement.
Diesel hammer stroke (bounce chamber pressure) is higher than calculated.	The field observed stroke exceeds the wave equation calculated stroke by more than 10%. Check that hammer was set to correct setting. 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 and assure that pre-ignition is not a problem, *e.g.*, by measurements.
Diesel hammer stroke (bounce chamber pressure) is lower than calculated.	The field observed stroke is less than 90% of the stroke calculated by the wave equation. Check that hammer was set to correct setting. 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 12-12 Wave Equation Analysis Problems

Problem	Solution
Cannot find hammer in data file.	Contact the hammer manufacturer or the author(s) of the wave equation program. Pile Dynamics, Inc., for example, regularly updates and posts its hammer data file on its web page. Alternatively, the user may utilize a hammer of same type and of similar ram weight and energy rating and modify its data to match the unlisted hammer's specifications as closely as possible.
Cannot find an acceptable hammer to drive pile within driving stress and penetration 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 nominal resistance may be chosen lower than required. Nominal resistance 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 the cushion thickness/hammer energy setting combination yields acceptable stresses with minimum cushion thickness. Specify that the corresponding cushion thickness and hammer fuel setting be used in the field and their 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 12-12 Wave Equation Analysis Problems (Continued)

Problem	Solution
Calculated driving times are 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 are 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, diesel hammer combustion pressure, external combustion hammer stroke, coefficients of restitution, hammer cushion properties for steel piles, pile cushion properties for concrete piles, and pile top properties for timber piles. Resistance distribution also may affect the pile top forces.

12.11 ADVANTAGES, DISADVANTAGES, AND LIMITATIONS

One of the primary advantages of wave equation analysis is the ability to evaluate the drivability and suitability of a pile foundation design early in the design process. This allows economic evaluations on pile section selection to be easily and rationally decided based on predicted driving stresses and penetration resistances. For the contractor, wave equation analysis provides one of the best methods for equipment selection, determining the need for pile installation aids, and selecting pile cushion thickness. For the engineer, wave equation analysis provides the best method for hammer approval. No other computational tool for the simulation of the dynamic pile driving event is able to match the convenience and realism of this approach. Once the necessary information for program input is in hand, (closest boring, proposed driving system, and foundation details), a wave equation analysis can be performed relatively quickly.

A disadvantage of wave equation analysis is the information needed on the soil conditions, pile and foundation details, and hammer system for the analyst to make informed program input selections. A dynamic formula is simpler and quicker, albeit also less accurate, as the only input information needed for formula use is generally the hammer energy. The program user should have knowledge of the software

program, soil mechanics, and pile design and construction to understand and correctly apply results in unique and specific applications.
Wave equation limitations include, in some soil conditions, conservative estimates of the nominal resistance. Open end pipe piles or H-piles in granular profiles often behave differently under dynamic and static loading conditions. Under dynamic conditions, a plug may slip and produce additional shaft resistance. However under static loading, a plug may form over the full cross section resulting in soil resistance developing on a larger toe area. The wave equation model in this scenario may suitably predict the static behavior. The nominal resistance determined from wave equation analysis can be unconservative at low blow counts (less than 30 blows per foot) and overly conservative at very high blow counts (greater than 120 blows per foot).

12.12 PRACTICAL ISSUES AND CONSIDERATIONS

When confronted with the task of performing drivability analyses for a major bridge project, the question is often how many different analyses have to be performed to adequately represent the various pile, nominal resistance and soil conditions. Performing too many different analyses is not only time consuming it also may lead to confusions.

Nominal resistance determination by wave equation analysis or dynamic testing in soils with unknown long term behavior requires restrike testing. However, if a restrike test is performed, then it would add little cost to do dynamic testing at the same time and take advantage of a higher resistance factor. Thus wave equation based nominal resistance determination is only practical for small jobs where experience with similar conditions exist so that no restrike testing is necessary. In that case the use of the Inspector's Chart is a valuable tool. For open end diesel hammers, this would, however, require that the hammer stroke is accurately monitored. Also in that case, care should be taken to check whether or not there are unusual changes in stroke and/or blow count vs. depth behavior which could, for example, be indications of an overheating hammer.

When analyzing the driving long concrete piles with a large number of hammer blows, it is important to consider that pile cushion replacement should be done after approximately 1500 hammer blows. A drivability analysis has to properly reflect the pile cushion replacement by a change of stiffness. It is not advisable to schedule such a cushion change shortly before the end of driving where a new cushion would cause a low energy transfer and therefore an inflated blow count.

When specifying a blow count criterion two considerations are important: if the end of penetration resistance is low because of a large hammer, check to see if the hammer energy can be sufficiently reduced to provide a driving criterion above 30 blows/ft. Considering soil setup and allowing for a criterion with reduced nominal resistance may lead to a situation where the pile driving hammer is incapable to producing a blow count less than 120 blows/ft during a restrike. If nominal resistance verification by dynamic methods is important, then a larger hammer (e.g., a drop hammer) should be provided for the restrike test. Alternatively, the pile driving hammer has to be chosen large enough to produce restrike blow counts below 120 blows/ft.

When driving to a hard rock surface it would be unwise to specify the required penetration resistance in blows/foot. Instead it is recommended to specify a maximum pile penetration under a certain number of blows, for example, not more than ½ inch under 10 hammer blows. Bearing graphs and Inspector's Charts can be provided with this alternative penetration resistance measurement.

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CHAPTER 13

DYNAMIC FORMULAS

13.1 BACKGROUND

Engineers have attempted to find rational methods to determine the geotechnical resistance of a driven pile as long as they have been used for structure foundations. Initially, prediction methods were proposed using pile penetration observations made during driving. However, the only realistic measurement that could be obtained during driving was the pile set per blow. Thus energy concepts equating the potential energy of the hammer to the penetration resistance of the pile (set per blow) as it is driven through the soil were developed to estimate the geotechnical capacity or nominal pile resistance. In equation form this can be expressed as:

$$\mathrm{W\,h} = R_n\, s_b \qquad \text{Eq. 13-1}$$

Where:

- W = ram weight.
- h = ram stroke.
- R_n = nominal 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. Numerous dynamic formulas have also been proposed over time and some include consideration of pile weight, energy losses in drive system components, pile temporary compression, and other factors. Whether simple or more complex dynamic formulas are used, the nominal resistances determined by dynamic formulas have generally shown poor correlations and wide scatter when statistically compared with the nominal resistances determined by static load test results.

AASHTO (2014) LRFD Bridge Design Specifications include two dynamic formulas; the FHWA modified Gates formula, discussed herein in Section 13.3.1, and an AASHTO modified version of the Engineering News formula, discussed further in Section 13.3.2. Both the AASHTO design and construction specifications state a dynamic formula should not be used when the nominal resistance exceeds 600 kips.

13.1.1 Historical Accuracy of Dynamic Formulas

Wellington proposed the popular Engineering News formula in 1892. It was developed for evaluating the nominal resistance or capacity of timber piles driven primarily by drop hammers in sand. Concrete and steel piles were unknown at that time, as were many of the pile hammer types and hammer sizes used today. Therefore, it should be of little surprise that the formula performs poorly in predicting the capacity or nominal resistance of the pile foundations used today.

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 static 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 database was 91 tons. After reviewing the database, 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. A more detailed discussion of both the 1941 ASCE debate as well as the inadequacies of dynamic formulas can be found in Likins et al. (2012).

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 2/3 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.

As part of a FHWA research project, Rausche et al. (1996) compiled a database 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 signal matching 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 13-1 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 13-1. The closer the mean value of the ratio of the predicted/static load test result is to 1.0 and the smaller the coefficient of variation (COV) the more reliable the method.

Prediction method performance using driving observations of blow count and hammer stroke are identified as EOD for end of driving observations or BOR for beginning of restrike.

In the 1998 version of the FHWA pile manual, the database compiled by Rausche et al. (1996) was modified to include resistance predictions from the allowable load version of the Engineering News as well as the FHWA Modified Gates dynamic formulas at both the end of driving and beginning of restrike. The database for the dynamic formulas was also expanded and included additional data sets. The allowable load determined using the Engineering News formula in this study was compared to one half of the nominal resistance determined from the static load test, while the nominal resistance from the Modified Gates formula was compared directly to the nominal resistance determined from the static load test. The correlation results of the dynamic formulas are included in Table 13-1.

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 Modified 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 13-1 clearly shows the Engineering News formula has a tendency to overpredict capacity. The higher coefficient of variation also suggests that the Engineering News formula is significantly less reliable than the Modified Gates formula.

Table 13-1 also 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 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 Modified 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 database indicates an average factor of safety of 2.3 as compared to the factor of safety of 6.0 theoretically included in the allowable load version of 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 capacity. The fact that 12% of the database has a factor of safety of 1.0 or less is also significant. However, complete failure of a bridge due to inadequate geotechnical resistance determined by Engineering News formula is unusual. The problem usually is indicated by long term damaging settlements which occur after construction.

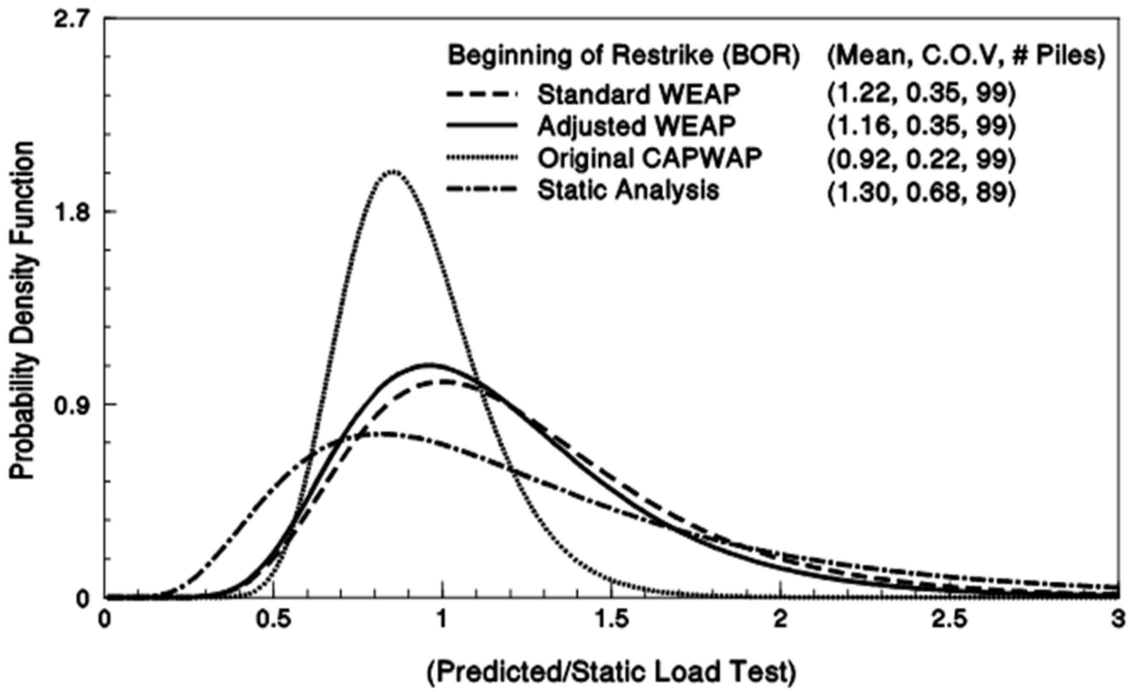

Figure 13-1 Log normal probability density function for four resistance predictions (after Rausche et al. 1996).

Dynamic formulas were historically used on small projects where conservative design load estimates, greater foundation redundancy, and resultant reserve foundation capacity helped mitigate some foundation performance problems. This hidden reserve resistance has been largely reduced in LRFD designs that utilize fewer, larger size piles, with higher nominal resistances.

The version of the Engineering News formula in the AASHTO (2014) LRFD Bridge Design Specifications is a modified from the historical allowable load version and calculates an ultimate capacity or nominal resistance. However, inherent problems with dynamic formulas remain as discussed in Section 13.1.2.

Table 13-1 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
Modified Gates Formula	EOD	0.96	0.41	139
Modified Gates Formula	BOR	1.33	0.48	122

* From Rausche et al. (1996)

EOD = End of Driving, BOR = Beginning of Restrike

13.1.2 Basic Limitations with Dynamic Formulas

Dynamic formulas have limitations, and are therefore less reliable than other field methods for nominal resistance verification. The basic limitations associated with pile driving formulas can be traced to the modeling of each component within the pile driving process: the driving system, the pile, and the soil. Dynamic formulas poorly represent 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 drivability, and further assume that the soil resistance is constant and instantaneous to the impact force. A more detailed discussion of these limitations is presented below.

The derivation of most formulas is not based on a realistic treatment of the driving system. Most formulas only consider the potential 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 determine the ability of the driving system to advance the pile into the soil.

Dynamic formulas also assume that the pile is 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. Compression waves which travel to the pile toe are responsible for advancing the pile into the ground.

The soil resistance is also very crudely treated by assuming that it is a constant force. This assumption neglects the 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 in no way similar to the static soil resistance. However, most dynamic formulas consider the resistance during driving equal to the nominal resistance or capacity, and do not consider the dynamic behavior of the soil during pile penetration. 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 soil 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 under static loads.

13.2 RESISTANCE FACTORS FOR DYNAMIC FORMULAS

Resistance factors applicable to the nominal axial geotechnical resistance determined from select dynamic formulas are provided in Table 13-2. The resistance factor varies depending on the dynamic formula and, in some cases, pile type. Only the FHWA modified Gates formula and AASHTO modified Engineering News formula have resistance factors in AASHTO. If a dynamic formula is used to establish driving criterion, AASHTO (2010) LRFD Bridge Construction Specifications Article 4.4.4.5 recommends use of the FHWA modified Gates formula. If a dynamic formula other than the FHWA modified Gates formula or AASHTO modified Engineering News formula is used, AASHTO specifications state that it should be calibrated based on measured static load test results to obtain an appropriate resistance factor.

The lower reliability of dynamic formulas is supported by the resistance factors for dynamic formulas contained in the AASHTO (2014) LRFD Bridge Design

Specifications. The AASHTO resistance factor for nominal resistances determined by dynamic formula is 0.40 for the FHWA modified Gates formula and 0.10 for the AASHTO modified Engineering News Formula. For non-redundant foundations consisting of 4 piles or less, the resistance factor associated with a given dynamic formula should be further reduced by 20%.

Table 13-2 Resistance Factors for Dynamic Formulas (modified from AASHTO 2014)

Formula	Resistance Determination Method	AASHTO (2014) Resistance Factor
FHWA Modified Gates Formula, ϕ_{dyn}	Nominal resistance determined using FHWA Modified Gates Formula at end of driving condition only. See Section 13.3.1.	0.40
AASHTO Modified Engineering News Formula, ϕ_{dyn}	Nominal resistance determined using AASHTO Modified Engineering News Formula at end of driving condition only. See Section 13.3.2.	0.10
WSDOT Dynamic Formula, ϕ_{dyn}	Nominal resistance determined using WSDOT Dynamic Formula at end of driving condition only. See Section 13.3.3.1.	Not in AASHTO See discussion in text.
MnDOT Dynamic Formula, ϕ_{dyn}	Nominal resistance determined using MnDOT Dynamic Formula. See Section 13.3.3.2.	Not in AASHTO See discussion in text.

13.3 DYNAMIC FORMULAS

13.3.1 FHWA Modified Gates Formula

For small projects where a dynamic formula is used, AASHTO states that the FHWA Modified Gates formula is preferable, since it correlates better with static load test results. The FHWA Modified Gates formula below has been revised to reflect the nominal resistance in kips and includes the 80 percent efficiency factor on the rated energy, E_d, recommended by Gates. The specified units below must be used.

$$R_{ndr} = 1.75\sqrt{E_d}\log_{10}(10\,N_b) - 100 \qquad \text{Eq. 13-2}$$

Where:

R_{ndr} = nominal driving resistance (kips).
E_d = developed hammer energy (ft-lbs). If ram velocity is not measured, it may be assumed equal to the potential energy of the ram in the form of ram weight, W, (lbs) times stroke height, h (ft).
N_b = pile penetration resistance (blows/inch).

It is often desirable for construction inspection personnel or contractors to know the number of hammer blows per foot of pile penetration which will be required to obtain the specified nominal resistance. The FHWA modified Gates formula for this purpose can be re-written as follows:

$$N_{ft} = 12\ (10^x) \qquad \text{Eq. 13-3}$$

In which:

$$x = [(R_{ndr} + 100)\ /\ 1.75\sqrt{E_d})] \qquad \text{Eq. 13-4}$$

Where:

N_{ft} = pile penetration resistance (blows/foot).
x = exponent defined by Equation 13-4 and terms per Equation 13-2.

All dynamic formulas are empirically developed based the resistance predicted using end of drive blow count observations with the resistance determined from static load test results performed at a later time. The FHWA modified Gates formula therefore inherently includes time dependent resistance changes due to soil setup or relaxation. AASHTO (2014) recommends a resistance factor, φ_{dyn}, of 0.40 for the FHWA Modified Gates formula, and that the formula be used only for end of drive conditions.

13.3.2 AASHTO Modified Engineering News Formula

The Engineering News formula was developed to provide an estimated nominal resistance based upon hammer energy and the observed pile set. The formula uses the pile set during driving to empirically estimate the long term nominal resistance. Therefore, this dynamic formula also incorporates any time dependent soil resistance changes as a result of the empirical procedure. For this reason, restrike set observation should not be used to calculate the nominal resistance.

The AASHTO modified version for nominal resistance calculations based on end of driving conditions is presented in Equation 13-5. The specified units for energy and set must be used.

$$R_{ndr} = \frac{12\,E_d}{s_b + 0.1} \qquad \text{Eq. 13-5}$$

Where:

R_{ndr} = nominal driving resistance (kips).
E_d = developed hammer energy (ft-kips).
s_b = permanent pile set (inches).

As noted in Section 13.1.1, the Engineering News formula has long been recognized to be one of the least accurate and least consistent of the dynamic formulas. AASHTO specifications (2014) reflect this by assigning a resistance factor of 0.10 to the Engineering News formula and recommending that the formula be applied to only end of drive conditions.

13.3.3 Other Dynamic Formulas

A few state transportation agencies have developed their own dynamic formulas and associated resistance factors through calibration with static load test databases. Hammer type, pile type and size, as well as geologic conditions were considered in the dynamic formulas developed by the Washington State DOT and Minnesota DOT.

13.3.3.1 Washington State DOT Pile Driving Formula

The Washington State DOT utilized a database of 141 static pile load results from Paikowsky et al. (2004) to evaluate nominal resistance predictions made by the FHWA Modified Gates formula relative to the static load test results. The original WSDOT intent was to slightly modify and improve the FHWA modified Gates formula. However, enough changes were made that a new dynamic formula was developed and named the WSDOT pile driving formula. The formula is presented in Equation 13-6 and the associated research study is presented in Allen (2005).

$$R_{ndr} = 6.6\mathrm{F}_{eff}E_d \ \ln(10N_b) \qquad \text{Eq. 13-6}$$

Where:

- R_{ndr} = nominal pile resistance measured during driving (kips).
- F_{eff} = Hammer efficiency factor.
  - 0.55 for air/steam hammers on all pile types.
  - 0.35 for closed end diesel hammers on all pile type.
  - 0.47 for open end diesel hammers on steel piles.
  - 0.37 for open end diesel hammers on timber or concrete piles.
  - 0.58 for hydraulic hammers on all pile types.
  - 0.28 for drop hammers on all pile types.
- E_d = developed energy, equal to W * h (ft-kips).
- W = ram weight (kips).
- h = observed ram stroke (feet).
- N_b = number of blows for 1.0 inch of pile permanent set, averaged over the last four inches of driving.

Table 13-3 summarizes the pile types, the range in pile section size, the range in load test failure load, and the number of data sets for a given pile type in the correlation database. Based on the database used to develop the formula, Allen estimated the average amount of resistance from soil setup included in the formula resistance prediction is about 30 to 70%. Hence, the formula should not be used for nominal resistance assessments using restrike observations since setup is already included in the dynamic formula correlation calibration.

Monte Carlo simulations were used for reliability analyses to estimate the reliability index, β, and the resistance factor needed to achieve a target reliability index value of 2.3 or 3.0, Allen (2005). Based on a target β of 2.3, WSDOT recommended a resistance factor, φ_{dyn}, of 0.55 be used with this formula for redundant foundations. For non-redundant foundations, defined as 4 piles or less, the WSDOT recommended resistance factor is 0.45 based on a target β of 3.0. The WSDOT dynamic formula, either as presented in Equation 13-6 or in a slightly modified version, has been adopted by the state transportation agencies in Washington, New Mexico, and Illinois.

Table 13-3 Pile Types and Sizes Contained in WSDOT Formula Database

Pile Type	Range in Pile Size	Range in Static Load Test Failure Load for Pile Type (kips)	Number of Datasets
Precast Concrete	12 to 36 inch	308 to 1797	49
Closed End Pipe	10 to 48 inch O.D.	237 to 1300	46
H-pile	10 to 14 inch	214 to 1239	29
Open End Pipe	24 to 60 inch O.D.	586 to 1984	9
Concrete Cylinder	20 to 54 inch O.D.	324 to 1452	5
Monotube	N/A	227 to 463	2
Timber	N/A	200	1

13.3.3.2 Minnesota Pile Formula 2012 (MPF12)

Paikowsky et al. (2009) combined pipe pile and H-pile load test information from Database PD/LT 2000, developed for the NCHRP Report 507 LRFD calibration study, with additional pipe and H-pile correlation data gathered from MnDOT practice. The original MnDOT/LT 2008 database included 40 H-pile, 65 closed end pipe pile, and 12 open end pipe pile data sets typical of the pile types, sizes, and soil conditions encountered in MnDOT practice. With time, additional datasets were added to the database. The expanded database was used for development and calibration of a new dynamic formula named the Minnesota Pile Formula 2012 (MPF12).

In using the formula, it was recommended that the hammer energy (W)(h) be limited to 85% of the hammer manufacturer's maximum rated energy in cases where the hammer energy exceeded the 85% value. The Minnesota Pile Formula 2012 (MPF12) for use on steel and concrete piles is presented in Equation 13-7.

$$R_{ndr} = 40\sqrt{\frac{W\,h}{1000}}\,\log\left(\frac{10}{s_b}\right) \qquad \text{Eq. 13-7}$$

Where:

R_{ndr} = nominal driving resistance (kips).
W = ram weight (lbs).
h = observed ram stroke (feet).
s_b = permanent set of last hammer blow (inches).
Wh = not to exceed 85% of manufacturer's maximum rated energy (ft-lbs).

The first order second moment method (FOSM) procedure and Monte Carlo simulations were used for reliability analyses to estimate a target reliability index value of 2.33 for redundant foundations. Based on these procedures, Paikowsky et al. (2014), recommended resistance factors, φ_{dyn}, of 0.50 for pipe piles, 0.60 for H-piles, 0.50 for non-voided, prestressed concrete piles up to 24 inch in size, and 0.80 for voided prestressed concrete piles 20 inch to 54 inch in size. It was recommended that the MPF12 formula be further modified as part of a later research study for the large, voided, concrete piles. All of the resistance factors recommended in the study were also based on the pile penetration resistance falling between 2 and 15 blows per inch.

The MPF12 dynamic formula was also evaluated for timber piles. It was noted that a modifier of 0.5 should be applied to the formula for timber piles due to the increased energy loss. The hammer energy (W)(h) should be limited to 85% of the hammer manufacturer's maximum rated energy in cases where the hammer energy exceeds the 85% value. The resulting modified form of MPF12 for timber piles is as follows:

$$R_{ndr} = 20\sqrt{\frac{W\,h}{1000}}\,\log\left(\frac{10}{s_b}\right) \qquad \text{Eq. 13-8}$$

Where:

R_{ndr} = nominal driving resistance (kips).
W = ram weight (lbs).
h = observed ram stroke (feet).
s_b = permanent set of last hammer blow (inches).
Wh = not to exceed 85% of manufacturer's maximum rated energy (ft-lbs).

Paikowsky et al. (2014) recommended a resistance factor, φ_{dyn}, of 0.60 be used for the nominal resistance determined with Equation 13-8 for timber piles. This resistance factor was also based on a pile penetration resistance of between 2 and 15 blows per inch.

The research study proposed that the MPF12 dynamic formulas could be used for either end of driving or restrike conditions with no change in the resistance factor or formula. This is somewhat unusual as most dynamic formulas are limited to only end of drive applications since they are empirically correlated to load test results and therefore inherently include time dependent changes in the nominal geotechnical resistance.

13.4 DYNAMIC FORMULA CASE HISTORY

To illustrate the variable performance of dynamic formulas compared to more reliable methods, a case history will be briefly discussed. A 50 feet long, 24 inch square, prestressed concrete pile was driven 45 feet below grade with an ICE I-46 single acting diesel hammer. The factored load to be supported by the pile was 380 kips. Soil conditions consisted of 30 feet of loose to medium dense sand overlying a 5 feet thick layer of medium dense cemented sand and limestone. The cemented sand was underlain by the intended bearing layer of medium dense to dense sand.

At the end of driving the test pile had a penetration resistance of 49 blows per foot at an average hammer stroke of 8.14 feet. The test pile was restruck 5 days after initial driving and had restrike penetration resistances of 4, 2, 2, and 2 blows per inch. The corresponding average hammer stroke at the beginning and end of this restrike was 8.05 and 8.55 feet, respectively. An axial compression load test was performed on this pile one week after initial driving (3 days after the restrike). Following the static load test, the pile was again restruck. The penetration resistances per inch of the second restrike were 4, 3, 3, 3, 3, 3, 3, and 4 blows per inch. The average hammer stroke at the beginning of this second restrike was 7.21 feet.

Nominal resistance estimates from dynamic formulas as well as from wave equation analysis, dynamic testing with signal matching, and static load testing are summarized in Table 13-4. The wave equation nominal resistance predictions were obtained from a fixed stroke bearing graph analysis with the soil model determined using default values from the GRLWEAP ST soil model. Dynamic testing signal matching results are based on CAPWAP results. The static load test failure load was determined using the Davisson Offset limit criteria.

For the presented case, the MnDOT dynamic formula provided the closest nominal resistance prediction to the static load test result based on inputting end of initial driving penetration resistance and stroke values into the four dynamic formulas. Overall the closest correlation to the static load test determined nominal resistance was achieved from wave equation analysis (1% underprediction) and dynamic testing with signal matching (4% underprediction). A comparison of the maximum factored resistance that could be supported based on the results from a given method is presented in Table 13-5. The factored dynamic test results with signal matching on beginning of restrike data correlated best to the maximum factored resistance determined from the static load test.

Table 13-4 Case History – Comparison of Calculated Nominal Resistances

Nominal Resistance Method	Test Condition	Resistance Factor for Test Method φ_{dyn}	Nominal Resistance (kips)	Difference Relative to Static Load Test Resistance (%)
AASHTO EN Formula	EOD	0.10	2872	+ 463
AASHTO EN Formula	BOR-1	- - -	2799 (2)	+ 449
AASHTO EN Formula	EOR-1	- - -	1734 (2)	+ 240
AASHTO EN Formula	BOR-2	- - -	2507 (2)	+ 391
FHWA Modified Gates Formula	EOD	0.40	710	+ 39
FHWA Modified Gates Formula	BOR-1	- - -	701 (2)	+ 37
FHWA Modified Gates Formula	EOR-1	- - -	570 (2)	+ 12
FHWA Modified Gates Formula	BOR-2	- - -	658 (2)	+ 29
WSDOT Formula	EOD	0.55 (1)	950	+ 86
WSDOT Formula	BOR-1	- - -	934 (2)	+ 83
WSDOT Formula	EOR-1	- - -	806 (2)	+ 58
WSDOT Formula	BOR-2	- - -	837 (2)	+ 64
MNDOT Formula	EOD	0.50 (1)	585	+ 15
MNDOT Formula	BOR-1	0.50 (1)	579	+ 14
MNDOT Formula	EOR-1	0.50 (1)	485	- 5
MNDOT Formula	BOR-2	0.50 (1)	548	+ 7
Wave Equation Analysis	EOD	0.50	560	+ 10
Wave Equation Analysis	BOR-1	0.50	560	+ 10
Wave Equation Analysis	EOR-1	0.50	440	- 14
Wave Equation Analysis	BOR-2	0.50	505	- 1
Dynamic Test & Signal Matching	EOD	0.65	600	+ 18
Dynamic Test & Signal Matching	BOR-1	0.65	523	+ 3
Dynamic Test & Signal Matching	EOR-1	0.65	460	- 10
Dynamic Test & Signal Matching	BOR-2	0.65	490	- 4
Static Load Test	- - -	- - -	510	- - -

Notes: (1) – Resistance factor not in AASHTO.
(2) – Method not recommended to be used in restrike condition by AASHTO or formula developer.

Table 13-5 Case History – Comparison of Factored Resistance

Nominal Resistance Method	Test Condition	Resistance Factor for Test Method φ_{dyn}	Maximum Factored Resistance From Method (kips)	Difference Relative to Load Test Determined Factored Resistance (%)
AASHTO EN Formula	EOD	0.10	287	- 13
AASHTO EN Formula	BOR-1	- - -	280 (2)	- 16
AASHTO EN Formula	EOR-1	- - -	173 (2)	- 48
AASHTO EN Formula	BOR-2	- - -	251 (2)	- 24
FHWA Modified Gates Formula	EOD	0.40	284	- 14
FHWA Modified Gates Formula	BOR-1	- - -	280 (2)	- 15
FHWA Modified Gates Formula	EOR-1	- - -	228 (2)	- 31
FHWA Modified Gates Formula	BOR-2	- - -	263 (2)	- 21
WSDOT Formula	EOD	0.55 (1)	522	+ 58
WSDOT Formula	BOR-1	- - -	514 (2)	+ 55
WSDOT Formula	EOR-1	- - -	443 (2)	+ 34
WSDOT Formula	BOR-2	- - -	460 (2)	+ 39
MNDOT Formula	EOD	0.50 (1)	293	- 12
MNDOT Formula	BOR-1	0.50 (1)	289	- 13
MNDOT Formula	EOR-1	0.50 (1)	242	- 27
MNDOT Formula	BOR-2	0.50 (1)	274	- 17
Wave Equation Analysis	EOD	0.50	280	- 16
Wave Equation Analysis	BOR-1	0.50	280	- 16
Wave Equation Analysis	EOR-1	0.50	220	- 34
Wave Equation Analysis	BOR-2	0.50	253	- 24
Dynamic Test & Signal Matching	EOD	0.65	390	+ 18
Dynamic Test & Signal Matching	BOR-1	0.65	340	+ 3
Dynamic Test & Signal Matching	EOR-1	0.65	299	- 10
Dynamic Test & Signal Matching	BOR-2	0.65	319	- 4
Static Load Test	- - -	0.75	332	- - -

Notes: (1) – Resistance factor not in AASHTO.

(2) – Method not recommended to be used in restrike condition by AASHTO or formula developer.

13.5 ADVANTAGES, DISADVANTAGES, AND LIMITATIONS

Dynamic formulas offer a method to quickly estimate nominal resistance during driving. Because of the simple inputs such as hammer energy and pile set, relatively little engineering judgement is needed to perform the calculations. The primary advantages of dynamic formula use are the immediate availability of the driving criterion and the minimal delay to pile driving operations.

Conversely, dynamic formulas have several disadvantages. They do not consider the entire driving system (i.e. hammer, pile, and soil), variation in hammer performance, nor energy losses due to pile stiffness. However, the primary disadvantages are formula accuracy and reliability. Most shortcomings of dynamic formulas can be overcome by a more realistic analysis of the pile driving process, such a wave equation analysis. Dynamic testing and analysis is another tool which is superior to use of dynamic formulas.

AASHTO limits dynamic formula use to piles with a nominal resistance of 600 kips or less. This nominal resistance value is well beyond the typical nominal resistance values in historical databases evaluating dynamic formula performance. Other codes such as the International Building Code limit formula use to 160 kips because of their limitations. Dynamic formulas are based on Newtonian impact theory which is invalidated by the use of hammer and pile cushions.

13.6 PRACTICAL ISSUES AND CONSIDERATIONS

Use of a dynamic formula is often “justified” because of the small number of piles on a project or the presence of a consistent and hard bearing layer such as bedrock. On small projects with a limited number of piles, the cost of the extra pile length resulting from use of a less reliable nominal resistance verification method may be more economical than the testing cost or test method impact on the construction schedule. Similarly when piles are driven to a hard bearing layer, little additional pile length may be necessary to achieve the higher nominal resistance required by a dynamic formula and more reliable resistance verification methods may not be economically justified. However, driving stresses and their control must be considered in this situation and this assessment cannot be made using a dynamic formula.

The Indiana Department of Transportation evaluated installation costs on driven pile foundations for state bridge projects installed from 2009 to 2014. The pile foundations were installed using either the FHWA modified Gates dynamic formula or dynamic testing with signal matching. On the dynamic testing controlled projects, each test pile was dynamically monitored during both initial driving and during restrike with the restrike test conducted between 1 and 7 days after initial driving depending on the subsurface conditions. The cost per lineal foot of pile and the cost per kip of load support for pile foundation projects controlled by dynamic formula or dynamic testing with signal matching as reported by Zaheer et al. (2015) is summarized in Table 13-6.

Table 13-6 INDOT Dynamic Formula and Dynamic Testing Comparison

Details	FHWA Modified Gates Formula	Dynamic Testing with Signal Matching	Totals
Plan Contract Length (ft)	246,052	995,100	1,241,152
Paid Pile Length (ft)	216,664	937,873	1,154,517
Pile Length underrun or {overrun} (lf)	29,408	57,227	86,638
Pile Length underrun or {overrun} (%)	13.6%	6.1%	7.5%
Cost Paid to Contractor	$11,653,634	$46,178,800	$57,832,434
Average Unit Cost (per lf)	$ 53.79	$ 49.24	$ 50.09

Zaheer et al. (2015) determined that the use of a simple dynamic formula has several economic drawbacks. Their review also concluded the factored load carried by a dynamic formula controlled pile was 21% less than the factored load carried by a dynamically tested pile. For the same factored load, pile lengths determined through use of a dynamic formula were 10 to 20% greater than the length determined from dynamic testing. The cost per kip of supported structure load for a dynamic formula pile was 39% higher than that of a dynamically tested pile. Overall, the average unit cost per linear foot of dynamic formula installed pile was 9.2% more than the cost per foot of dynamic test with signal matching installed pile.

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CHAPTER 14

CONTRACT DOCUMENTS

14.1 OVERVIEW OF PLAN AND SPECIFICATION REQUIREMENTS

Pile foundations generally cannot be inspected after installation. Therefore, construction specifications and monitoring are of prime importance for a successful pile foundation. Preparation of the contract plans, plan details, and specifications related to piling issues are the responsibility of the foundation designer in cooperation with materials and construction personnel. Project plans should include:

- Location of piles.
- Pile numbering system to clearly identify each pile in group or bent.
- Pile type, section, and estimated length.
- Pile toe details, driving shoe, closure plate, etc.
- Pile splicing details.
- Pile cut off elevation.
- Pile cap connection details.
- Estimated pile toe elevation.
- Minimum pile toe elevation, if needed.
- Required pile batter and direction.
- Orientation of H-piles.
- Factored resistance, R_r.
- Nominal resistance, R_n.
- Nominal driving resistance, R_{ndr}.
- Location of subsurface borings.
- Results of subsurface exploration.

It is the designer's responsibility to confirm that plans and specifications have been prepared using compatible language. This is particularly true in defining the required nominal driving resistance, which is an important component of any driven pile specification. Problems can arise when plans provide only the factored resistance, the nominal resistance, or the nominal driving resistance without other details such as the resistance factor associated with the construction control method or anticipated losses in resistance due to scour, liquefaction, or relaxation. For example, plan statements such as "piles shall have a resistance of 300 kips," provide an unclear description of the contract requirements as the resistance could be interpreted as the factored resistance, nominal resistance, or nominal driving resistance. Plans should therefore clearly indicate the factored resistance, the resistance factor, the nominal resistance, the additional resistance from any unsuitable layers (scour, liquefaction and their elevations), resistance changes following driving (setup or relaxation effects) and the resulting nominal driving resistance.

This chapter includes a generic pile specification for highway projects that was originally developed with input from State and Federal bridge and geotechnical engineers and released in FHWA Geotechnical Guideline 13. It has been updated over time as necessary and modified from an ASD to LRFD specification. AASHTO LRFD Bridge Construction specifications (2010) provide a similar document with additional commentary. A good driven pile specification should include the basic components in Table 14-1.

The intent of the attached generic specification is to provide highway designers and transportation agencies with a comprehensive driven pile specification. However, this specification is not intended to be used directly for project application without review and modification by the foundation designer and transportation agency. Commentary sections are included where appropriate to assist the foundation designer in tailoring the generic specification to project requirements. The commentary sections explain the reasons behind development of particular sections of the specification and the relationship of the specification requirements to necessary pile design or construction activities.

Note that only driven piles are addressed in the specification. Other deep foundation types such as drilled shafts require completely different construction controls and are therefore not appropriate for inclusion in this generic pile specification.

In conventional design-bid-build contracts, agency standard specifications are used. The agency warrants to the contractor that the drawings and prescriptive specifications are complete and free from error (agency takes the risk). In design-build contracts, specifications are performance based which allows the design-builder to use their design and construction expertise to satisfy project requirements. The design-builder warrants to the agency that it will produce design documents that are complete and free from error (design-builder takes the risk). Standard specifications are used In CM/GC contract documents.

Table 14-1 Items to Include in a Driven Pile Specification

Category	Item
Pile Materials	Material type, grade, and strength. Coating details. Transportation and handling.
Driving System and Equipment	Hammer. Hammer and pile cushions. Helmet and inserts. Pile leads. Followers. Predrilling equipment. Jetting equipment. Spudding equipment.
Installation Issues	Driving sequence. Pile location tolerances. Pile alignment tolerances. Pile shoe or toe protection requirement. Pile splices. Pile cutoff. Pile cap connection. Pile heave. Pile rejection criteria.
Resistance Verification	Static load testing. Dynamic testing. Rapid load testing. Wave equation analysis. Dynamic formulas.
Basis of Payment	Method of measurement. Payment items.

14.2 GENERIC DRIVEN PILE SPECIFICATION

14.2.1 SECTION X.01 DESCRIPTION

This item shall consist of furnishing and driving foundation piles of the type and dimensions designated, including cutting off or building up foundation piles when required. Piling shall conform to and be installed in accordance with these specifications, and at the location, and to the elevation, penetration depth and/or nominal resistance shown on the plans, or as directed by the Engineer.

Except when test piles are required, the Contractor shall furnish the piles in accordance with the dimensions shown in the contract documents. When test piles are required, the pile lengths shown on the plans are for estimating purposes only and the actual lengths to be furnished for production piles will be determined by the Engineer after the test piles have been driven and tested. The lengths given in the order list will be based on the lengths which are assumed after cutoff to remain in the completed structure. The Contractor shall, without added compensation, increase the lengths to provide for fresh heading and for such additional length as may be necessary to suit the Contractor's method of operation.

Where required by the contract documents, the Contractor shall perform the static and/or dynamic and/or rapid load tests of the type and quantity specified at the locations indicated on the plans.

Commentary:

The objective of this specification is to provide criteria by which the Owner can assure that designated piles are properly installed and the Contractor can expect equitable compensation for work performed. The Owner's responsibility is to estimate the pile lengths required for the nominal resistances. Pile lengths should be estimated based on subsurface explorations, testing and analysis which are completed during the design phase, and expected variability in the subsurface conditions. Pile contractors who enter contractual agreements to install piles for an owner should not be held accountable or indirectly penalized for inaccuracies in estimated lengths. The Contractor's responsibility is to provide and install designated piles, undamaged, to the length specified by the Owner. This work is usually accomplished within an established framework of restrictions necessary to insure a "good" pile foundation. The price bid for this item of work will reflect the Contractor's estimate of both actual cost to perform the work and perceived risk.

14.2.2 SECTION X.02 SUBMITTALS AND APPROVALS

A) Pile Installation Plan:

A Pile Driving Installation Plan shall be prepared by the Contractor and submitted to the Engineer no later than 30 days before driving the first pile. The Pile Driving Installation Plan shall include the following:

1. List and size of proposed equipment including cranes, barges, pile driving equipment, jetting equipment, compressors, and predrilling equipment. Manufacturer's data sheets on hammers should be included.

2. Methods to determine hammer energy in the field for determination of nominal resistance. Include in the submittal necessary charts and recent calibrations for any pressure measuring equipment.

3. Detailed drawings of any proposed followers.

4. Detailed drawings of any templates.

5. Details of proposed load test equipment, all load test instrumentation, and load test procedures. Submitted load test program information should also include the details and arranged of the reaction frame and reaction piles as well as recent calibrations of jacks, required load cells and all monitoring equipment.

6. Sequence of driving of piles for each different configuration of pile layout.

7. Proposed schedule for test pile program and production pile driving.

8. Details of proposed means and procedures to document the preconstruction condition of existing nearby structures and utilities as well as proposed procedures for their protection and monitoring during the contract period.

9. Required shop drawings for piles, cofferdams, etc.

10. Methods and equipment proposed to prevent movement of piles during placement and compaction of fill within 15 feet of the piles.

11. Proposed method for placing steel reinforcement and concrete in concrete filled pipe piles.

12 Methods to prevent deflection of battered piles due to their own weight and to maintain their as-driven position until casting of the pile cap is complete.

13. Proposed pile splice locations and details of any mechanical or proprietary splices to be used.

B) Pile Driving Equipment Approval by Wave Equation:

All pile driving equipment furnished by the Contractor shall be subject to the approval of the Engineer. All pile driving equipment should be sized such that the project piles can be driven with reasonable effort to the estimated contract lengths without damage. Approval of pile driving equipment by the Engineer will be based on wave equation analysis unless a dynamic formula has been specified for nominal resistance verification in the field. In no case shall the driving equipment be transported to the project site until approval of the Engineer is received in writing. The Contractor shall submit a completed Pile Driving and Equipment Data Form (shown in Figure 14-1) to the Engineer for approval as part of the pile installation plan at least 30 days prior to the start of pile driving. If a follower is to be used, detailed drawings of the follower shall be included as part of this submittal.

Commentary:

Use of wave equation analysis for approval of driving equipment can substantially reduce pile driving costs and pile driving claims by checking that the equipment mobilized to the project can drive the pile to the required penetration depth without damage. Agencies should encourage Contractors to use wave equation analysis to select the optimum hammer for each project. In cases where disputes arise over rejection of pile driving equipment, the Engineer should request the Contractor to submit proof of the adequacy of the pile driving equipment. Proof should consist of, but not be limited to, a wave equation analysis of the proposed driving equipment performed by a registered professional engineer. All costs of this submission shall be the responsibility of the Contractor. The Pile and Driving Equipment Data Form should be submitted for approval even if wave equation analysis will not be used for hammer approval. The approved form should be used by

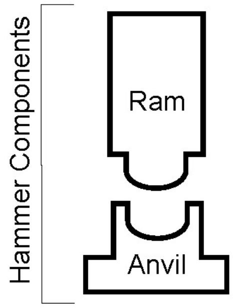
Contract No.:____________________ Structure Name and/or No.:______________________________

Project:________________________ ______________________________

________________________ Pile Driving Contractor or Subcontractor:______________________

County:________________________ ______________________________

(Piles driven by)



Hammer

Manufacturer:____________________ Model No.:______________

Hammer Type:____________________ Serial No.: ______________

Manufacturers Maximum Rated Energy:____________________(ft-lbs)

Stroke at Maximum Rated Energy:________________________(ft)

Range in Operating Energy:______________to______________(ft-lbs)

Range in Operating Stroke:______________to______________(ft)

Ram Weight:________________(kips)

Modifications:____

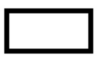
____

____

Striker Plate

Weight:________________(kips) Diameter:________________(in)

Thickness:______________(in)



Hammer Cushion

Material #1

Name:____________________

Area:____________________(in^2)

Thickness/Plate:____________(in)

No. of Plates:______________

Material #2 (Composite Cushion)

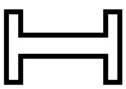
Name:____________________

Area:____________________(in^2)

Thickness/Plate:____________(in)

No. of Plates:______________

Total Thickness of Hammer Cushion:________________________(in)



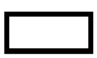
Helmet

Weight:________________(kips)

Insert (If Any)

Weight:________________(kips)

Total Weight of Helmet and Insert:________________________(kips)



Pile Cushion

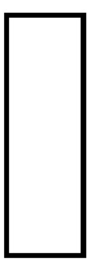
Material:____

Area:____________________(in^2) Thickness/Sheet:____________(in^2)

No. of Sheets:______________

Total Thickness of Pile Cushion:______________(in)

Maximum Thickness Accommodated by Helmet :________________(in)



Pile

Pile Type:____

Wall Thickness:____________(in) Taper:____________________

Cross Sectional Area:________(in^2) Weight/ft:____________________

Ordered Length:________________(ft)

Factored Resistance:______________(kips)

Nominal Resistance:______________(kips)

Description of Splice:____

____

Driving Shoe/Closure Plate Description:______________________________

____

Submitted By:__________________ Date:____________________

Telephone No.:__________________ Email:____________________

Figure 14-1 Drive system submittal form.

the pile inspector to check the proposed hammer and drive system components are as furnished and are maintained during the driving operation.

The criteria, which the Engineer will use to evaluate the pile driving equipment from the wave equation results, consists of both the required number of hammer blows per foot of penetration as well as the pile driving stresses at the required nominal resistance. The required penetration resistance (blow count) indicated by the wave equation at the required nominal resistance shall be between 30 and 96 blows per foot for the driving equipment to be acceptable.

Commentary:

Practical refusal is defined later in this generic specification as 10 blows per inch. Therefore, the upper limit of the penetration resistance for hammer approval should be less than the criteria for practical refusal. Otherwise, slight variations in assumed soil behavior or hammer performance will result in refusal driving conditions occurring prior to achieving the nominal resistance with an approved driving system.

In addition, the pile driving stresses indicated by the wave equation analysis of the proposed driving equipment shall not exceed material specific limits for the driving system to be acceptable. The AASHTO (2014) resistance factor for driven pile drivability analysis, φ_{da}, is 1.0 for all pile types with the exception of timber piles where it is 1.15. For steel piles, maximum compressive driving stresses shall not exceed the resistance factor, φ_{da}, times 90 percent of the minimum yield strength of the pile material. For prestressed concrete piles in normal environments, tensile stresses shall not exceed φ_{da} times 0.095 multiplied by the square root of the concrete compressive strength, f'_c, plus the effective prestress value, f_{pe} (with both f'_c and f_{pe} in ksi). For prestressed concrete piles in severe corrosive environments, tensile stresses shall not exceed φ_{da} times f_{pe}. Compressive stresses for prestressed concrete piles shall not exceed φ_{da} times 85 percent of the compressive strength minus the effective prestress value, (i.e. $0.85\ f'_c - f_{pe}$). For timber piles, the compressive driving stress shall not exceed φ_{da} times the reference design value of wood in compression parallel to the grain, F_{co}, as listed on the plans or provided in AASHTO (2014) Table 8.4.1.1-1.

The Contractor will be notified of the acceptance or rejection of the driving system within 14 calendar days of the Engineer's receipt of the Pile and Driving Equipment Data Form. If the wave equation analyses show that either pile damage or inability to drive the pile with a reasonable driving resistance to the desired nominal resistance will result from the Contractor's proposed equipment or methods, the Contractor shall modify or replace the proposed methods or equipment at his expense until subsequent wave equation analyses indicate the piles can be reasonably driven to the desired nominal resistance, without damage. The Engineer will notify the Contractor of the acceptance or rejection of the revised driving system within 7 calendar days of receipt of a revised Pile and Driving Equipment Data Form.

During pile driving operations, the Contractor shall use the approved system. No variations in the driving system will be permitted without the Engineer's written approval. Any change in the driving system will only be considered after the Contractor has submitted the necessary information for a revised wave equation analysis. The Contractor will be notified of the acceptance or rejection of the driving system changes within 7 calendar days of the Engineer's receipt of the requested change. The time required for submission, review, and approval of a revised driving system shall not constitute the basis for a contract time extension to the Contractor.

Commentary:

The nominal driving resistance, R_{ndr}, is the soil resistance which must be overcome (including resistance from unsuitable layers, liquefiable soils, and scour zone soils) to reach the pile penetration depth where the factored resistance can be achieved with an appropriate resistance factor based on the nominal resistance verification method used in the field. The resistance factor depends on the reliability of the resistance determination method as well as, for some methods, the number of tests performed. Table 14-2 provides a summary of the resistance determination methods and their associated resistance factors. AASHTO does not provide a recommended resistance factor for rapid load test methods as discussed in Section 11.4.7.

The nominal driving resistance is affected by:

1. *The resistance in unsuitable soil support layers overlying suitable support layers.*

2. *Minimum penetration requirements.*

3. *Temporary loss or increase in soil strength due to driving operations.*

4. *Pile installation methods which alter the in place soil resistance such as jetting, predrilling, etc.*

The designer must estimate the nominal driving resistance. Only on the most routine pile projects will the nominal driving resistance be equal to nominal resistance (i.e. the nominal resistance is the factored resistance divided by the resistance factor). More typically, piles are used to penetrate upper soil layers which are unsuitable for load support due to either poor soil characteristics, or future loss of load support by scour or liquefaction. In such cases, resistance in the unsuitable layers is not considered in determining the pile penetration necessary to support the factored resistance. However, the estimated nominal driving resistance must include the resistance encountered in penetrating those unsuitable support layers, in addition to the nominal resistance.

The nominal driving resistance must be shown on the contract documents to permit the Contractor to properly size the driving equipment and the Engineer to judge the acceptability of the Contractor's driving equipment. Optimum pile installation generally occurs when the nominal driving resistance is obtained with a driving effort below the point of maximum curvature (typically around 84 to 96 blows per foot) of the wave equation bearing graph. Larger penetration resistance result in negligible pile penetration per blow and generally inefficient driving conditions. Excessive driving resistances can also result in damage to the pile or the driving system.

Table 14-2 Resistance Factors for Field Determination Methods (after AASHTO 2014)

Condition	*Resistance Determination Method*	*AASHTO Resistance Factor*
Nominal Resistance of Single Pile in Compression *Dynamic Analysis and Static Load Test Methods,* ϕ_{dyn}	*Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing with signal matching of at least two piles per site condition, but no less than 2% of the production piles.*	*0.80*
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing.	*0.75*
	Driving criteria established by dynamic testing with signal matching conducted on 100% of production piles.	*0.75*
	Driving criteria established by dynamic testing with signal matching of at least two piles per site condition, but no less than 2% of the production piles.	*0.65*
	Wave equation analysis, without pile dynamic measurements or load test, at end of drive conditions only.	*0.50*
	FHWA Modified Gates dynamic pile formula (End of Drive condition only).	*0.40*
	Engineering News dynamic pile formula (End of Drive condition only).	*0.10*
Nominal Resistance of Single Pile in Tension *Dynamic Analysis and Static Load Test Methods,* ϕ_{up}	*Static load test.*	*0.60*
	Dynamic testing with signal matching.	*0.50*

C) Pile Driving Equipment Approval by Alternate Method:

An alternate method of driving equipment approval will be used when the contract documents state that nominal resistance verification in the field will be determined from a dynamic formula. The alternate approval method requires that the energy of the driving equipment be rated by the manufacturer at or above the appropriate minimum energy level in Table 14-3 corresponding to the nominal resistance shown on the plans. The penetration resistance required by the dynamic formula for the submitted hammer and required nominal resistance should not exceed 10 blows per inch.

During pile driving operations, the Contractor shall use the approved system. If the Engineer determines the Contractor's hammer is unable to transfer sufficient energy to the pile, the hammer shall be removed from service until repaired to the satisfaction of the Engineer. No variations in the driving system will be permitted without the Engineer's written approval. Any changes in the driving system will be considered only after the Contractor has submitted a new Pile and Driving Equipment Data form. The Contractor will be notified of the acceptance or rejection of the proposed change in driving equipment within 7 calendar days of the Engineer's receipt of the form.

Table 14-3 Alternate Approval Method Minimum Pile Hammer Requirements

Nominal Resistance (kips)	Minimum Manufacturers Rated Hammer Energy (ft-lbs) *
180 and under	-----
181 to 300	-----
301 to 420	-----
421 to 540	-----
541 to 600	-----
Over 600	-----

* See commentary.

Commentary:

A table of the minimum rated hammer energy vs. nominal resistance should be developed using wave equation analyses of commonly available driving systems for the pile types, pile lengths, and pile loads routinely used by the specific agency. These analyses should model the typical soil and pile *installation conditions. The wave equation results should be evaluated for driving stress levels and penetration resistance (blow count) to determine which hammer energies are too large (driving stress problems or penetration resistance at nominal resistance less than 30 blows/ft) and which energies are too small (penetration resistance at nominal resistance greater than practical refusal value of 120 blows /ft).*

Once the specific table of energy values has been developed, it should only be considered for routine projects in uniform soil conditions. Projects involving long piles or large nominal resistances relative to the factored load (such as piles subject to significant scour depths or piles to be driven through embankments) should use project specific wave equation analyses to establish minimum driving equipment requirements. Piles to soft and hard rock should also be evaluated by wave equation analysis to reduce the risk of pile damage from too large a hammer.

14.2.3 SECTION X.03 MATERIALS

Materials shall meet the requirements in the following Subsections of Section X.03 Materials:

Portland Cement Concrete
Reinforcing Steel
Prestressing Strands / Post-Tensioning Tendons
Structural Steel
Castings for Pile Shoes
Steel Shells for Cast in Place Piles
Timber Piles
Timber Preservative and Treatment
Protective Coatings

Commentary:

The appropriate sections of each agency's standard specifications should be included under the X.03 Materials. A generic materials section cannot be provided herein, considering the vast combinations of materials used in piling operations and the varying control methods used by individual agencies. The above list contains the common material components. Additions or deletions may be required to this list based on the content of individual agency *standard specifications and the pile type specified. The 3rd Edition of the AASHTO LRFD Bridge Construction Specifications contains additional text and commentary on pile material topics which should be reviewed and used to update agency standard specifications as appropriate.*

14.2.4 SECTION X.04 DRIVING EQUIPMENT AND APPURTENANCES

A) Pile Hammers:

Piles may be driven with air, steam, diesel, or hydraulic hammers. Drop hammers, if specifically permitted in the contract, shall not be used for concrete piles or for piles whose required nominal resistance exceeds 120 kips. When drop hammers are permitted, the ram shall have a weight not less than 2.0 kips and the height of the drop shall not exceed 12.0 feet. In no case shall the ram weight of the drop hammer be less than the combined weight of helmet and pile. All drop hammers shall be equipped with hammer guides to insure concentric impact on the helmet.

Air/steam hammers shall be operated and maintained within the manufacturer's specified ranges. The plant and equipment furnished for air/steam hammers shall have sufficient capacity to maintain at the hammer, under working conditions, the volume and pressure specified by the manufacturer. The hose connecting the compressor or boiler with the hammer shall be at least the minimum size recommended by the hammer manufacturer. The plant and equipment shall be equipped with accurate pressure gauges which are easily accessible to the Engineer. The weight of the striking parts of air/steam hammers shall not be less than one third the weight of helmet and pile being driven, and in no case shall the striking parts weigh less than 2.75 kips. If a wave equation analysis is used for hammer approval the minimum ram weight requirements shall not apply.

Open end (single acting) diesel hammers shall be equipped with a device such as rings on the ram to permit the Engineer to visually determine hammer stroke at all times during pile driving operations. Also, the Contractor shall provide the Engineer a chart from the hammer manufacturer equating stroke and blows per minute for the open end diesel hammer to be used. For open end diesel hammers, the contractor shall provide and maintain in working order for the Engineer's use, an approved device to automatically determine and display ram stroke.

Closed end (double acting) diesel hammers shall be equipped with a bounce chamber pressure gauge, in good working order, mounted near ground level so as to be easily read by the Engineer. Also, the Contractor shall provide the Engineer a chart, calibrated to actual hammer performance within 60 days of use, equating bounce chamber pressure to either equivalent energy or stroke for the closed end diesel hammer to be used.

Hydraulic hammers shall be equipped with a system for measuring and immediately displaying in the field, the kinetic energy or ram impact velocity. The system shall be maintained in good working order and operational at all times piles are driven.

Vibratory hammers, when permitted for installing production piles, shall be used only after the pile toe elevation for the nominal resistance is established by load testing and/or from test piles restruck with an impact hammer. The Contractor shall perform, at his cost, the load tests and/or extra work required by the Engineer as needed for approval of the vibratory hammer use. Installation of production piles with vibratory hammers shall be controlled according to power consumption, rate of penetration, specified toe elevation, or other means acceptable to the Engineer which assures the nominal resistance equals or exceeds the nominal resistance of the test pile. In addition, the first of every 10 piles installed with a vibratory hammer shall be restruck with an impact hammer of suitable energy to verify the nominal resistance before driving the remaining piles.

Commentary:

Pile inspectors frequently do not possess adequate knowledge or technical information concerning even the most basic details of the Contractor's hammer. Chapters 15 and 18 provide information on driving equipment and monitoring. Agencies and contractors should also provide pile “inspectors” with basic manuals such as FHWA/RD 86/160 "The Performance of Pile Driving Systems: "Inspections Manual" or "Inspectors Manual for Pile Foundations" and "A Pile Inspectors Guide to Hammers, Second Edition" available from the Deep Foundation Institute, 120 Charlotte Place, Englewood Cliffs, NJ 07632.

On large projects or on projects requiring large pile resistances, specifications should consider requiring kinetic energy readout devices for hammers as described in Section 15.19 of Chapter 15. Several manufacturers can equip their hammers with these devices when requested. Any existing hammer can also be retrofitted with a kinetic energy readout device. These devices allow improved quality assurance and can detect changes in hammer performance over time that may necessitate adjustment to the pile installation criterion.

At present no formula exists to reliably predict the nominal resistance of piles driven with vibratory hammers. Until reliable procedures are developed for vibratory installation, special precautions must be taken to insure foundation piles installed with vibratory hammers have both adequate nominal resistance and structural integrity. As discussed in Section 7.10.5, the use of vibratory hammers may also affect the shaft resistance that develops on the pile. On critical projects, "owners" should consider the use of dynamic testing during restrike to substantiate pile nominal resistance and integrity.

B) Drive System Components and Accessories:

1. Hammer Cushion: Impact pile driving equipment designed to be used with a hammer cushion shall be equipped with a suitable thickness of hammer cushion material to prevent damage to the hammer or pile and to insure uniform driving behavior. Hammer cushions shall be made of durable manufactured materials, provided in accordance with the hammer manufacturer's guidelines. Wood, wire rope, and asbestos hammer cushions are specifically disallowed and shall not be used. A striker plate, as recommended by the hammer manufacturer, shall be placed directly above the hammer cushion to insure uniform compression of the cushion material. The hammer cushion shall be removed from the helmet and inspected in the presence of the Engineer when beginning pile driving at each structure or after each 100 hours of pile driving, whichever is less. Any reduction of hammer cushion thickness exceeding 25 percent of the original thickness shall be replaced by the Contractor before driving is permitted to continue.

Commentary:

For hammers requiring cushion material, mandatory use of a durable hammer cushion material that will retain uniform properties during driving is necessary to accurately relate penetration resistance (blow count) to nominal pile resistance. Non-durable materials which deteriorate during driving cause erratic estimates of nominal resistance and, if allowed to dissolve, result in damage to the pile or driving system.

2. Helmet: Piles driven with impact hammers require an adequate helmet or drive head to distribute the hammer blow to the pile head. The surface of the helmet in contact with the pile shall be plane and smooth and shall be aligned parallel with the hammer base and the pile head. The helmet shall be guided by the leads and not be free swinging. The helmet shall fit around the pile head in such a manner as to prevent transfer of torsional forces during driving, while maintaining proper alignment of hammer and pile. An insert may be used with a helmet to adapt the helmet to different types or sizes of piles.

For steel and timber piling, the pile heads shall be cut squarely and a helmet, as recommended by the hammer manufacturer, shall be provided to hold the axis of the pile in line with the axis of the hammer.

For timber piles, the least inside helmet or hammer base horizontal dimension shall not exceed the pile head diameter by more than 2.0 inches. If the timber pile diameter slightly exceeds the least helmet or hammer base dimension, the pile head shall be trimmed to fit the helmet.

For precast concrete and prestressed concrete piles, the pile head shall be plane and perpendicular to the longitudinal axis of the pile to prevent eccentric impacts from the helmet.

For special types of piles, appropriate helmets, mandrels or other devices shall be provided in accordance with the manufacturer's recommendations so that the piles may be driven without damage.

3. Pile Cushion: The heads of concrete piles shall each be protected by a pile cushion having the same cross sectional area as the pile top. Pile cushions shall be made of plywood, hardwood, or composite plywood and hardwood materials.

The minimum pile cushion thickness placed on the pile head prior to driving shall be determined by wave equation analysis so that driving stress limits are not exceeded. If a dynamic formula is used, the minimum pile cushion thickness shall be at least 4 inches.

A new pile cushion shall be provided for each pile. In addition, the pile cushion shall be replaced during the driving of any pile if the cushion is compressed more than one-half the original thickness or it begins to burn. Pile cushions shall be protected from the weather, and kept dry prior to use. Pile cushion shall not be soaked in any liquid unless approved by the Engineer. The use of manufactured pile cushion materials in lieu of a wood pile cushion shall be evaluated on a case by case basis.

A used pile cushion in good condition shall be used for restrike tests. The used pile cushion shall be the same cushion from the end of initial driving unless that pile cushion condition has deteriorated. If the original pile cushion has deteriorated, another used pile cushion of similar thickness as the original cushion at the end of drive shall be used.

Commentary:

A pile cushion is only needed for the protection of concrete piles. If the wave equation analysis of the Contractor's hammer indicates tension stresses exceed specification limits, the pile cushion may need to be substantially thicker than 4 inches. Pile cushion thicknesses greater than 18 inches have been used to mitigate tension stresses. Compressive stresses at the pile head can generally be controlled with a relatively thin pile cushion. However, wood pile cushions may become overly compressed and hard after about 1000 to 1500 hammer blows. Conversely, cushions exposed to less than 50 blows are generally not suitable for restrikes.

4. Leads: Piles shall be supported in line and position with leads while being driven. Pile driver leads shall be constructed in a manner that affords freedom of movement of the hammer while maintaining alignment of the hammer and the pile to insure concentric impact for each blow.

Leads may be either fixed or swinging type. Swinging leads shall be adequately embedded in the ground or the pile constrained in a structural frame such as a template to maintain alignment and location tolerances. Swinging leads shall be fitted with a pile gate at the bottom of the leads unless used with a template. Leads shall be of sufficient length to make the use of a follower unnecessary. Leads used for driving batter piles shall be designed to permit and maintain proper alignment of the batter pile. A horizontal brace may be required between the crane and base of leads to maintain alignment and location tolerances in some batter pile installation conditions.

5. Followers: Followers shall only be used when approved in writing by the Engineer, or when specifically stated in the contract documents. When a follower is proposed, a wave equation analysis shall be used to evaluate the suitability of the proposed driving system. As a general guide, the cross sectional area of a steel follower when driving concrete piles should be at least 20 percent of the cross sectional area of the concrete pile. When driving steel or timber piles, the cross sectional area of the steel follower should have an impedance between 50 percent and 200 percent of the pile impedance.

The follower and pile shall be held and maintained in equal and proper alignment during driving. The follower shall be of such material and dimensions to permit the piles to be driven to the penetration depth determined necessary from the driving of the full length piles. The follower shall be designed with guides adapted to the leads that maintain the hammer, follower and pile in alignment during driving. The lower end of the follower shall be equipped with a helmet or follower-pile connection suitable for the pile type being driven.

The final position and alignment of the first two piles installed with followers in each substructure unit shall be verified to be in accordance with the location and alignment tolerances in Section X.06 C) 4 before additional piles are installed.

Commentary:

The use of a follower often causes substantial and erratic reductions in the hammer energy transmitted to the pile due to the follower flexibility, poor connection to the pile head, frequent misalignment, etc. Reliable correlations of penetration resistance with nominal resistance are very difficult when followers are used. Therefore, the nominal resistance of select follower driven piles should be checked with either a static load test or dynamic testing with signal matching. Severe problems with pile alignment and location frequently occur when driving batter piles with a follower in a cofferdam unless a multi-tier template is used.

6. Jets: Jetting shall only be permitted if approved in writing by the Engineer or when specifically stated in the contract documents. The contractor shall determine the number of jets and the volume and pressure of water at the jet nozzles to freely erode the material adjacent to the pile without affecting the lateral stability of the final in place pile.

The Contractor shall control and dispose of all jet water in a manner satisfactory to the engineer or as specified in the contract documents. If jetting is specified or approved by the engineer and the jetting is performed as specified or approved, the contractor shall not be held responsible for any damage to the site caused by the jetting operations. If jetting is performed for the contractor’s convenience, the contractor shall be responsible for all damage to the site caused by jetting operations.

Jet pipes shall be removed when the bottom of the jet pipe is 5 feet above the minimum or prescribed toe elevation unless otherwise indicated by the contract documents or the Engineer. Following jet removal, the jetted pile shall be driven to the required nominal resistance with an impact hammer. If the required nominal resistance is not achieved at the prescribed toe elevation, the pile may be allowed to setup and the required nominal resistance determined through restriking. The jetting procedures should be reviewed by the Engineer and adjustments made if applicable so that the required nominal resistance can be achieved without restrike verifications.

When jetting is used, the Contractor shall submit details of the proposed jetting and pile driving plan. Where practical, all piles in a pile group shall be jetted to the required penetration depth before beginning pile driving. When large pile groups or pile spacing and batter make this impractical, restrike tests on a select number of previously driven piles shall be performed to check nominal resistance after jetting operations are completed.

7. Predrilling Equipment: When stated in the contract documents, the Contractor shall provide predrilling equipment to drill holes at pile locations of the size specified and to the depths shown in the contract documents or as approved in writing by the Engineer. If subsurface obstructions, such as boulders or rock layers are encountered, the diameter of the predrilled hole may be increased with Engineers approval to the least dimension adequate for pile installation.

Commentary:

The appropriate diameter of the predrilled hole depends on the purpose of the predrilled hole. If predrilling is performed to minimize problems with maintaining alignment tolerances, or to mitigate heave or vibrations, predrilled holes are typically smaller than the diameter or diagonal of the pile. When predrilling is performed to penetrate through an embankment or to bypass obstructions, a larger predrilled hole with a diameter not more than the largest dimension of the pile plus 6 inches may be acceptable. In either case, the excavated zone surrounding the pile is generally backfilled with an approved material of sand, pea gravel, or grout after the pile is driven depending on design requirements.

8. Spuds: When stated in the contract documents or approved by the Engineer, the Contractor shall provide spudding equipment to displace obstructions, debris, or unsuitable materials at pile locations. The spudding equipment shall create an opening through the material of the size specified and to the depths shown in the contract documents or as approved by the Engineer in writing.

14.2.5 SECTION X.05 DETERMINATION OF NOMINAL RESISTANCE

A) Axial Compression Resistance:

The nominal resistance of piles in axial compression shall be determined by the Engineer based on one of the methods listed below.

1. Static Load Tests: Compression load tests shall be performed by procedures set forth in ASTM D1143 using the quick load test method, except that the test shall be taken to geotechnical plunging failure or the capacity of the loading system. Testing equipment and measuring systems shall conform to ASTM D1143, except that the loading system shall be capable of applying 150 percent of the nominal resistance. A load cell and spherical bearing plate shall be used.

The Contractor shall submit to the Engineer for approval detailed plans prepared by a licensed professional engineer of the proposed loading apparatus. The submittal shall include calibrations for the hydraulic jack, load cell, and pressure gage conducted within 30 days of the load test. If requested by the Engineer, the jack, load cell, and pressure gage shall be recalibrated after the load test.

The loading apparatus shall be constructed to allow the various increments of the load to be placed gradually, without causing vibration to the test pile. When the approved method requires the use of reaction piles, the reaction piles shall be of the same type and diameter as the production piles. Reaction piles shall be surveyed and monitored for upward movement during the load test. Reaction piles driven at production pile locations that have a permanent upward movement of 0.25 inches or more upon completion of the load test shall be redriven. Timber or tapered piles installed in permanent locations shall not be used as reaction piles.

While performing the load test, the contractor shall provide safety equipment and employ adequate safety procedures. Adequate support for the load test plates, jack, and ancillary devices shall be provided to prevent them from falling in the event of a release of load due to hydraulic failure, test pile failure, or other cause.

The nominal geotechnical resistance or failure load of a pile statically tested in axial compression is defined by the pile head movement under load. For piles 24 inches or less in diameter or width, the failure load is the pile head load which produces a measured movement of the pile head equal to:

$$s_f = \Delta + \left(0.15 + \frac{\mathrm{b}}{120}\right) \qquad \text{Eq. 14-1}$$

Where:

s_f = pile head movement at failure (inches).
Δ = elastic deformation of total pile length (inches).
b = pile diameter or width of side for square piles (inches).

For piles larger than 36 inches in diameter, additional pile toe movement is necessary to develop the toe resistance. For these larger diameter piles, the nominal geotechnical resistance or failure load can be defined as the load which produces movement at the pile head equal to:

$$s_f = \Delta + \mathrm{b}/30 \qquad \text{Eq. 14-2}$$

Where:

s_f = pile head movement at failure (inches).
Δ = elastic deformation of total pile length (inches).
b = pile diameter or width of side for square piles (inches).

For piles greater than 24 inches in diameter but less than 36 inches in diameter, linear interpolation should be performed between Eq. 14-1 and 14-2.

The top elevation of the test pile shall be determined immediately after driving and again just before load testing to check for heave. If more than ¼ inch of heave occurs the load test pile may require re-driving before the load test is performed.

Unless otherwise specified in the contract documents or by the Engineer, the static load test shall not be performed sooner than 5 days after the test pile or any reaction piles were driven.

On completion of the load testing, any test or reaction piling not a part of the finished structure shall be removed or cut off at least 1 foot below either the bottom of footing or the finished ground elevation, if not located within the footing area.

Commentary:

The nominal resistance may increase (soil setup) or decrease (relaxation) after the end of driving. Therefore, it is essential that static load testing be performed after equilibrium conditions in the soil have re-established. Static load tests performed before equilibrium conditions have re-established will underestimate the long term nominal resistance in soil setup conditions and overestimate the long term nominal resistance in relaxation cases. For piles *in clays, specifications should require at least 2 weeks or longer to elapse between driving and load testing. In sandy silts and sands, 5 days to a week is usually sufficient. Load testing of piles driven into shales should also be delayed for at least 2 weeks after driving. Additional discussion on time dependent changes in nominal resistance may be found in Section 7.2.4.*

Each static load test pile should determine the load transferred to the pile toe. Instrumentation commonly consists of strain gages and/or telltale rods mounted at varying depths above the pile toe. Also, a load cell and spherical bearing plate should be mounted between the load frame and the pile head to verify the readings from the hydraulic jack pressure gauge. Due to jack ram friction, loads indicated by a jack pressure gauge are commonly 10 percent to 20 percent higher than the actual load imposed on the pile.

If the static load tests are to be performed by an independent firm retained by the Contractor and not by the Engineer, an additional specification section detailing the complete load test instrumentation monitoring requirements as well as the report submission requirements for the load test results and result interpretation must be added. A corresponding pay item must then be added to this specification for load test reporting. Alternatively, the report requirements can be described herein and then included as part of the static load test pay item.

When static load tests are used to control production pile driving, the time required to analyze and/or review the load test results as well as to establish driving criteria should be specified so that the delay time is clearly identified. Static load testing is discussed in greater detail in Chapter 9 of this manual.

2. Dynamic Testing with Signal Matching: Dynamic measurements shall be obtained using dynamic test processing equipment, calibrated transducers, and procedures set forth in ASTM D4945. The measurements will be taken by a qualified engineer during the driving of piles designated as dynamic test piles. Signal matching analysis shall be performed on representative data collected at the end of initial driving and at the beginning of each restrike events. Additional signal matching analysis may be performed as determined by the Engineer.

Commentary:

This section on dynamic testing covers only the Contractor's activities as they relate to the dynamic tests. If the dynamic tests are to be performed by an *independent firm retained by the Contractor and not by the Engineer, an additional specification section detailing the dynamic test analysis and reporting requirements must be added. Merely referencing the ASTM D4945 standard is insufficient. ASTM D4945 does not specify signal matching requirements or their frequency; it does not specify if, how, and by whom, driving criteria are established; nor does it identify what substructure locations are covered by the criteria.*

Dynamic testing personnel should have attained an appropriate level of expertise (Expert, Master, Advanced, Intermediate, Basic, or Provisional) on the "Dynamic Measurement and Analysis Proficiency Test" sponsored by the Pile Driving Contractors Association (PDCA) and Pile Dynamics, Inc. (PDI) for providers of dynamic testing services. Dynamic testing methods are discussed in Chapter 10.

Whenever static load tests are specified, dynamic tests are recommended to be performed on at least half the reaction piles prior to driving the static load test pile as well as on the static load test pile when it is driven. The dynamic test results are used both to confirm that the desired nominal resistance can be attained at the estimated static load test pile penetration depth and to fine tune the dynamic test procedures for site soil conditions. Dynamic monitoring of the static load test pile during restrike after completion of the static load test is also highly recommended. This restrike test allows correlation of static test results with dynamic test results. Signal matching analysis of dynamic test data is required for nominal resistance determination per AASHTO (2014) and also assists in quantifying the dynamic soil parameters, soil quake and damping, for the site.

When dynamic tests are specified on production piles, the first pile driven in each substructure foundation is typically tested. The total number of dynamic tests performed will vary from two piles per site condition, but not less than 2% of production piles, to 100% of the production piles. The number of dynamic tests required per AASHTO depends on the variability of the site conditions as well as the resistance factor selected for design verification. Additional discussion on the number of piles dynamically tested and the associated resistance factor can be found in Section 10.3.

Prior to placement in the leads, the Contractor shall make diametrically opposite faces of each pile to be dynamically tested available for predrilling the instrumentation attachment holes. The dynamic testing engineer will furnish the equipment, materials, and labor necessary for drilling holes in the piles for mounting the instrumentation. The instruments will typically be attached 2 to 3 pile diameters below the head of the pile with bolts placed in masonry anchors for the concrete piles, or through drilled holes on the steel piles, or with wood screws for timber piles.

The Contractor shall provide the dynamic testing engineer reasonable means of access to the pile for attaching instruments after the pile is placed in the leads. A manlift or platform with minimum size of 4 feet x 4 feet designed to be raised to the top of the pile while the pile is located in the leads shall be provided and operated by the Contractor. Alternatively, Contractor’s personnel following the dynamic testing engineer’s instructions can attach the instruments to the pile after it is placed in the leads. For some pile types and project conditions, the dynamic testing engineer may also recommend instrument attachment to the pile prior to lifting. It is estimated that approximately 1 hour per pile will be needed for instrument attachment and removal.

If requested, the Contractor shall furnish electric power for the dynamic test equipment. The power supply at the outlet shall be 10 amp, 115 volt, 55-60 cycle, A.C. only. Field generators used as the power source shall be equipped with functioning meters for monitoring voltage and frequency levels.

For dynamic testing conducted from a barge or other difficult to access sites, the Contractor shall furnish a shelter to protect the dynamic test equipment from the elements. The shelter shall have a minimum floor size of 8 feet x 8 feet and minimum roof height of 6.5 feet. The inside temperature of the shelter shall be between 45 and 95 degrees and be located within 50 feet of the pile location.

With the dynamic testing equipment attached, the Contractor shall drive the pile to the design penetration depth or to a depth determined by the Engineer. The Engineer will use the nominal resistance estimates at the time of driving and/or restriking from dynamic test methods to determine the required pile penetration depth for the nominal resistance. The stresses in the piles will be monitored during driving so that the measured stresses do not exceed the specified material values in Section X.02 B). If necessary, the Contractor shall reduce the driving energy transmitted to the pile by using additional cushions or reducing the energy output of the hammer in order to maintain stresses below the values in Section X.02 B). If non axial driving is indicated by dynamic test equipment measurements, the Contractor shall immediately realign the driving system.

The Contractor shall wait up to 24 hours (or a longer duration per Table 14-4 if specified in the contract documents) and restrike the dynamic load test pile with the dynamic testing instruments attached. It is estimated that the Engineer will require approximately 30 minutes to reattach the instruments. A cold hammer shall not be used for the restrike. The hammer shall be warmed up before restrike begins by applying at least 20 blows to another pile or to timber mats placed on the ground. The maximum amount of penetration required during restrike shall be 3 inches, or the maximum total number of hammer blows required will be 20, whichever occurs first. After restriking, the Engineer will either provide the cutoff elevation or specify additional pile penetration and testing.

Commentary:

A dynamic test often includes monitoring during both initial driving and during one restrike event within a specified time period. Alternatively, the initial driving and restrike test events can be individual items. Any long term restrike tests after the initial restrike should be paid as a separate item unless the restrike schedule is specifically stated in the dynamic test specification.

The restrike time and frequency should be clearly stated in the specifications and should be based on the time dependent strength change characteristics of the soil. Table 14-4 provides restrike durations commonly used for various soil types.

Table 14-4 Common Time Delay for Restrike Based Upon Soil Type

Soil Type	Time Delay Until Restrike
Clean Sands	1 Day
Silty Sands	2 Days
Sandy Silts	3-5 Days
Silty Clays	7-14 Days*
Shales	10-14 Days*

* Longer times sometimes required.

The restrike time interval is particularly important when dynamic testing is used for construction control. Specifying too short of a restrike time for friction piles in fine grained deposits may result in pile length overruns. However, it is sometimes difficult for long term restrikes to be accommodated in the construction schedule. In these cases, multiple restrikes are often specified on selected piles with shorter term restrikes at other locations.

The time necessary to analyze the dynamic test results and provide driving criteria to the contractor once restrikes are completed should also be stated in the specifications. This is important when the testing is done by agency personnel or their consultants as well as when the testing firm is retained by the contractor. In cases where the testing is retained by the contractor, the time required for agency review of the test results and to provide driving criteria should be specified relative to the agency's receiving the test results.

3. Rapid Load Tests: The nominal resistance shall be determined by a rapid load test performed in accordance with the procedures set forth in ASTM D7383. The rapid load test may be performed using either a combustion gas and reaction mass system or with a cushioned drop weight system. The peak force shall exceed the targeted nominal geotechnical resistance plus the dynamic soil resistance. The applied force pulse shall exceed 50% of the actual peak force for a time duration of 4L/C and the static pre-load force for a time duration of at least 12L/C where L is the pile length in feet and C is the pile wave speed in ft/s. A time duration of less than 12L/C is acceptable if additional force and movement measurements devices are used along the pile length in accordance with ASTM D7383.

The Contractor shall trim the head of the test pile flat, level, and perpendicular to the pile axis at the elevation directed by the rapid load testing agency. The area surrounding the test pile shall also be prepared in accordance with the requirements necessary to support the rapid load test device and conduct the test. Depending on the site conditions, this may necessitate site grading, use of crane pads, constructing a support frame, and/or other measures for positioning and supporting the rapid load test device.

The force shall be measured using a calibrated force transducer or load cell placed between the loading apparatus and the pile head. The load capacity of the transducer shall be at least 10% greater than the targeted peak force. The pile head movement shall be measured by one or more calibrated displacement transducers.

The nominal geotechnical resistance determined by a rapid load test shall be assessed by an approved interpretation procedure. The procedure shall identify the loading rate reduction factor used with the analysis method.

Commentary:

Section 11.4.1 through 11.4.5 describes nominal resistance interpretation methods for rapid load tests. The pile length and soil conditions are primary *factors in selecting the interpretation method. If additional force and movement devices are necessary for the interpretation method, time must be allotted for obtaining and installing this instrumentation in or on the test pile. A significant permanent pile head displacement is also needed for nominal resistance determination. The loading rate in a rapid load test also affects the nominal resistance and must be considered.*

AASHTO (2014) does not include a resistance factor for rapid load tests. Hence, this must be stipulated by the designer whenever rapid load tests are specified. A discussion of resistance factors currently in use with rapid load tests is provided in Section 11.4.7. All rapid load tests are performed by independent testing firms. Therefore, the time required for agency review of the rapid load test results should be specified relative to when the agency's receives the rapid load test results. A detailed discussion of rapid load test methods is provided in Chapter 11.

Unless otherwise specified in the contract documents or by the Engineer, the rapid load test shall not be performed sooner than 5 days after driving the test pile.

Commentary:

This specification addresses only the Contractor's activities as they relate to performing a rapid load test. If the rapid load tests are to be performed by an independent testing firm retained by the Contractor and not retained by the Engineer, an additional specification section detailing the complete rapid load test instrumentation monitoring requirements as well as the report submission requirements for the rapid load test results and result interpretation must be added. A corresponding pay item must then be added to this specification for rapid load test reporting. Alternatively, the report requirements can be described herein and then included as part of the rapid load test pay item.

The Contractor's independent testing firm shall submit to the Engineer, for approval, detailed plans of the rapid load test equipment arrangement, proposed test procedure, and nominal resistance interpretation method. The submittal shall also include calibrations for the transducer or load cells and all additional instrumentation. All calibrations shall be within the calibration period specified within ASTM D7383.

4. Wave Equation: The nominal resistance shall be determined by the Engineer based on a wave equation analysis. Piles shall be driven with the approved driving equipment to the ordered length or other lengths necessary to obtain the required nominal resistance. Jetting, predrilling, or other methods to facilitate pile penetration shall be modeled in the analysis if proposed and allowed by the contract documents. Adequate pile penetration depth for the nominal resistance shall be considered obtained when the wave equation penetration resistance is achieved within 5 feet of the estimated pile toe elevation. Piles not achieving the penetration resistance within this limit shall be driven to penetration depths established by the Engineer.

5. Dynamic Formula: The nominal resistance shall be determined by dynamic formula if the contract documents contain a provision that dynamic formula be used to establish driving criteria. Dynamic formulas should not be used if the required nominal resistance is greater than 600 kips. Formula results should not be considered applicable when the pile head is crushed, broomed, or damaged, or when a follower is used.

If a dynamic formula is used to establish driving criteria, piles shall be driven to a penetration depth necessary to obtain the nominal resistance according to the Modified Gates formula with specified units as follows:

$$R_{ndr} = 1.75\sqrt{E_d}\,\log_{10}(10\,N_b) - 100 \qquad \text{Eq. 14-3}$$

Where:

R_{ndr} = nominal driving resistance (kips).
E_d = developed hammer energy, (W)(h), during the observed set (ft-lbs).
W = ram weight (lbs).
h = average hammer stroke during set observation (ft).
N_b = number of hammer blows per inch (blows/in).

The number of hammer blows per foot of pile penetration required to obtain the nominal resistance shall be calculated as follows:

$$N_{ft} = 12\,(10^x) \qquad \text{Eq. 14-4}$$

In which:

$$x = \left[\frac{R_{ndr}+100}{1.75\sqrt{E_d}}\right] - 1 \qquad \text{Eq. 14-5}$$

Where:

N_{ft} = number of hammer blows for final foot of driving (blows/ft).
R_{ndr} = nominal driving resistance (kips).
E_d = developed hammer energy, (W)(h), during the observed set (ft-lbs).
W = ram weight (lbs).
h = average hammer stroke during set observation (ft).

Commentary:

Additional dynamic formulas besides the FHWA Modified Gates formula were presented in Chapter 13. Other dynamic formulas may be used based on agency practice and local calibrations.

Driven piles should be monitored in terms of their nominal driving resistance. The nominal driving resistance at a given pile penetration depth reflects the total resistance mobilized by the pile. This may include resistance in soil deposits unsuited for long term load support, as well as suitable layers. Therefore, the penetration resistance (blow count) should be established for the nominal driving resistance that must be overcome in order to reach anticipated pile penetration depth. These nominal driving resistances are determined by static analysis procedures.

In the case of piles to be driven to a specified minimum pile toe elevation, the nominal driving resistance must be computed by static analysis to include the resistance of all soil layers penetrated by the pile above the minimum pile toe elevation as well as the toe resistance at that depth. The minimum pile toe elevation may have been specified for reasons other than axial compression resistance in order to meet lateral, uplift, or serviceability requirements. The nominal driving resistance is directly related to the maximum pile driving stress during installation. The driving stress is more critical than the stress imparted after installation by the factored design load.

Good piling practices dictate use of the wave equation in place of dynamic formulas. AASHTO design specifications require a wave equation drivability analysis be performed during the foundation design stage. The soil profile used in this design stage wave equation analysis can be easily re-used along with details on the Contractors proposed driving system to obtain a construction stage wave equation analysis that includes the penetration resistance and maximum pile stresses for the required nominal driving *resistance. FHWA recommends that all agencies use wave equation analysis with a goal of minimizing use of dynamic formulas on all pile projects. Wave equation analysis is discussed in greater detail in Chapter 12 of this manual.*

B) Axial Tension Resistance:

The nominal resistance of piles in axial tension shall be determined by the Engineer based on one of the methods listed below.

1. Static Load Tests: Tension load tests shall be performed by procedures set forth in ASTM D3689 using the quick load test method, except that the test shall be taken to plunging failure or the capacity of the loading system. Testing equipment and measuring systems shall conform to ASTM D3689, except that the loading system shall be capable of applying 150 percent of the anticipated nominal resistance.

The Contractor shall submit to the Engineer for approval detailed plans prepared by a licensed professional engineer of the proposed loading apparatus. The submittal shall also include calibrations for the hydraulic jack, load cell, and pressure gage conducted within 30 days of the load test. If requested by the Engineer, the jack, load cell, and pressure gage shall be recalibrated after the load test.

The loading apparatus shall be constructed to allow the various increments of the load to be placed gradually, without causing vibration to the test pile.

When the approved method requires the use of reaction piles, the reaction piles shall be of the same type and diameter as the production piles. Reaction piles shall be surveyed and monitored for movement during the load test. Reaction piles driven at production pile locations that have a permanent upward displacement greater than 0.25 inch shall be redriven upon completion of the static load test. Timber or tapered piles installed in permanent locations shall not be used as reaction piles.

While performing the load test, the contractor shall provide safety equipment and employ adequate safety procedures. Adequate support for the load test plates, jack, and ancillary devices shall be provided to prevent them from falling in the event of a release of load due to hydraulic failure, test pile failure, or other cause.

The nominal geotechnical resistance or failure load of a pile statically tested in axial tension is defined by the pile head movement under load. The failure load is the pile head load which produces a measured upward movement of the pile head equal to:

$$s_f = \Delta + (0.15) \qquad \text{Eq. 14-6}$$

Where:

s_f = pile head upward movement at failure (inches).

Δ = elastic lengthening of the total pile length (inches).

The maximum factored resistance under tension loading is the tension load test failure load determined using Equation 14-6 multiplied by the resistance factor, ϕ_{up}, for a static load test in axial tension from Table 14-2.

Unless otherwise specified in the contract documents or by the Engineer, the static load test shall not be performed sooner than 5 days after the test pile or any anchor piles were driven.

On completion of the load testing, any test or anchor piling not a part of the finished structure shall be removed or cut off at least 1 foot below either the bottom of footing or the finished ground elevation, if not located within the footing area.

Commentary:

If the static load tests are to be performed by an independent firm retained by the Contractor and not by the Engineer, an additional specification section detailing the complete load test instrumentation monitoring requirements as well as the report submission requirements for the load test results and result interpretation must be added. A corresponding pay item must then be added to this specification for load test reporting. Alternatively, the report requirements can be described herein and then included as part of the static load test pay item.

It is essential that static load testing be performed after equilibrium conditions in the soil have re-established as discussed in the commentary of Section 14.2.5.1 A. The time required to analyze and/or review the load test results should be specified so that the delay time is clearly identified. Static load testing is discussed in greater detail in Chapter 9 of this manual.

2. Dynamic Testing with Signal Matching: Dynamic measurements following procedures set forth in ASTM D4945 will be taken by the Engineer during the driving of piles designated as dynamic test piles. Signal matching analysis should be performed on representative data collected at the end of initial driving and at the beginning of all restrike events. Additional signal matching analysis may be performed as determined by the Engineer. The maximum factored resistance for tension loading can be taken as the signal matching determined shaft resistance value multiplied by the resistance factor, ϕ_{up}, for dynamic testing in Table 14-2. Signal matching results from restrike tests should be used in this evaluation.

Commentary:

The commentary provided under the dynamic testing section for determination of the nominal axial compression resistance is applicable and should be reviewed for additional guidance.

C) Lateral Resistance:

The resistance and movement of piles subject to lateral loads shall be determined by the Engineer based on the following method.

1. Static Load Tests: Lateral load tests shall be performed by procedures set forth in ASTM D3966. Unless otherwise specified, the lateral load shall be applied incrementally following the standard loading procedure up to maximum applied load of 200% of the design lateral load. The lateral load shall be applied by a hydraulic jack acting between two piles, or between one pile and a reaction system. Testing equipment and measuring systems shall conform to ASTM D3966. A load cell and spherical bearing plate shall be used with the loading apparatus.

The Contractor shall submit to the Engineer for approval, detailed plans of the proposed loading apparatus prepared by a licensed professional engineer. The submittal shall also include calibrations for the hydraulic jack, load cell, and pressure gage conducted within 30 days of the load test. If requested by the Engineer, the jack, load cell, and pressure gage shall be recalibrated after the load test. The loading apparatus shall be constructed to allow the various increments of the load to be placed gradually, without causing vibration to the test pile.

Commentary:

ASTM D3966 provides a standard and 7 optional lateral load test procedures. All of the procedures have different loading schedules. Therefore, the loading procedure should be clearly identified so that the time required to conduct the lateral test is a function of the procedure.

If the lateral load tests are to be performed by an independent firm retained by the Contractor and not by the Engineer, an additional specification section detailing the complete lateral load test instrumentation monitoring requirements as well as the report submission requirements for the load test results and result interpretation must be added. A corresponding pay item must then be added to this specification for lateral load test reporting. Alternatively, the report requirements can be described herein and then included as part of the lateral load test pay item.

Unless otherwise specified or directed by the Engineer, lateral pile deflection measurements versus depth shall be obtained for each lateral load increment applied during the test using a string of in-place inclinometers, a Shape Accel Array, or other approved measure.

While performing the lateral load test, the contractor shall provide safety equipment and employ adequate safety procedures. The load test plates, jack, and ancillary devices shall be restrained to limit movement in the event of a sudden release of load due to hydraulic failure, test pile failure, or other cause.

14.2.6 SECTION X.06 PREPARATION AND DRIVING

A) Site Work:

1. Excavation and Fill Placement: Where practical, the site grade in the immediate work area shall be excavated to the specified elevation before the piles are driven. Material forced up between and adjacent to driven piles within the cap area shall be removed to the required elevation prior to pile cap concrete placement.

At bridge abutments and other locations directly adjacent to fill placement, fill material shall be placed and the fill settlement complete to the magnitude specified by the foundation designer before driving piles.

Commentary:

When approved by the Engineer, piles may be installed prior to embankment construction when minimal settlement or lateral displacement is expected in the soils beneath the embankment.

2. Predrilling for Driving: When required by the contract documents, the Contractor shall predrill holes of a size specified, at pile locations, and to the depths shown in the contract documents or approved in writing by the Engineer. After pile placement in the predrilled hole, the pile shall be driven with an impact hammer to the driving criteria specified by the Engineer. Any void space remaining around the pile in the predrilled zone after completion of driving shall be filled with sand or other approved material unless specifically prohibited or otherwise directed by contract documents. Material resulting from the drilled holes shall be disposed of as approved by the Engineer. The use of spuds shall not be permitted in lieu of predrilling, unless specifically approved in writing by the Engineer.

> **Commentary:**
>
> *Except for end bearing piles, predrilling shall be stopped at least 5 feet above the plan pile toe elevation and the pile shall be driven with an impact hammer to a penetration resistance criteria specified by the Engineer. Where piles are to be end bearing on rock or hardpan, predrilling may be carried to the surface of the rock or hardpan.*

If the Engineer determines that predrilling has disturbed the nominal resistance of previously installed piles, those piles that have been disturbed shall be restored to conditions meeting the requirements of this specification by redriving or by other methods acceptable to the Engineer. Redriving or other remedial measures shall be instituted after the predrilling operations in the area have been completed. The Contractor shall be responsible for the costs of any necessary remedial measures, unless the predrilling method was specifically included in the contract documents and properly executed by the Contractor.

> **Commentary:**
>
> *Augering, wet rotary drilling, or other methods of predrilling shall be used only when approved by the Engineer or in the same manner as used for any indicator piles or load test piles. When permitted, such procedures shall be carried out in a manner which will not impair the nominal resistance already in place or the safety of existing adjacent structures.*

3. Predrilling through Embankments: If required by contract documents, predrilled holes extending to natural ground shall be used where piles are to be driven through compacted fill or embankments greater than 5 feet in thickness. The predrilled hole shall have a diameter not more than the greatest dimension of the pile cross section plus 6 inches unless a different predrilled hole diameter is specified. Material resulting from the drilled holes shall be disposed of as approved by the Engineer.

B) Preparation of Piles for Driving:

1. Pile Heads: The heads of all piles shall be plane and perpendicular to the longitudinal axis of the pile before the helmet is attached. Precast concrete pile heads shall be flat, smooth, and perpendicular to the longitudinal axis to prevent eccentric impact from the helmet. Prestressing strands shall be cutoff below the surface of the end of the pile. The heads of all concrete piles shall be protected with a suitably thick pile cushion during driving as described in Section X.04 B) 3. For concrete and timber piles, the pile head shall be chamfered on all sides.

2. Collars and Bands: Timber pile heads shall be equipped with collars, bands, or other devices to protect against splitting and brooming of the pile head when timber piles are to be driven to a nominal resistance in excess of 200 kips or when driving conditions require them.

3. Pile Shoes and Closure Plates: Pile shoes and closure plates of the type and dimensions specified shall be provided and installed on piles when designated on the contract plans or specifications. Pile shoes for H-piles and open end pipe piles shall be fabricated from cast steel conforming to ASTM A148/A148M (Grade 90-60). End closure plates for closed end pipe piles shall be made of ASTM A36/A36M steel or better. The closure plate diameter and thickness shall be as specified by the Engineer. Shoes for timber piles shall be steel and shall be fastened securely to the pile. Timber pile toes shall be carefully shaped to secure an even uniform bearing on the pile shoe.

> **Commentary:**
>
> *H pile shoes composed of steel plates welded to the flanges and webs are not recommended because this reinforcement provides neither protection nor increased strength at the critical area of the flange to web connection. The designer should select and detail on the plans the proper pile shoe to suit the application. Additional information on pile shoes is presented in Chapter 16 of this manual.*

4. Pile Marking for Prior to Driving: The Contractor shall mark the piles in 1 foot increments beginning at the pile toe and continuing to the pile head. The cumulative distance from the pile toe shall be marked on the pile at 5 foot intervals from the pile toe. These cumulative distances shall be noted just above the corresponding foot marker. Prior to driving, the Contractor shall, if necessary, add inch marks between the 1 foot markers over a 10 foot length of pile as directed by the Engineer.

C) Pile Driving:

1. Test, Probe, and Indicator Piles: Test, probe, or indicator piles (hereafter referred to collectively as test piles), shall be driven before other piles are ordered. These piles shall be driven at the locations shown on the plans and to the penetration depths, penetration resistances (blow count) or nominal resistances as directed by the Engineer. In general, the specified length of test piles will be slightly greater than the estimated length of production piles in order to provide for variation in soil conditions. The driving equipment used for driving test piles shall be the approved system that the Contractor proposes to use on the production piling. Approval of driving equipment shall follow the requirements of in Section X.02 of these Specifications. The Contractor shall excavate the ground at each test pile location to the elevation of the bottom of the footing before the test pile is driven.

Test piles which do not attain the nominal driving resistance before reaching a distance of 1 foot above the estimated pile toe elevation on the plans shall be allowed to "set up" for 12 to 24 hours, or as directed by the Engineer, before being redriven. A cold hammer shall not be used for redrive. The hammer shall be warmed up before driving begins by applying at least 20 blows to another pile. If the specified nominal driving resistance is not attained on redriving, the Engineer may direct the Contractor to drive a portion or all of the remaining test pile length and repeat the "set up" redrive procedure. Test piles that have not achieved the required nominal driving resistance during redrive shall be spliced, if necessary, and driven until the required nominal driving resistance is obtained or as directed by the Engineer.

A record of driving of the test pile will be prepared by the Engineer, including the number of hammer blows per 1 foot intervals over the entire driven length, the as-driven length of the test pile, cutoff elevation, penetration in ground, and any other pertinent information. Near the end of initial driving, the Engineer may record the number of hammer blows per 1 inch of pile movement. If a redrive is necessary, the Engineer will record the number of hammer blows per 1 inch of pile movement for the first 1 foot of redrive. The Contractor shall not order piling to be used in the permanent structure until test pile data has been reviewed and pile order lengths are determined by the Engineer. The Engineer will provide the pile order list within 7 calendar days after completion of all test pile driving specified in the contract documents.

Commentary:

Test piles are recommended on projects where: 1) large quantities or long lengths of friction piling are estimated, even if load tests are to be used at adjacent footings; 2) large nominal soil resistance is expected in relation to the factored load and, 3) concrete piles are used.

The pile order lengths based on the test pile results should consider the anticipated variation in subsurface conditions. This is a particularly important consideration for concrete or timber piles which are both problematic if ordered shorter than required.

2. Production Piles: Production piles shall be driven by the Contractor to either the required nominal resistance, or the required nominal resistance and minimum pile toe elevation (if specified), or to a designated pile toe elevation. The pile penetration resistance (blow count) and the associated pertinent hammer performance observation for the hammer type (hammer stroke, impact velocity, hammer blow rate) should be documented during all driving sequences. Pile penetration resistance and the corresponding hammer performance documentation are required to evaluate the nominal resistance of impact driven piles.

Jetting or other methods shall not be used to facilitate pile penetration unless specifically permitted in the contract plans or in writing by the Engineer. The nominal resistance of jetted piles shall be based on the penetration resistances recorded during impact driving after the jet pipes have been removed. Jetted piles not attaining the nominal resistance at the ordered length shall be spliced, as required, at the Contractor's cost, and driven with an impact hammer until the nominal resistance is achieved using the approved driving criteria.

The nominal resistance of piles driven with followers shall be considered acceptable when the follower driven piles satisfy the driving criteria established by wave equation analysis and the criteria is either substantiated or modified based on a static load test or dynamic testing with signal matching.

The nominal resistance of piles driven with vibratory hammers shall be based on the penetration resistance recorded during impact driving after the vibratory equipment has been removed from the first pile in each group of ten piles. Vibratory installed piles not attaining the nominal resistance at the estimated pile toe elevation shall be spliced, at the Contractor's cost, and driven with an impact hammer until the nominal resistance is achieved. When the nominal resistance is attained, the remaining 9 piles shall be installed to similar pile toe elevation with similar vibratory hammer power consumption and rate of penetration as the first pile.

Approval of a pile hammer relative to material driving stress levels shall not relieve the Contractor of responsibility for piles damaged because of misalignment of the leads, deterioration of cushion materials, failure of splices, malfunctioning of the pile hammer, or other improper construction methods. Piles damaged for such reasons shall be rejected and replaced at the Contractor's expense when the Engineer determines that the damage impairs the strength of the pile.

3. Driving Stress: Compression and tension stresses occurring in the pile material during driving shall not exceed the maximum stress levels defined in Section X.02 B) unless otherwise specified in the contract documents or by the Engineer.

4. Installation Sequence: The order of placing and final driving of individual piles within pile groups shall be either starting from the center of the group and proceeding outwards in both directions or starting at the outside row and proceeding progressively across the group.

5. Pile Location, Alignment, and Cutoff Tolerance: Piles shall be driven with a variation of not more than 0.25 inches/foot (1:50) from the vertical or not more than 0.5 inches /foot (1:25) from the batter shown in the contract documents. Piles for trestle bents shall also be driven so that the bent cap may be placed in its proper location without adversely affecting the resistance of the piles.

The pile head location after driving shall be within 6 inches of plan location for all piles capped below final grade, and shall be within 3 inches of plan location for bents supported by piles.

No pile shall be closer than 4 inches from any edge of the pile cap. Any increase in pile cap dimensions or added reinforcing steel caused by out-of-tolerance or out-of-position piles shall be at the Contractor’s expense.

The final cutoff elevation of the pile head shall be no more than 1.5 inches above or more than 4 inches below the cutoff elevation shown in the plans. In addition, the pile shall have a minimum embedment into the pile cap of at least 8 inches.

If the location and/or tolerances specified in the preceding paragraphs are exceeded, the extent of overloading shall be evaluated by the Engineer. The cost of redesign shall be at the Contractor's expense.

Commentary:

Conditions exist, such as soft overburden soils directly overlying a sloping bedrock, where final pile location and/or alignment may be beyond the contractor's control. These cases should be identified during the design stage with specifications tailored to meet the site and project requirements. Tight tolerances of 3 inches or less are not practical.

6. Heaved Piles: Level readings to check for pile heave after driving shall be made by the Engineer at the start of pile driving operations and shall continue periodically until the Engineer determines that such checking is no longer required. If pile heave is observed, accurate level readings referenced to a fixed datum shall be undertaken by the Engineer on all piles immediately after installation and periodically thereafter as adjacent piles are driven to determine the magnitude of the pile heave. Piles that derive their nominal resistance predominant through end bearing shall be redriven if more than 0.25 inch of heave is measured. Piles that achieve their nominal resistance primarily through shaft resistance shall be redriven if more than 1.5 inches of heave is measured. If pile heave is detected on any piles that have been concrete filled, the piles shall be redriven to original position after the concrete has obtained sufficient strength. Redriving shall be done using a hammer-pile cushion system satisfactory to the Engineer. The Contractor shall be paid for all work performed in conjunction with redriving piles because of pile heave provided the initial driving was done in accordance with the specified installation sequence and approved pile installation plan.

7. Obstructions: If piles encounter unforeseeable, isolated obstructions, the Contractor shall be paid for the cost of obstruction removal and for all remedial design or construction measures caused by the obstruction. Obstruction removal is only practical when obstructions are located near the ground surface.

8. Practical and Absolute Refusal: Practical refusal is defined as a pile penetration resistance (blow count) of 10 blows per inch for a maximum of 3 consecutive inches of pile penetration. Absolute refusal is defined as 20 blows for one inch or less of pile penetration. Driving should terminate immediately if either of these criteria are achieved. In the case of hard rock, an absolute refusal criterion of 5 blows per ¼ inch or 10 blows per ½ inch may be preferred to reduce the risk of pile toe or driving equipment damage. The practical and absolute refusal definitions are based on the approved hammer system operating at its maximum fuel or stroke setting unless approval was established on hammer operation at a reduced fuel or stroke setting. When the required pile penetration depth cannot be achieved by driving without exceeding practical or absolute refusal with the approved hammer, use of other penetration aids such as predrilling or jetting should be evaluated.

Commentary:

Clear definitions for practical and absolute refusal are problematic. Factors such as the site soil profile, the characteristics of the bearing layer, pile type, hammer type, and hammer manufacturer limitations to prevent hammer damage all influence what is considered an acceptable refusal criteria. Many hammer manufacturers state that the hammer warranty is voided when pile penetration resistance (blow count) exceed 10 blows per inch for 6 to 12 consecutive inches of driving or 20 blows per inch for more than 1 inch.

When driving is easy until final driving, a high penetration resistance may be satisfactory. However, penetration resistances greater than 10 blows per inch should be used with care, particularly for concrete or timber piles. Extended driving at a penetration resistance greater than 10 blows per inch with a properly operating hammer should be avoided.

9. Pile Splices: Full length piles shall always be used wherever practical. Where splices are unavoidable for steel or concrete piles, their number, locations and details shall be subject to approval of the Engineer. In no case shall timber piles be spliced.

When splicing of steel piles is permitted, the method of splicing piles shall be in accordance with ANSI/AWS D1.1 or as approved by the Engineer. Either shielded arc or submerged arc welding should be used when splicing steel piles. Only certified welders shall perform welding. Mechanical splices that are not welded may be used only for compression piles.

Where splicing concrete piles is permitted, the concrete pile splice details shall conform to contract documents or as approved by the Engineer. Mechanical splices may be used if they satisfy all compression, tension, and bending resistance requirements of the design.

10. Pile Cutoff: The pile head of all permanent piles and pile casings shall be cutoff to a true plane at the required elevation and anchored to the structure as shown on the contract documents. All cutoff lengths shall become the property of the Contractor, and shall be removed from the site and disposed of properly.

For treated timber piles, a liberal application of copper naphthenate shall be given to the cut area immediately after cutoff. The copper naphthenate solution shall have a minimum of 2 percent copper metal and should be applied until visible evidence of further penetration has ceased.

Treated timber piles for marine applications exposed to weather shall be capped with a permanently fixed coating such as epoxy or with conical or other caps attached to the piles.

Commentary:

Additional structural details for timber, steel, concrete and cast in place piles should be included by each agency in this driven pile specification, either directly or by reference to appropriate sections of the individual agency's standard specification. Typical items include: timber pile butt treatment and preservative treatment; precast concrete pile reinforcement, forming, casting, curing, and handling; steel pile field painting; cast in place pile details for shell piles, interior reinforcement and concrete.

11. Unsatisfactory Piles: The methods used in transportation, handling, and driving piles shall not subject the piles to excessive stresses or abuse producing cracking, crushing or spalling of concrete piles; splitting, splintering, or brooming of the timber piles; or deformation of steel piles. A concrete pile will be considered defective if a visible crack, or cracks, appears around the entire periphery of the pile, or if any defect is observed which, as determined by the Engineer, affects the strength or service life of the pile. Misaligned piles shall not be forced into proper position. Any pile damaged during driving by reason of internal defects, or by improper driving, or by defective splicing, or driven out of its proper location, or driven below the designated cutoff elevation, shall be corrected by the Contractor, without added compensation, by a method approved by the Engineer.

Commentary:

Defective piles can often be remediated by:

- *withdrawing the defective pile and replacing it with a new and, when necessary, longer pile. In removing piles, jets may be used in conjunction with jacks or other devices for pulling in an effort to remove the whole pile, or*

- *driving a second pile adjacent to the defective pile.*

Piles driven past their specified top of pile elevation can:

- *be spliced or built up as otherwise provided herein, or*

- *have a sufficient portion of the footing extended down to properly embed the overdriven pile.*

Piles driven out of location can be remediated by:

- *driving one or more replacement piles adjacent to the out of position piles, or*

- *extending the footing laterally to incorporate the out of location pile, or*

- *redesigning and adding more reinforcing steel to the pile cap.*

14.2.7 SECTION X.07 METHOD OF MEASUREMENT

A) Mobilization of Pile Driving Equipment:

Payment for mobilization will be made at the lump sum price bid for this item as follows: Seventy five percent (75 percent) of the amount bid will be paid when the equipment for driving piles is mobilized, assembled, and driving of satisfactory piles has commenced. The remaining 25 percent will be paid when the work of driving piles is completed. The lump sum price bid shall include the cost of furnishing all labor, materials and equipment necessary for transporting, erecting, maintaining, replacing any ordered equipment, dismantling and removing of the entire pile driving equipment.

The cost of all labor, including the manipulation of the pile driving equipment and materials in connection with driving piles, shall be included in the unit price bid for the piles to be driven.

B) Piles Furnished:

The quantity of piles furnished will be measured for payment using the number of piles furnished (per linear foot or each) in accordance with the contract documents and shall include full compensation for all costs involved with furnishing and delivering piles to the project site in the unit price bid for furnished piles.

When pile build-ups or extensions are necessary, the extension length, approved by the Engineer, will be included in the total length of piling furnished.

No allowance will be made for that length of piles furnished by the Contractor to replace piles which were previously accepted by the Engineer, but were subsequently damaged by the Contractor prior to completion of the contract.

C) Piles Driven:

The quantity of piles driven will be measured for payment using the number of piles driven (per linear foot or each) in accordance with the contract documents and shall include full compensation of all costs involved in the actual driving of the piles as well as all related pile costs for which compensation is not identified as a specific pay item. These related pile costs shall include furnishing all labor, equipment, and materials necessary to install and complete the pile. All costs shall be included in the unit price bid for the piles driven.

D) Test Piles Furnished:

The quantity of test piles furnished will be measured for payment using the number test piles furnished (per linear foot or each) in accordance with the contract documents and shall include full compensation of all costs involved with furnishing and delivering test piles to the project site in the unit price bid for furnished test piles.

E) Test Piles Driven:

The quantity of test piles driven will be measured for payment using the number of test piles driven (per linear foot or each) in accordance with the contract documents and shall include full compensation of all costs involved in the actual driving of test piles as well as all related test pile costs for which compensation is not identified as a specific pay item. These related test pile costs shall include furnishing all labor, equipment, and materials necessary to install and complete the test pile. All costs shall be included in the unit price bid for the test piles driven.

F) Static Pile Load Test:

The quantity of static load tests for payment will be measured by the number of load tests completed and accepted. This pay item shall include all labor, equipment, and materials needed to construct and perform the static pile load test in accordance with contract documents.

Reaction and test piling which are not a part of the permanent structure will be included in the unit price bid for each load test. Reaction and test piling, which are a part of the permanent structure, will be paid for under the appropriate pay item.

Static load tests performed at the option of the Contractor will not be included in the quantity measured for payment.

G) Dynamic Pile Load Test:

The quantity of dynamic pile tests for payment will be measured by the number of dynamic pile tests completed and accepted in accordance with the contract documents. The pay item for dynamic pile test (during driving), or dynamic pile test (during restrike) shall include full compensation for furnishing all labor, equipment, and materials necessary to perform the dynamic pile test as specified in the contract documents. If the dynamic test is performed at a test pile location (non-production location), the unit price for test piles furnished and test piles driven will be paid in addition to the unit price for the dynamic pile test (during driving), or dynamic pile test (during restrike). If the dynamic test is performed at a production pile location, the unit price for piles furnished and piles driven will be paid in addition to the unit price for the dynamic pile test (during driving), or dynamic pile test (during restrike).

If an unspecified dynamic pile test requires substantial repositioning, delay, or downtime of the pile driving rig (such as may occur for a second longer term restrike on a test pile conducted at the owner’s request), additional compensation shall be paid at the unit price per hour for the out of sequence move, delay, or downtime in addition to the applicable unit bid prices for dynamic pile test, test piles, or piles.

H) Rapid Pile Load Test:

The quantity of rapid load tests for payment will be measured by the number of load tests completed and accepted. This pay item shall include all labor, equipment, and materials needed to prepare the site for the test, construct and perform the test per the contract documents.

I) Lateral Load Test:

The quantity of lateral load tests for payment will be measured by the number of lateral load tests completed and accepted. When contract documents stipulate that two piles are to be laterally load tested by pushing apart or pulling together two piles, the lateral load test pay quantity shall comprise the lateral load testing of both piles. This pay item shall include all labor, equipment, and materials needed to prepare the site for the test, construct and perform the lateral load test per the contract documents.

J) Splices:

The quantity of splices measured for payment shall be only those splices actually made that were required to drive the piles in excess of the plan estimated and approved pile lengths as accepted for payment by the Engineer. The unit price bid per splice shall comprise full compensation for procurement, delivery, and attachment of the splice including all labor, equipment, and ancillary materials.

K) Pile Shoes:

The quantity of pile shoes measured for payment shall be those shoes actually installed on piles and accepted for payment by the Engineer. The unit price bid per pile shoe shall comprise full compensation for procurement, delivery, and attachment of the shoes including all labor, equipment, and ancillary materials.

> **Commentary**
>
> *Pile shoes can be alternatively be included in the furnished pile price if clearly stated on the plans and in the contract documents.*

L) Predrilling:

The quantity of predrilling measured for payment shall be taken to include full compensation for providing all labor, equipment, and materials necessary to perform the predrilling work in accordance with the contract documents.

M) Jetting

The quantity of jetting measured for payment shall be taken to include full compensation for providing all labor, equipment, and materials necessary to perform jetting work in accordance with the contract documents.

N) Pile Cutoff

The quantity of pile cutoffs measured for payment shall be taken to include full compensation for providing all labor and equipment needed to provide a plane and level pile head surface at the required cutoff elevation as specified by the contract documents. The contract unit price also includes full compensation for proper disposal of the cutoff material.

O) Spudding

The quantity of spudding measured for payment shall be taken to include full compensation for providing all labor, equipment, and materials necessary to perform the spudding work in accordance with the contract documents.

P) Delays, Downtime, Rig Moves

The quantity of time measured for payment for delays, downtime, or out of sequence pile driving rig moves caused by the owner, agents, or subcontractors not otherwise compensated in contract pay items shall be recorded for each claimed occurrence and shall be taken to include full compensation for all labor and equipment.

14.2.8 SECTION X.08 BASIS OF PAYMENT

The accepted quantities, determined as described above, will be paid for at the respective contract document price per unit of measurement for each of the general pay items listed below. Payment will be made for each size and type of pile shown in the contract documents. Prices and payment will be full compensation for the work prescribed by the contract documents. Pay item and pay units are described below in Table 14-5.

Table 14-5 Basis of Payment

Pay Item	Pay Unit
Mobilization and Demobilization	Lump sum
Piles Furnished	Linear foot or each
Piles Driven	Linear foot or each
Test Pile Furnished	Linear foot or each
Test Pile Driven	Linear foot or each
Static Load Test	Each
Dynamic Pile Test (during driving)	Each
Dynamic Pile Test (during restrike)	Each
Rapid Load Test	Each
Lateral Load Test	Each
Pile Splices	Each
Pile Shoes	Each
Predrilling	Linear foot or Each
Jetting	Linear foot or Each
Pile Cut-off (over 5 ft lengths only)	Each
Spudding (Punching)	Per hour
Delays, Downtime, or Out-of-Sequence Moves	Per hour

Commentary:

The pile payment items in Table 14-5 have been chosen to separate the major fixed costs from the variable costs. Many agencies oversimplify pile payment by including all costs associated with the driving operation in the price per foot of pile installed. Contractors bidding such "simple" items need to break down the total cost of the mobilization, splices, shoes, etc., to a price per linear foot based on the total estimated quantity. If that quantity underruns, the contractor does not recover the full cost of mobilization, splices, shoes, etc. If that quantity overruns, the agency pays an unfair price for the overrun quantity. The use of separate items for operations of major fixed cost such as mobilization can substantially mitigate the inequitable impact of length variations. Similarly, the ordered pile length is the agency's responsibility. Separate payment for furnishing piles and driving piles compensates the contractor for actual materials used and installation costs, even when modest overruns or underruns occur.

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CHAPTER 15

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 a pile's nominal geotechnical 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.

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. This chapter presents a basic description of each component of a driving system. For additional guidance, readers are referred to pile driving equipment manufacturers and suppliers. Unless otherwise noted, photographs are provided courtesy of GRL Engineers, Inc.

15.1 CRANES

The most common crane used for pile driving has historically been the crawler crane. Crawler cranes have very good mobility for most site conditions, good stability due to their wide base, and typically require minimal effort when walked and repositioned on the job site. A hydraulic power pack unit or air compressor may be mounted as a crane counterweight to facilitate pile driving operations. Crawler cranes also have a 360 degree hoisting radius. Crawler cranes must be trucked to the job site in pieces and assembled by another crane. Firm ground conditions are required for movement and operation. In soft ground conditions, the use of crane mats may be required. A photograph of a typical crawler crane set up for pile driving is presented in Figure 15-1.

Truck or mobile cranes are also used as pile driving rigs. Mobile cranes have good mobility on many job sites and can travel to the job under their own power. Setup and breakdown are relatively simple. However, mobile cranes cannot walk with heavy loads. Therefore, mobile crane movement between substructure locations

Figure 15-1 Crawler crane pile driving rig with lattice boom and swinging leads.

usually requires laying down and un-attaching the hammer and leads, moving the crane, and then re-attaching the hammer and lead system. Outriggers are used on mobile cranes for stability and leveling. The power pack unit or air compressor cannot be crane mounted. Mobile cranes also have restricted side lifting capabilities. A photograph of a mobile crane set up for pile driving is presented in Figure 15-2.

Figure 15-2   Truck crane pile driving rig with lattice boom and swinging leads.

Rough terrain cranes can also be used for pile driving. They can be set up fast and easily on a job site, they can be relocated and releveled relatively quickly, and can walk in some cases with loads over the front. When on location, outriggers are used for stability and leveling. Rough terrain cranes have greater mobility than truck cranes in difficult terrain. However, they have very restricted side lifting capabilities and a hammer power source cannot be crane mounted. A photograph of a rough terrain crane set up for pile driving is presented in Figure 15-3.

Figure 15-3 Rough terrain pile driving rig with telescoping boom and offshore leads.

15.2 DEDICATED AND UNIVERSAL RIGS

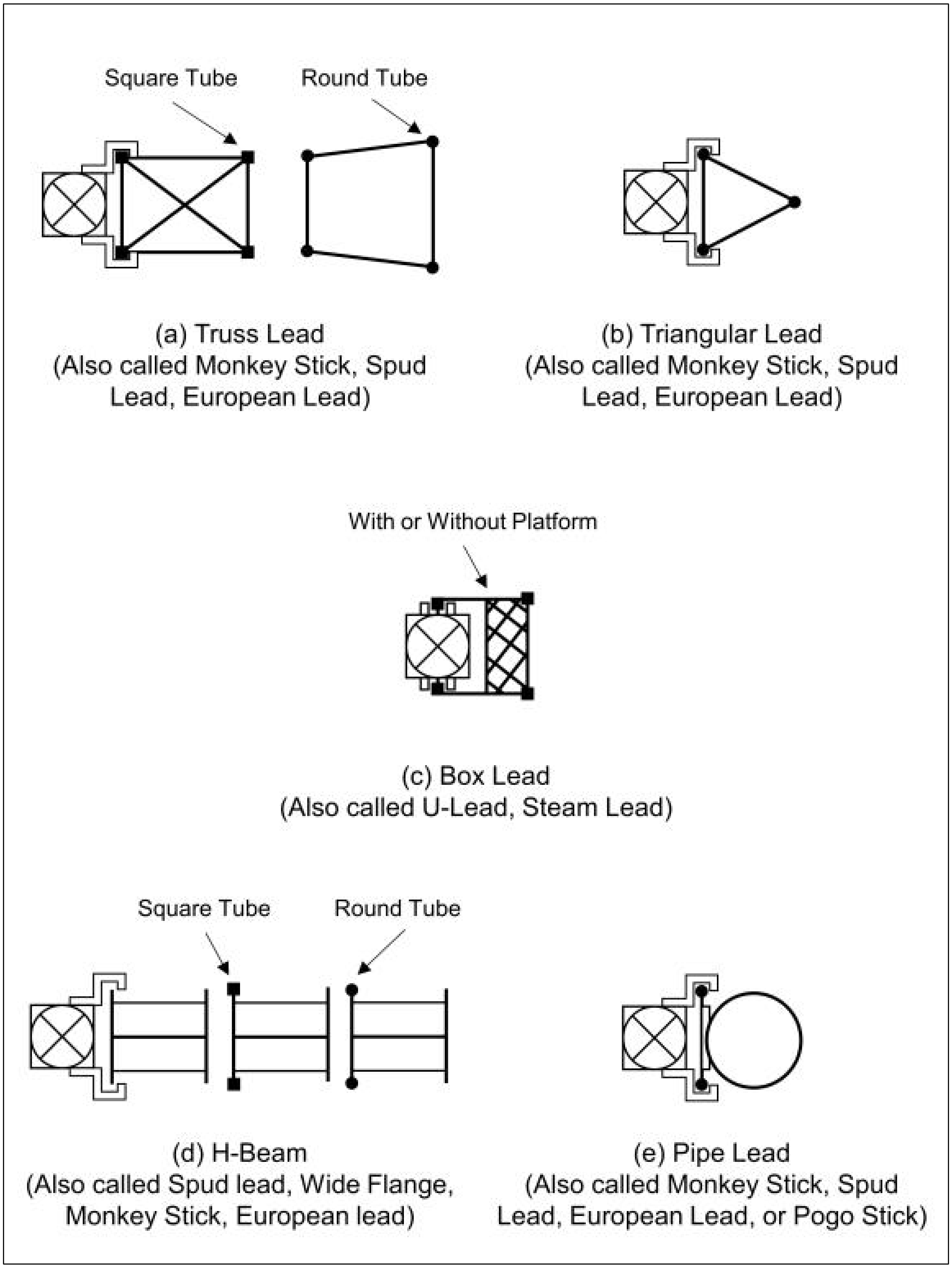
In recent years, use of dedicated and universal rigs has increased on pile driving projects. These specialty pile driving rigs or universal rigs include an attached telescoping leader that can be quickly transitioned from their horizontal transport position to the vertical driving position using hydraulically operated support arms. Hence, system setup is relatively fast without the need of additional heavy equipment. These systems include winch lines that can be used to offload piles as well as loft piles into position beneath the hammer prior to pile driving. A dedicated pile driving system equipped with a hydraulic pile hammer is presented in Figure 15-4, while a universal rig setup for driving piles with a diesel pile hammer is shown in Figure 15-5.

Figure 15-4 Dedicated pile driving system.

Figure 15-5 Universal rig setup for pile driving (courtesy Bauer-Pileco).

15.3 LEADS

Lead systems are used with crawler, truck, and rough terrain pile driving rigs. 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. Typical lead types are illustrated in Figure 15-6. The most common lead type is the box lead. Leads can be configured for use with the pile driving rig as a swinging lead, fixed lead, semi-fixed or vertical travel lead, or as an offshore lead. The most widely used lead configuration is a swinging lead depicted in Figure 15-7. Swinging leads are widely used because of their simplicity, lightness and low cost. Leads can be hung from



(a) Truss Lead
(Also called Monkey Stick, Spud Lead, European Lead)

(b) Triangular Lead
(Also called Monkey Stick, Spud Lead, European Lead)

(c) Box Lead
(Also called U-Lead, Steam Lead)

(d) H-Beam
(Also called Spud lead, Wide Flange, Monkey Stick, European lead)

(e) Pipe Lead
(Also called Monkey Stick, Spud Lead, European Lead, or Pogo Stick)

Figure 15-6   Typical lead types (after DFI Publication 1981).

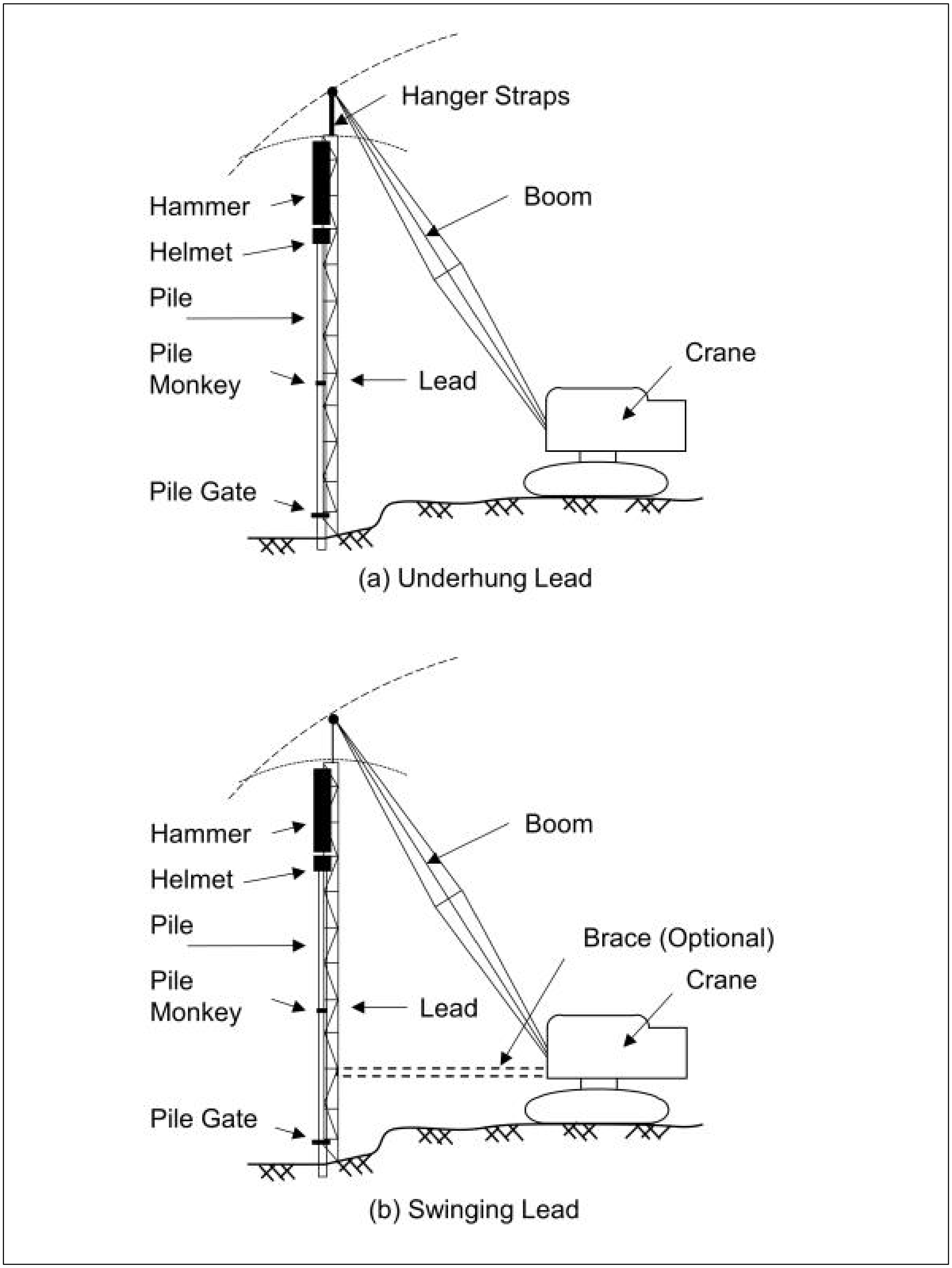


Figure 15-7 Swinging lead systems (after DFI Publication 1981).

the boom with hanger straps as illustrated in Figure 15-7(a) with the hammer held by a crane line. The most common arrangement however, is shown in Figure 15-7(b) where the leads and hammer are held by separate crane lines. 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.

Photographs of swinging lead arrangements on pile driving rigs were previously presented in Figure 15-1 and 15-2. A stabbing guide is located at the bottom of a swinging lead system as illustrated in Figure 15-8. Pile location and alignment are controlled by positioning the lead system over the pile location, lifting the leads, and then dropping the leads so that the stabbing guide at the bottom of the leads penetrates into the ground. The crane boom is then manipulated until the specified verticality or batter angle is achieved.

Pile driving specifications have historically penalized or prohibited swinging leads. This general attitude is not justified based on currently available equipment. 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. As noted earlier, the function of a lead is to hold the pile in good alignment with the driving system in order to prevent pile 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 (Figure 15-9). 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.

Fixed leads are connected to the boom point and to the crane frame using a spotter or brace that runs from the bottom of the leads to the crane. A schematic of a typical fixed lead system is depicted in Figure 15-10(a) and a photograph is presented in Figure 15-11. A fixed lead system attempts to hold the pile in true alignment while driving but may require more time for rig repositioning prior to driving.

Figure 15-8 Stabbing guide at bottom of box leads.

Figure 15-9 Pile gate with latch for use on truss lead (courtesy Berminghammer).

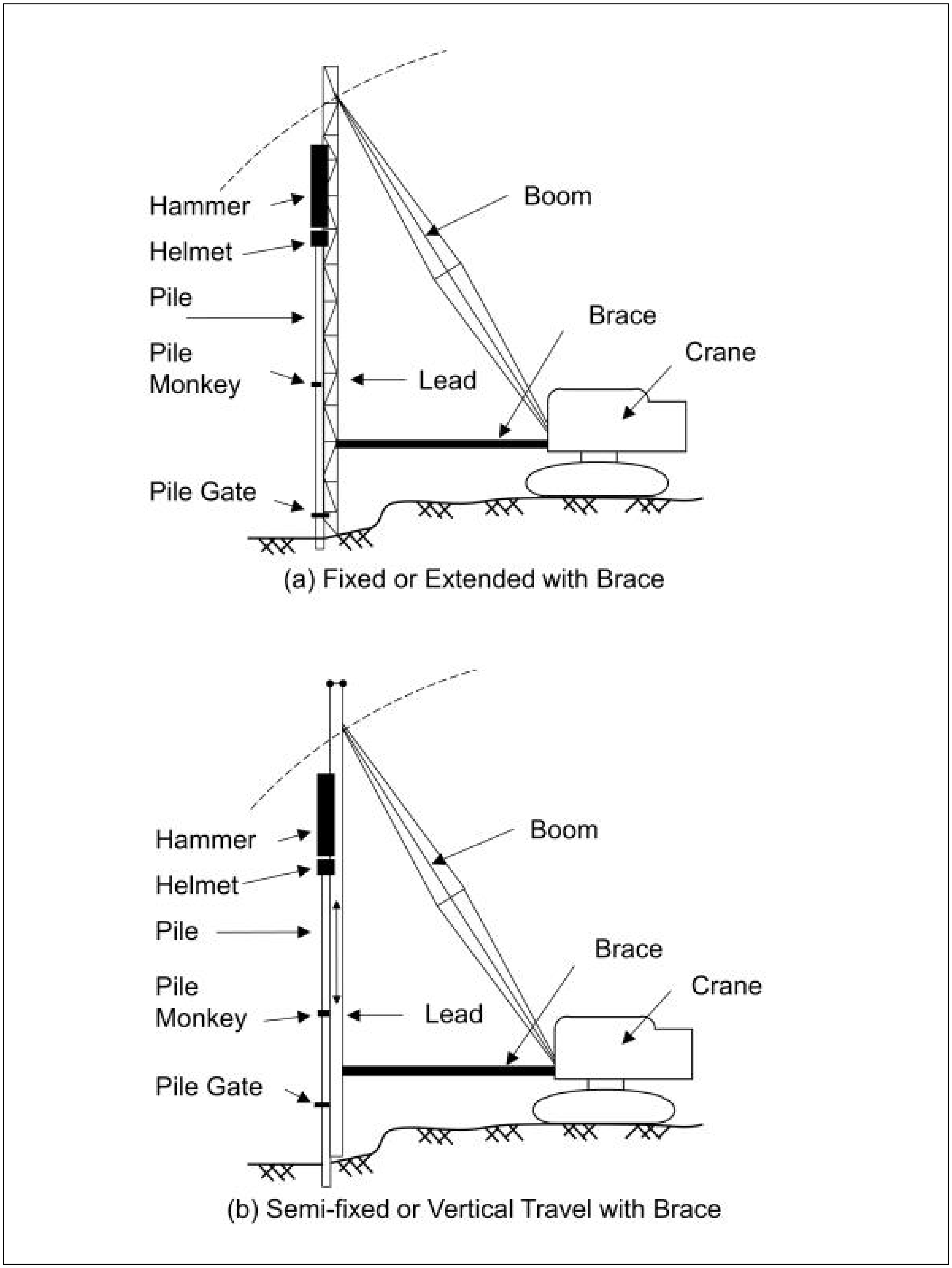


Figure 15-10 Fixed lead systems (after DFI Publication 1981).

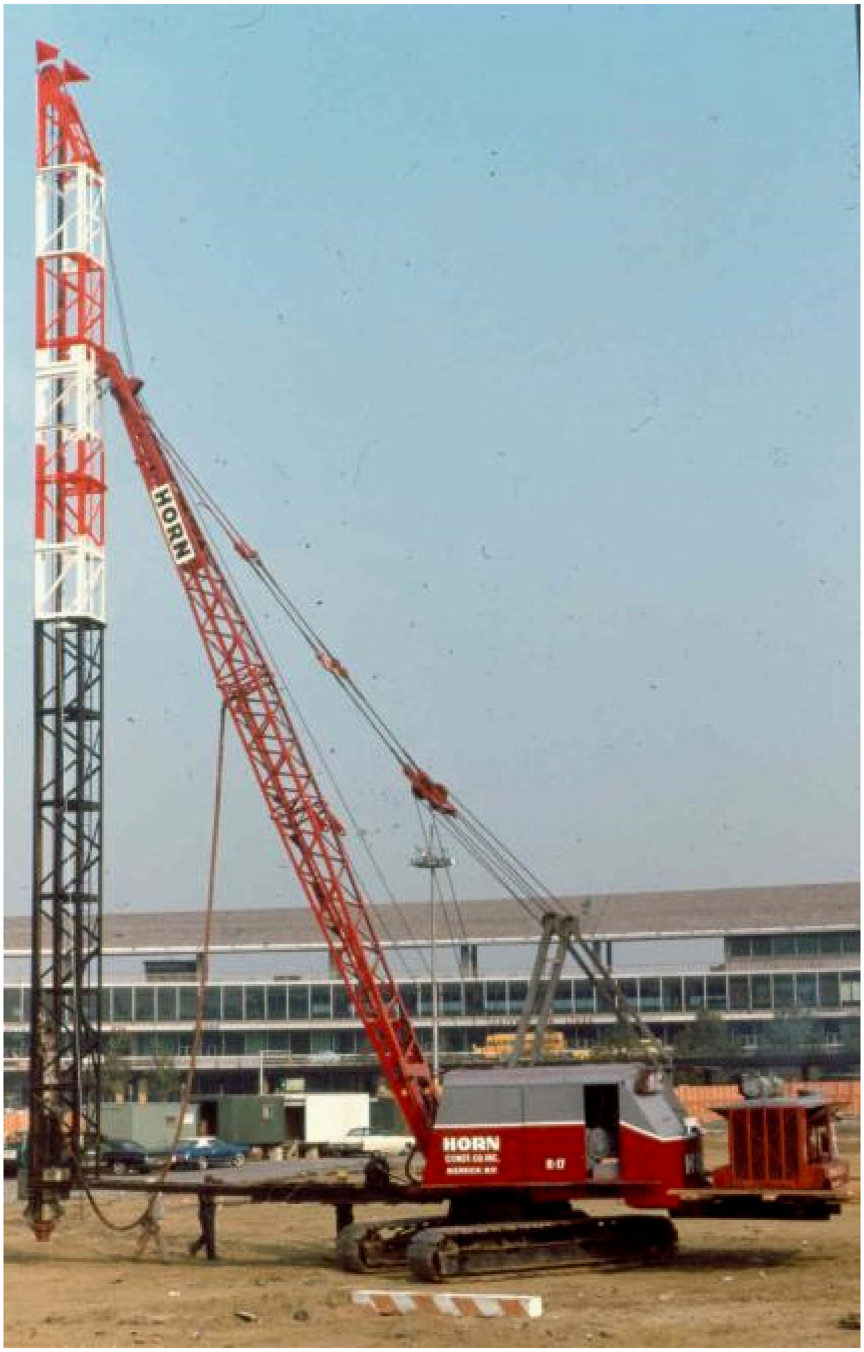


Figure 15-11 Fixed lead system with air compressor mounted as counterweight (courtesy FHWA Demo 66).

As an alternative to a fixed lead system, a semi-fixed or vertical travel lead may be used. A semi-fixed lead, as shown in Figure 15-12, allows vertical lead movement at the lead connection points to the boom and brace which the standard fixed lead system does not.

Figure 15-12 Vertical travel lead system (courtesy Berminghammer).

Figure 15-13(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 spotter or brace as depicted in Figure 15-13(b).

Offshore leads are similar to swinging leads in that they are free to rotate sufficiently to align the hammer and pile head, however they do not require precise alignment of the crane with the pile head, and they generally consist of only a short lead section with a pile guide at the base. An offshore lead schematic is depicted in Figure 15-14. The short lead section is intended only to hold and axially align the hammer with the pile head, and does not provide support for batter or full pile alignment during the driving process. When offshore leads are used, a template is constructed to hold the pile in position during driving. Section 15.3 provides a further discussion on templates. A photograph of an offshore lead with guide is shown in Figure 15-15 and of an offshore lead system in use with a rough terrain crane in Figure 15-3.

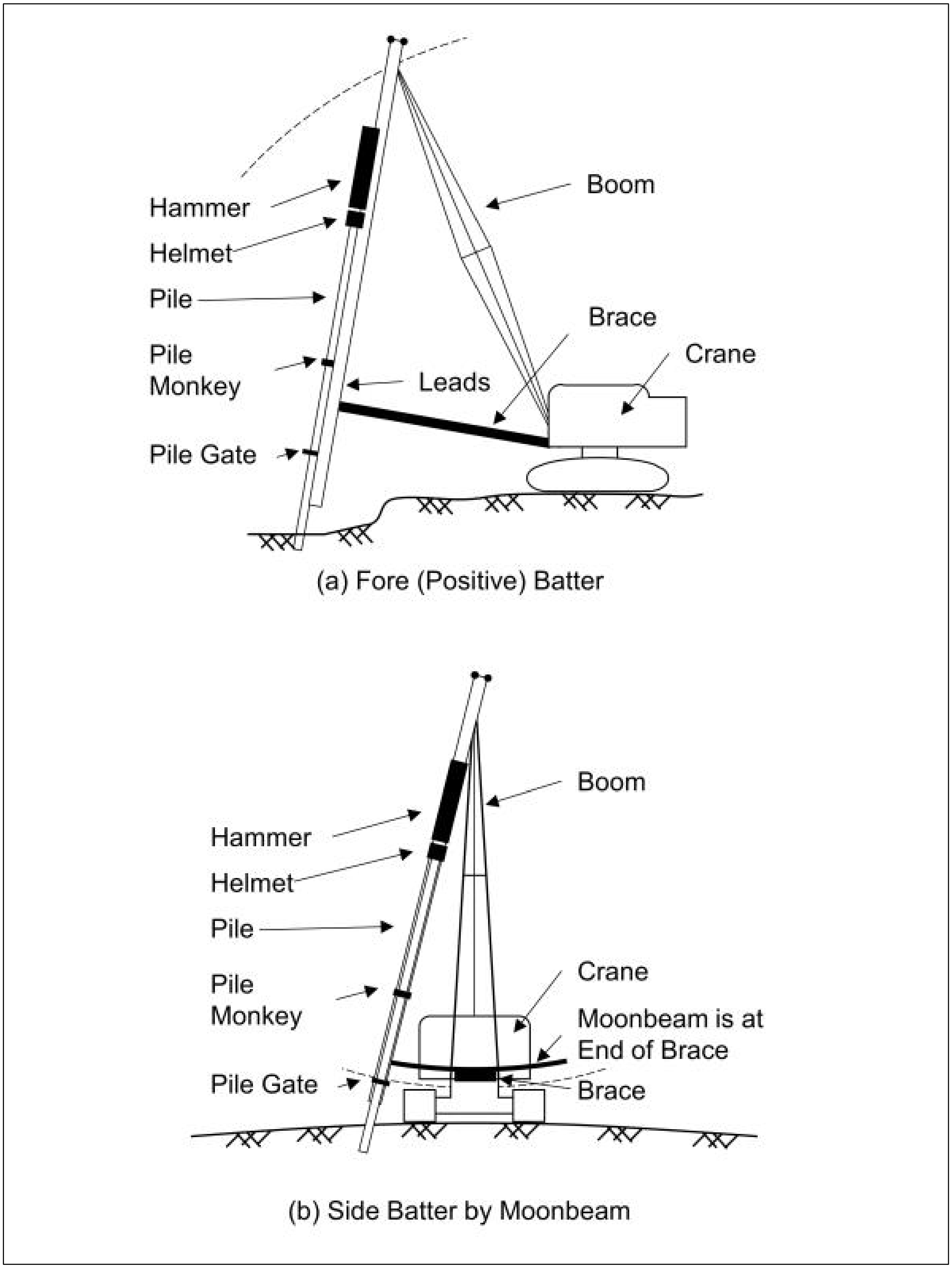


Figure 15-13 Lead configuration for batter piles (after DFI publication 1981).

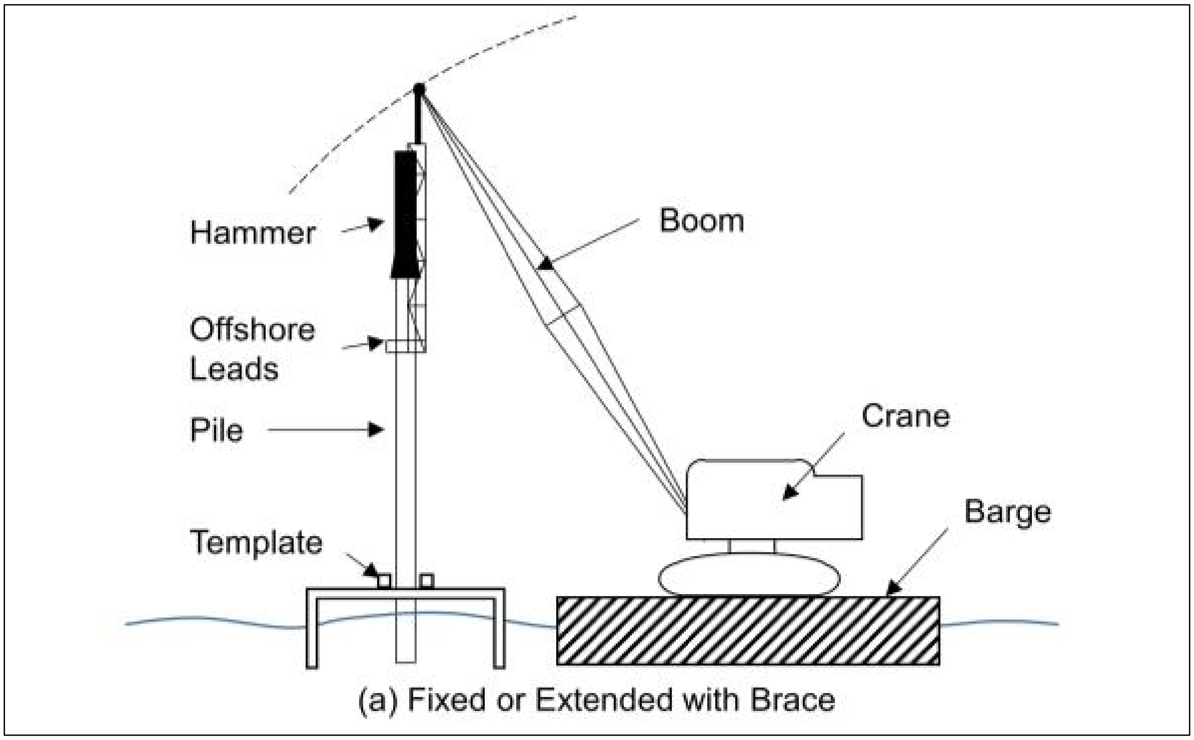


Figure 15-14 Typical offshore lead configuration.


Figure 15-15 Offshore lead and pile guide (left) with pipe pile helmet (right) (courtesy Berminghammer).

Regardless of lead type chosen, the pile must be kept in good alignment with the hammer to avoid eccentric impacts which can 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.

15.4 TEMPLATES

Templates are required to hold piles in proper position and alignment when an offshore or swinging lead system is used over water or excavations. The top of the template should be located within 5 feet 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 15-16. When templates include batter piles, it must be remembered that the correct location for the batter piles in the template arrangement will vary depending upon the template elevation relative to the pile cutoff elevation. For example, consider a template located 5 feet above pile cutoff elevation. If the plan pile locations at cutoff are used at the template elevation, a 1H:4V batter pile would be 15 inches out of location at the pile cutoff elevation. This problem is illustrated in Figure 15-17. Template construction should also allow the pile to pass freely through the template without binding. Templates with rollers are preferable, particularly for batter piles.

Figure 15-16 Template system.

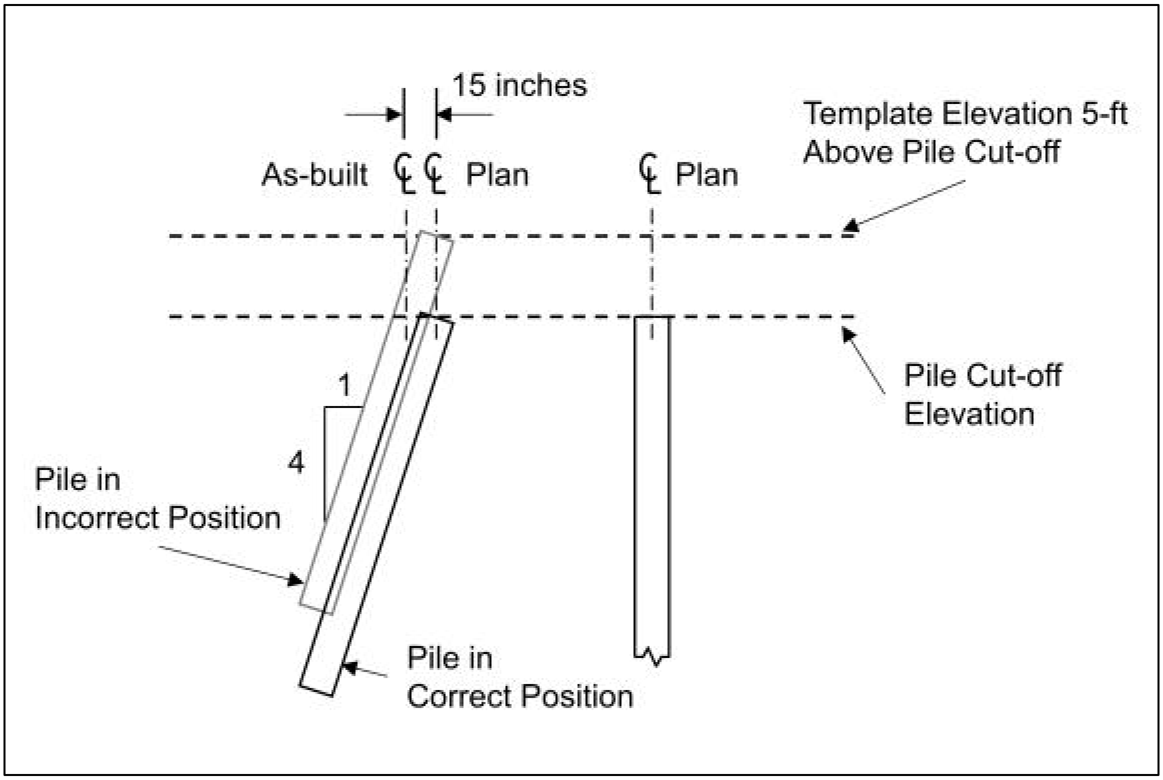
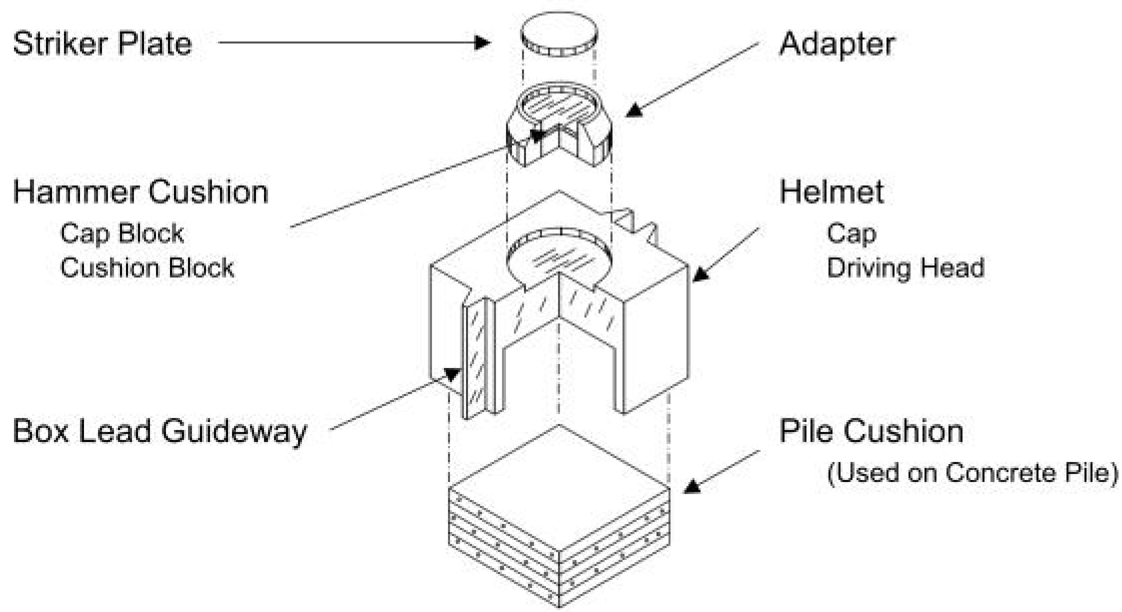


Figure 15-17 Template elevation effects on batter piles (after Passe 1994).

15.5 HELMETS

Figure 15-18 shows the components of a typical helmet and the nomenclature used for associated components. The helmet configuration and size used depends upon the lead type, pile type and the type of hammer used for driving. Helmets may be a one piece unit manufactured for driving a specific pile type and size, or may consist of two pieces consisting of a base helmet with an insert to accommodate various pile types and sizes. 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 or insert should fit loosely, but not so loosely as to prevent the proper alignment of hammer and pile. Helmets or inserts should be approximately 0.1 to 0.2 inches larger than the pile diameter. Proper hammer-pile alignment is particularly critical for precast concrete piles.

In Figure 15-19 a photograph of a helmet that cracked during driving provides a unique cross sectional view illustrating a typical striker plate, hammer cushion (consisting of two aluminum plates and one micarta plate), and helmet configuration. Photographs of one piece helmets for a concrete pile and pipe pile are presented in Figure 15-20(a) and 15-20(b), respectively.



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 inside of the helmet to change from size to size and shape to shape.

Figure 15-18 Helmet components (after DFI Publication 1981).

Figure 15-19 Cross section of cracked helmet with striker plate and aluminum and micarta hammer cushion (courtesy of WKG[2]).

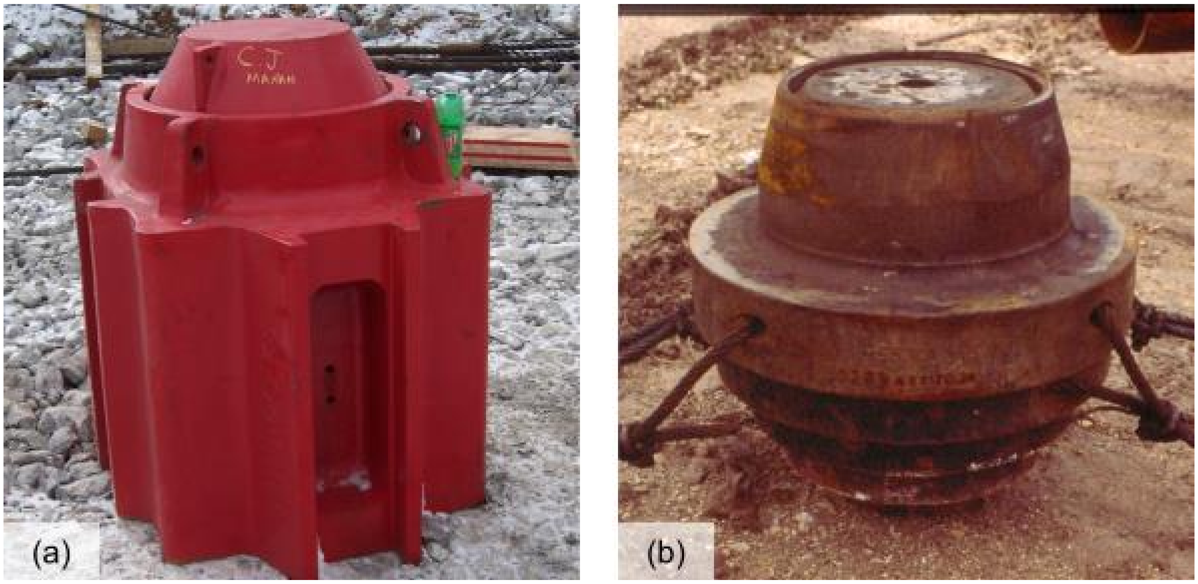


Figure 15-20 One piece helmets for (a) concrete pile and (b) pipe pile.

A two piece system consisting of a base or primary helmet and an insert can also be used to adapt a base helmet for driving different pile types. A base helmet with an insert for driving H-piles is presented in Figure 15-21(a) and a photograph showing several pipe pile inserts to accommodate different pipe pile diameters and wall thickness on the same project is presented in Figure 15-21(b).

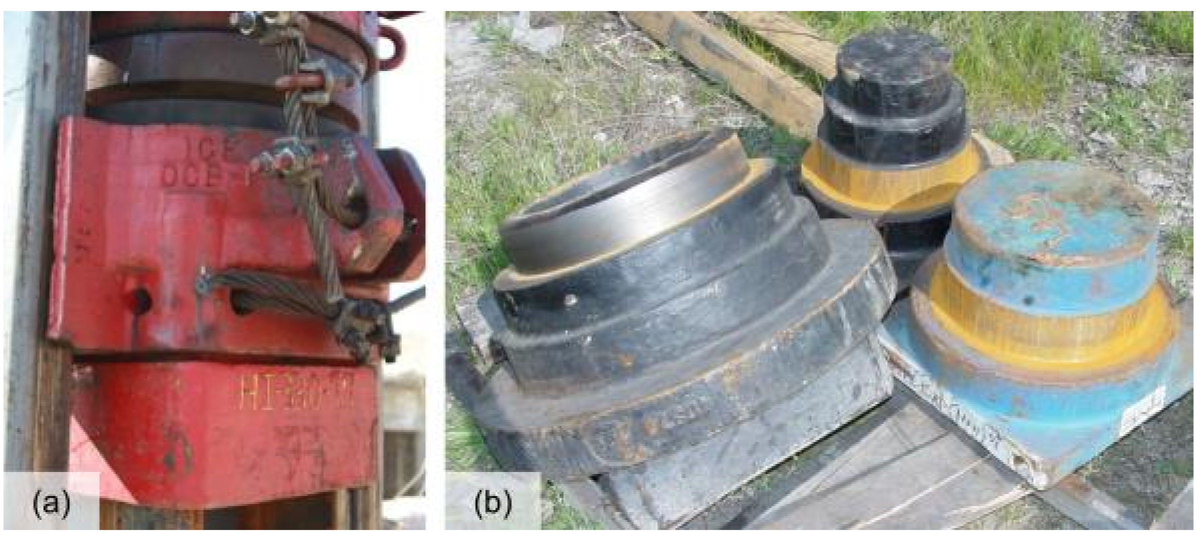


Figure 15-21 Inserts for (a) H-pile and (b) varying pipe pile diameter and wall thickness.

15.6 HAMMER CUSHIONS

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 can cause damage to hammer striking parts, anvil, helmet or pile. Hammer cushion materials are usually proprietary man-made materials such as micarta, nylon, urethane or other durable polymers. Over time, hammer cushion materials become compressed and stiffen as additional hammer impacts are applied. Therefore, hammer cushions eventually become ineffective, or may result in significant reduction in transferred energy or increased bending stress. Proprietary hammer cushions may last for up to 200 hours of driving.

In the past, a commonly used hammer cushion was made of hardwood (one piece), approximately 6 inches 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 hammer cushions from durable man-made materials, hammer cushions consisting of hardwood, small pieces of wood, wire rope, or other highly elastic material should not be permitted. Cushion materials containing asbestos are not acceptable because of health hazards. All of these hammer cushion materials are prohibited by AASHTO LRFD Bridge Construction Specifications.

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 proprietary hammer cushion materials last up to 200 hours, it is often sufficient on smaller projects to inspect the cushion material only once before the start of pile driving operation. 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 sandwiches of aluminum and another material such as micarta. The aluminum is used to transfer the heat generated during impact out of the cushion, thus prolonging its useful life. Some common proprietary hammer cushion materials are shown in Figure 15-22 and include Conbest and aluminum (top left), Blue Nylon (right), and aluminum and Micarta (lower left).

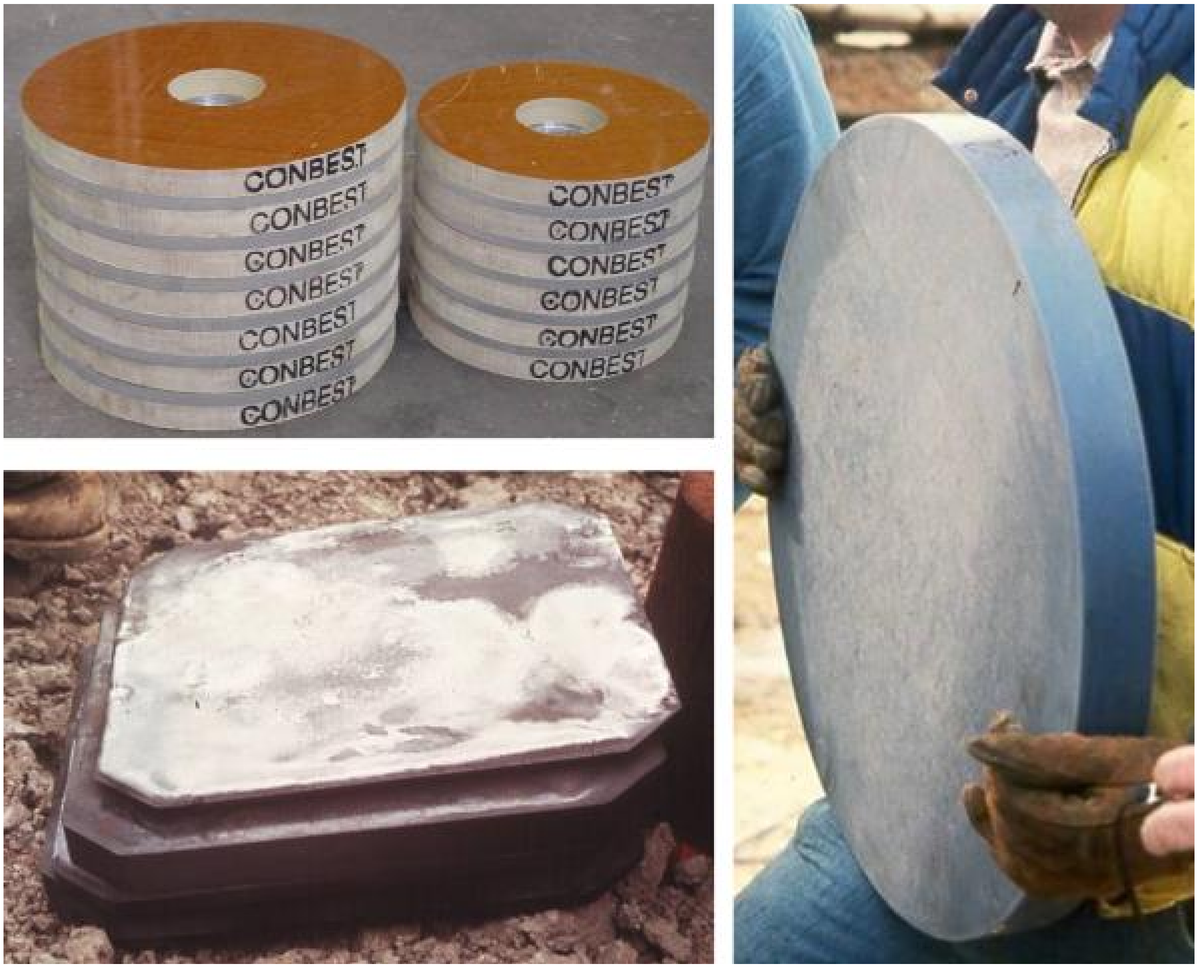


Figure 15-22 Typical hammer cushion materials.

15.7 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 4 inches. 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 plywood pile cushions is presented in Figure 15-23. An unused cushion is shown on the right, while a used and compressed cushion is shown on the left.

Figure 15-23 Plywood pile cushions: (left) used and (right) new.

15.8 HAMMERS

Pile hammers can be categorized in three main types: impact hammers, vibratory ahammers, and resonant hammers. Impact hammer are the primary hammer type used to install foundation piles. There are numerous types of impact hammers having variations in the types of power source, configurations, and rated energies. Figure 15-24 shows a classification of hammers based on motivation and configuration factors. Table 15-1 presents characteristics and uses of several types of hammers. A detailed discussion of various types of hammers follows later in this chapter. Discussion on the key inspection issues associated with each hammer type is provided in Chapter 18. Additional hammer guidance may be found in The Performance of Pile Driving Systems by Rausche et al. (1986), as well as in the Deep Foundation Institute Pile Inspector's Guide to Hammers (1995).

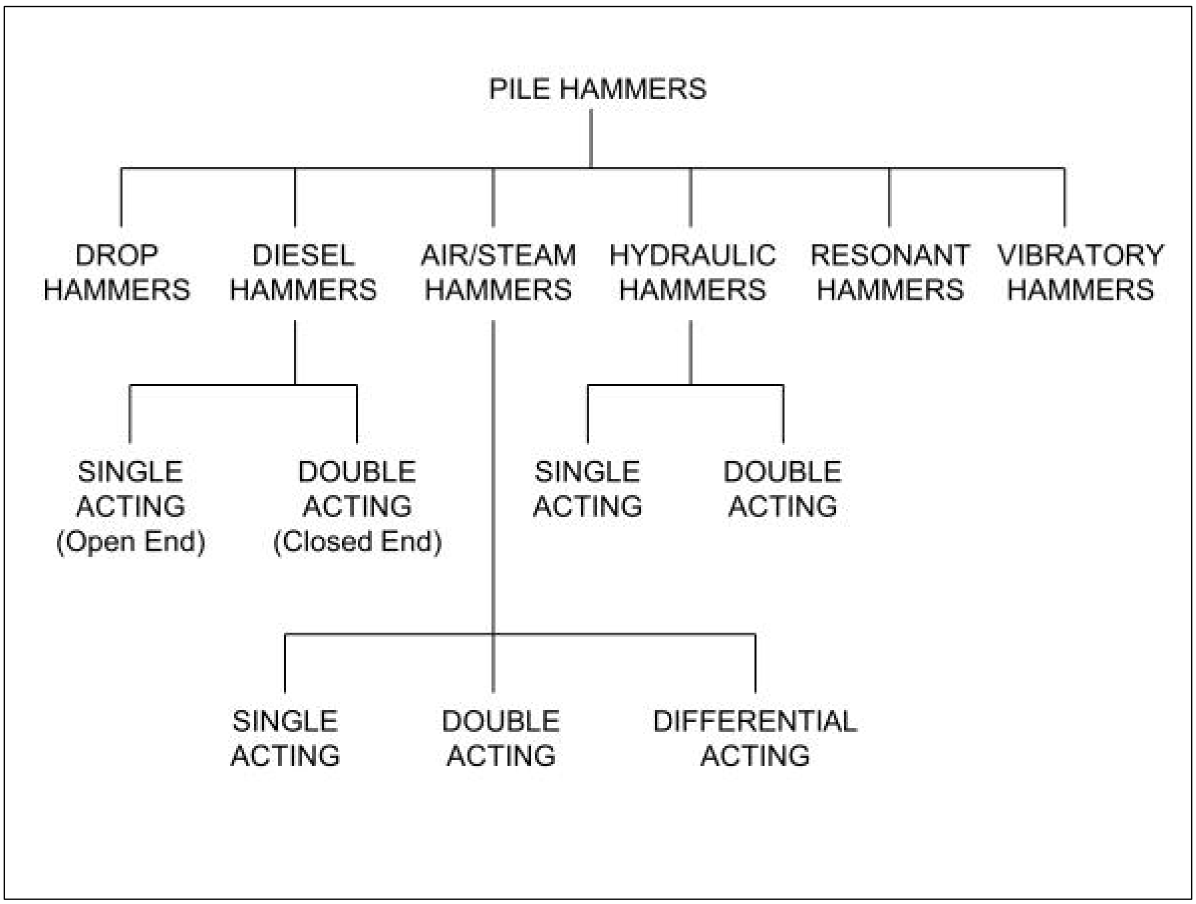


Figure 15-24 Pile hammer classification.

Table 15-1 Typical Pile Hammer Characteristics and Uses

Hammer Type	Single Acting Air / Steam	Double Acting Air / Steam	Differential Acting Air / Steam	Single Acting Diesel (open end)	Double Acting Diesel (closed end)
Rated energy (ft-kips)	7 to 1800	1 to 21	15 to 50	9 to 1620	5 to 73
Impact velocity (ft/sec)	8 to 16.5	15 to 20	13 to 15	10 to 16.5	8 to 16.5
Blow Rate (blows/minute)	35 to 60	95 to 300	98 to 300	40 to 60	80 to 105
Energy (per blow)	Ram weight x stroke.	(Ram weight + effective piston head area x effective fluid pressure) x stroke.	(Ram weight + effective piston head area x effective fluid pressure) x stroke.	Ram weight x stroke.	(Ram weight + bounce chamber pressure) x stroke.
Lifting power	Air or steam.	Air or steam.	Air or steam.	Provided by the combustion of injected diesel fluid.	Provided by the combustion of injected diesel fluid.
Maintenance	More complex than for drop hammer.	More complex than for single acting.	More complex than for single acting.	More complex than most air impact hammers.	More complex than most air impact hammers.
Hammer suitability for types of piles	Versatile for any pile, particularly large concrete and steel pipe.	Timber, steel H and pipe piles.	Timber, steel H and pipe piles.	All types of piles.	All types of piles.
Major advantages	Relatively simple and moderate cost.	More productive than single acting. Some double acting fully enclosed and can be used underwater.	More productive than single acting. Differential air 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.	Carry their own fuel from which power is internally generated. Stroke is a function of pile resistance.
Major disadvantages	Need air compressor or steam plant. Heavy compared with most diesel hammers.	Costs more than single acting. Need air compressor or steam plant.	Costs more than single acting. Need air compressor or steam plant. Heavy compared to diesel hammer.	Pollution from diesel exhaust. Higher cost hammer. Low blows per minute at higher strokes for single acting.	Pollution from diesel exhaust. Higher cost hammer. Hammer lift off in hard driving conditions.
Remarks	Commonly available hammer type.	Ram accelerates downward under pressure.	Ram accelerates downward under pressure.	Most commonly used hammer type. Variable stroke.	Limited availability and use in practice.

Table 15-1 Typical Pile Hammer Characteristics and Uses (Continued)

Hammer Type	Drop	Single Acting Hydraulic	Double Acting Hydraulic	Vibratory	Resonant
Rated energy (ft-kips)	7 to 60	25 to 2162	25 to 2581	----	----
Impact velocity (ft/sec)	23 to 33	5 to 18	5 to 23	----	----
Blow Rate (blows/minute)	4 to 8	30 to 50	40 to 90	750 to 2,400 vibrations/minute	up to 10,800 vibrations/minute
Energy (per blow)	Ram weight x height of fall.	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.	Hydraulic	Hydraulic	Hydraulic or electric power.	Hydraulic
Maintenance	Simple	More complex than other impact hammers.	More complex than other impact hammers.	Highest maintenance cost.	More complex than other hammer types
Hammer suitability for types of piles	All types except concrete piles	All types of piles.	All types of piles.	End bearing steel H and pipe piles. Very effective in granular soils.	Steel H piles and pipe piles.
Major advantages	Lowest initial cost equipment.	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. Fast operating pile installation tool.	Fast pile installation. Very low ground vibrations. Reduced noise.
Major disadvantages	Very high dynamic forces and danger of pile damage. Lowest pile productivity.	High initial cost. Energy readout device recommended to monitor performance.	High initial cost. Fully enclosed, need energy readout device to monitor performance.	High investment and maintenance. Not recommend for friction pile installations.	Limited availability at present.
Remarks	Generally obsolete.	Commonly available hammer type.	Commonly available hammer type.	Variable moment hammers can help control construction vibrations.	Newer hammer type and may require additional field inspection and/or testing.

Note: Vibratory and resonant hammer rated based on power, not energy.

15.8.1 Hammer Energy Concepts

Before the advent of computers and the availability of the wave equation to evaluate pile driving, driving criteria for a given nominal resistance 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 nominal geotechnical 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 generally been replaced by more reliable methods of resistance determination. 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, w, times stroke, h). Although single acting (open end) diesel hammers could also be rated this way, some manufacturers 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.

Ideally, the impact velocity, V_i, could be directly computed using basic laws of physics from the equivalent maximum stroke as shown in Equation 15-1.

$$V_i = \sqrt{2\ g\ h} \qquad \text{Eq. 15-1}$$

Where:

- V_i = impact velocity (ft/s).
- g = acceleration due to gravity (32.17 ft/s^2).
- h = ram stroke (feet).

The kinetic energy could be computed from Equation 15-2.

$$KE = \frac{1}{2}\ m\ {V_i}^2 \qquad \text{Eq. 15-2}$$

Where:

KE = kinetic energy (ft-lbs).
V_i = impact velocity (ft/s).
m = ram mass (slugs/kips).

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, pre-ignition, etc.) which result in the kinetic energy being less than the potential energy. It is the inspector's task to identify these losses when possible, and the contractor's task 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, while the pile head also has inelastic collision losses. Energy is transferred to the pile with time and therefore 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. This is referred to as the transferred energy or ENTHRU.

Pile length, stiffness and resistance 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, which can vary. The stroke of single acting air/steam hammers is somewhat dependent upon the pile resistance and rebound while the stroke of all double acting hammers is even more dependent on pile resistance due to lift-off considerations. In reality, transferred energy increases only when both the force and velocity are positive (compression forces; downward velocity). As resistance increases and/or for shorter pile lengths, 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.

15.9 DROP HAMMERS

The most rudimentary pile hammer still in use today is the drop hammer as shown in Figure 15-25. 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. Operation proceeds 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. Efficiency of the fall is low since the ram is attached by a cable to the hoist and must 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, the fall efficiency and impact energy is 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 resistance is not an issue. Drop hammers are not recommended for foundation pile installations.

Figure 15-25 Typical drop hammer.

15.10 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, the vast majority 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 15-26. A photograph of a typical single acting air/steam hammer is presented in Figure 15-27(a).

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 soil 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 2 inches 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 condition 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 3 to 5 feet. 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 can be used for many pile types, particularly large concrete and steel piles.

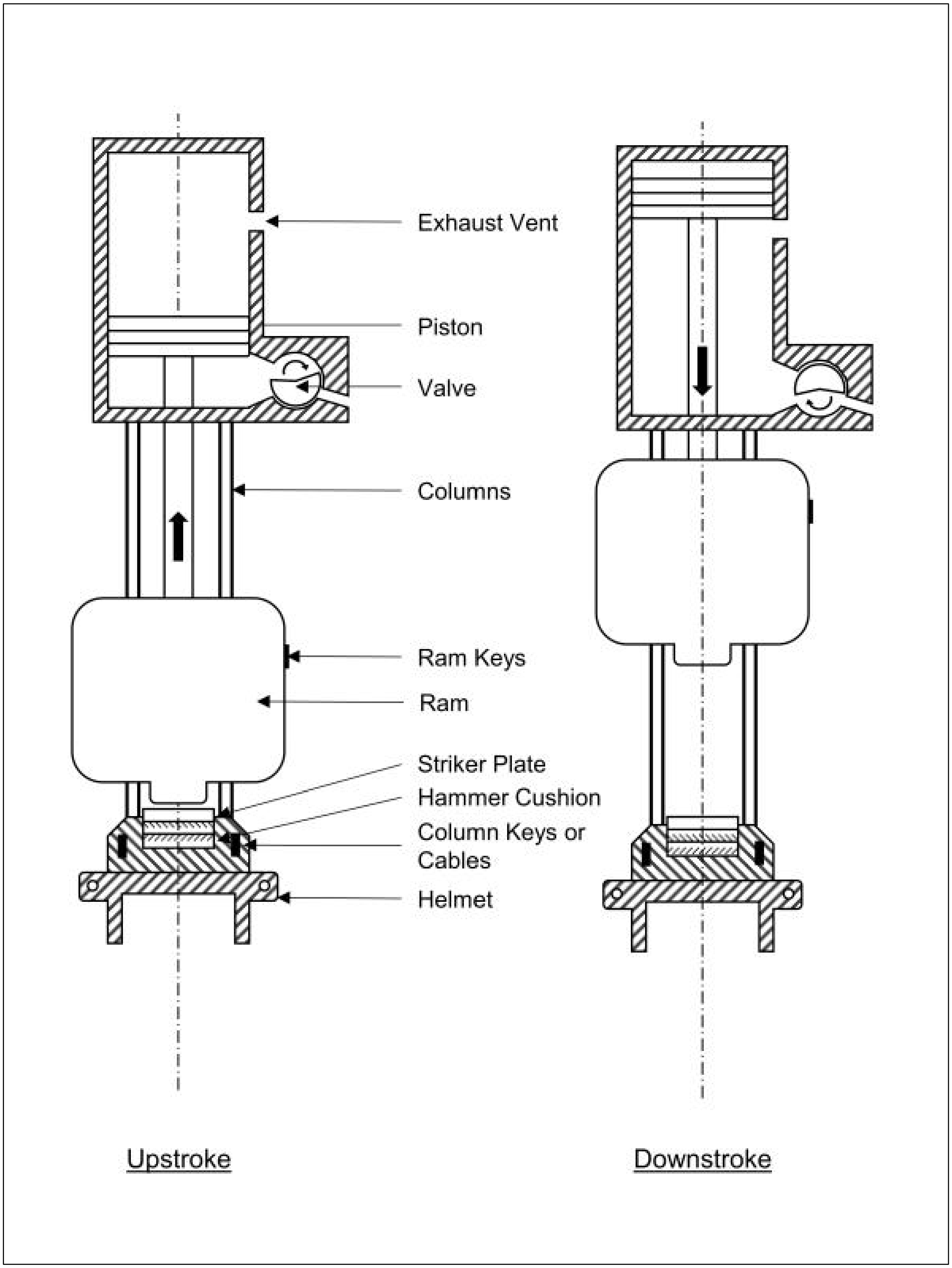


Figure 15-26 Schematic of a single acting air/steam hammer.

Figure 15-27 (a) Single acting air, and (b) differential acting air hammers.

15.11 DOUBLE ACTING AIR/STEAM HAMMERS

The working principle of a double acting air/steam hammer is illustrated in Figure 15-28. For a double acting hammer, the ram is raised by pressurized air or steam 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 air or steam 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, the hammer actually lifting up and away from the pile head. Since the maximum stroke is limited and the same pressure is applied during downstroke, a reduction in the operating pressure due to lift off 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.

Some double acting air/steam hammers are fully enclosed and can be operated underwater. 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. The use of double acting air/steam hammers has diminished and they are infrequently encountered in practice.

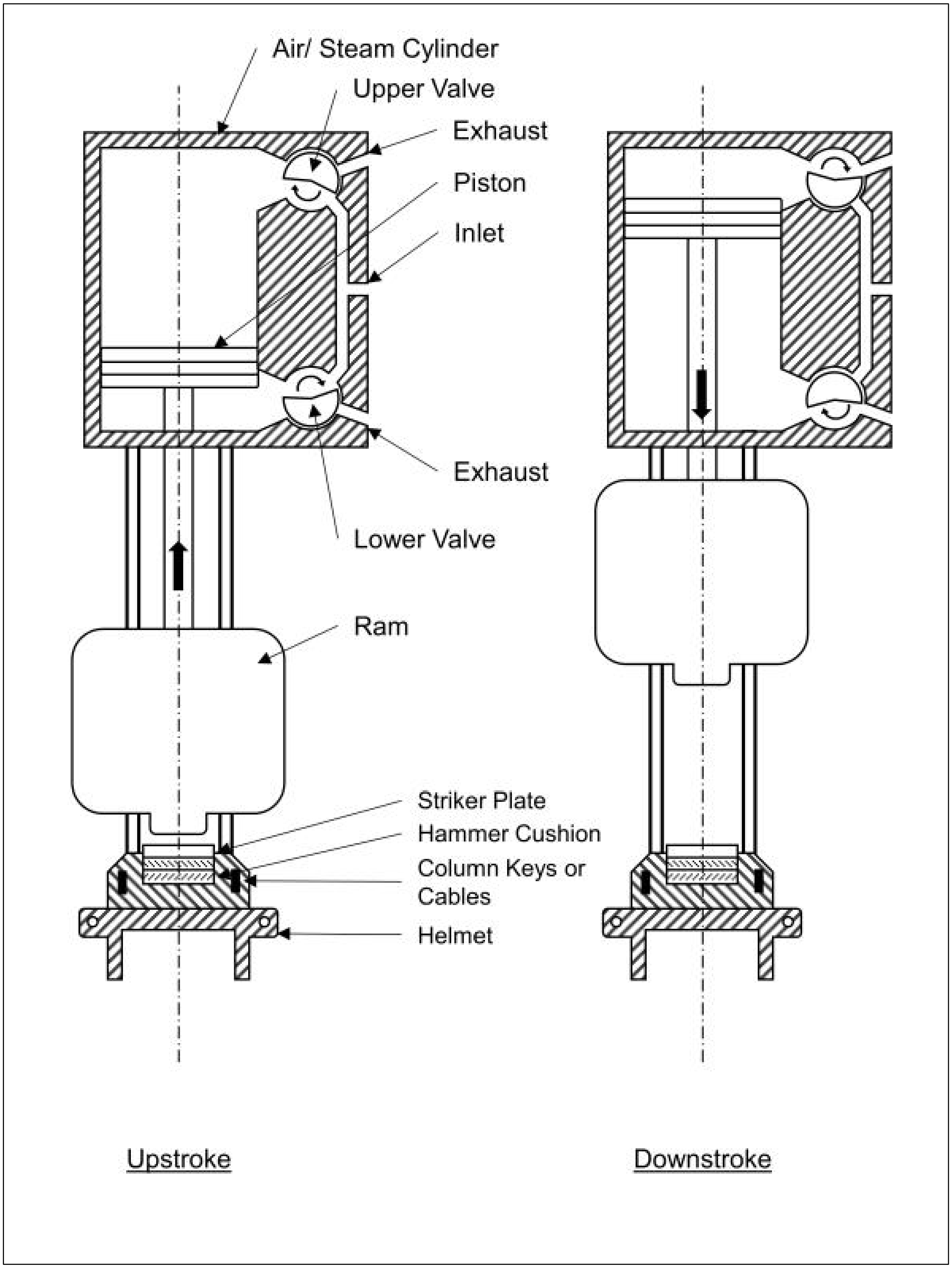


Figure 15-28 Schematic of a double acting air/steam hammer.

15.12 DIFFERENTIAL ACTING AIR/STEAM HAMMERS

The differential acting air/steam hammer is another type of double acting hammer with relatively short stroke and fast blow rates. A photograph of a differential acting air hammer is presented in Figure 15-27(b), while the working principle is illustrated in Figure 15-29. 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 120 and 140 psi <u>at the hammer inlet</u> are required for proper operation. However, most air compressors only produce pressures of about 120 to 130 psi <u>at the compressor</u>. 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. The use of differential acting air/steam hammers has also diminished and they too are infrequently encountered in practice.

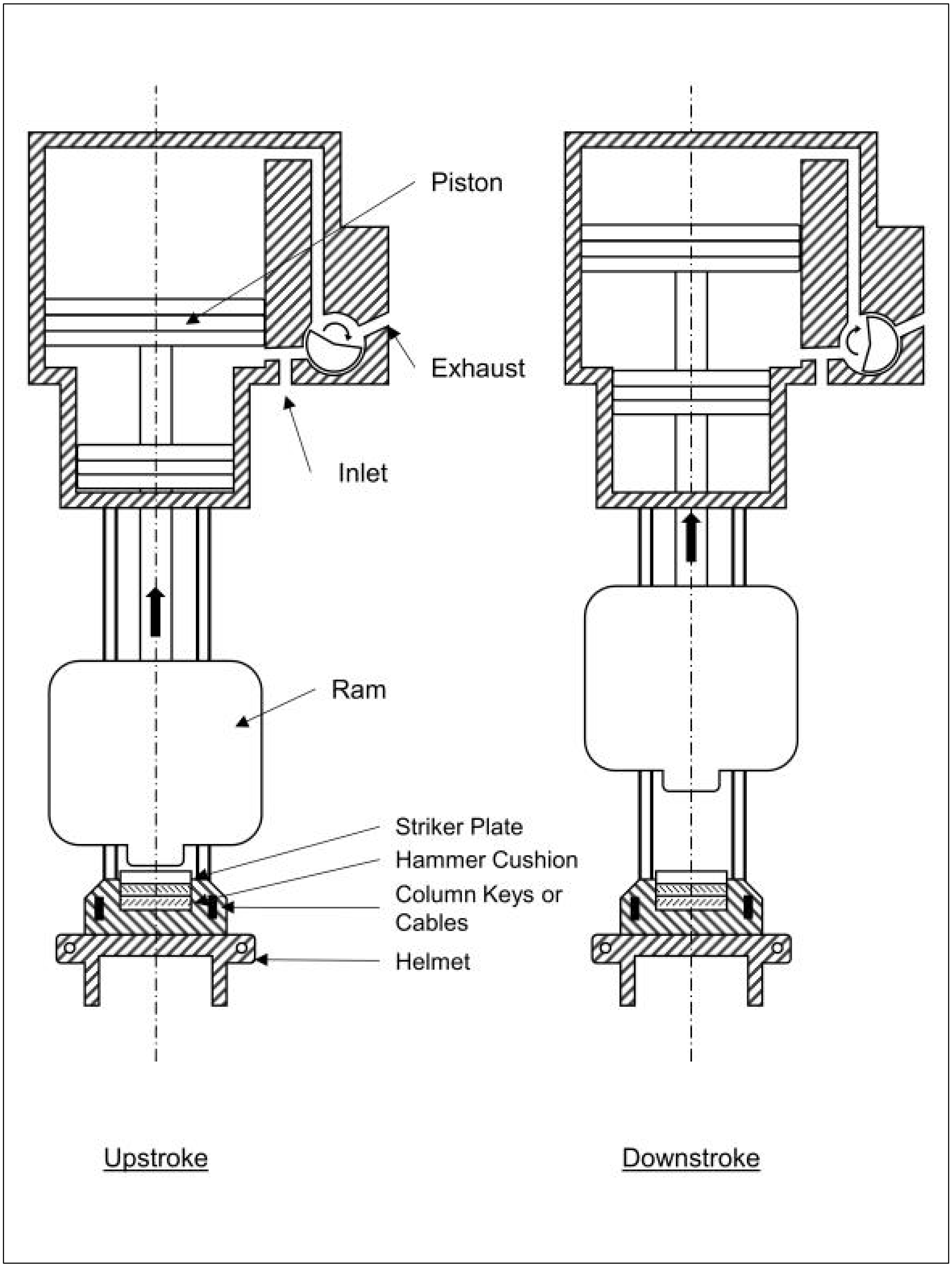


Figure 15-29 Schematic of a differential acting air/steam hammer.

15.13 SINGLE ACTING (OPEN END) DIESEL HAMMERS

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 15-30 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 15-30(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 fully 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 all 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 "pre-ignition", 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.

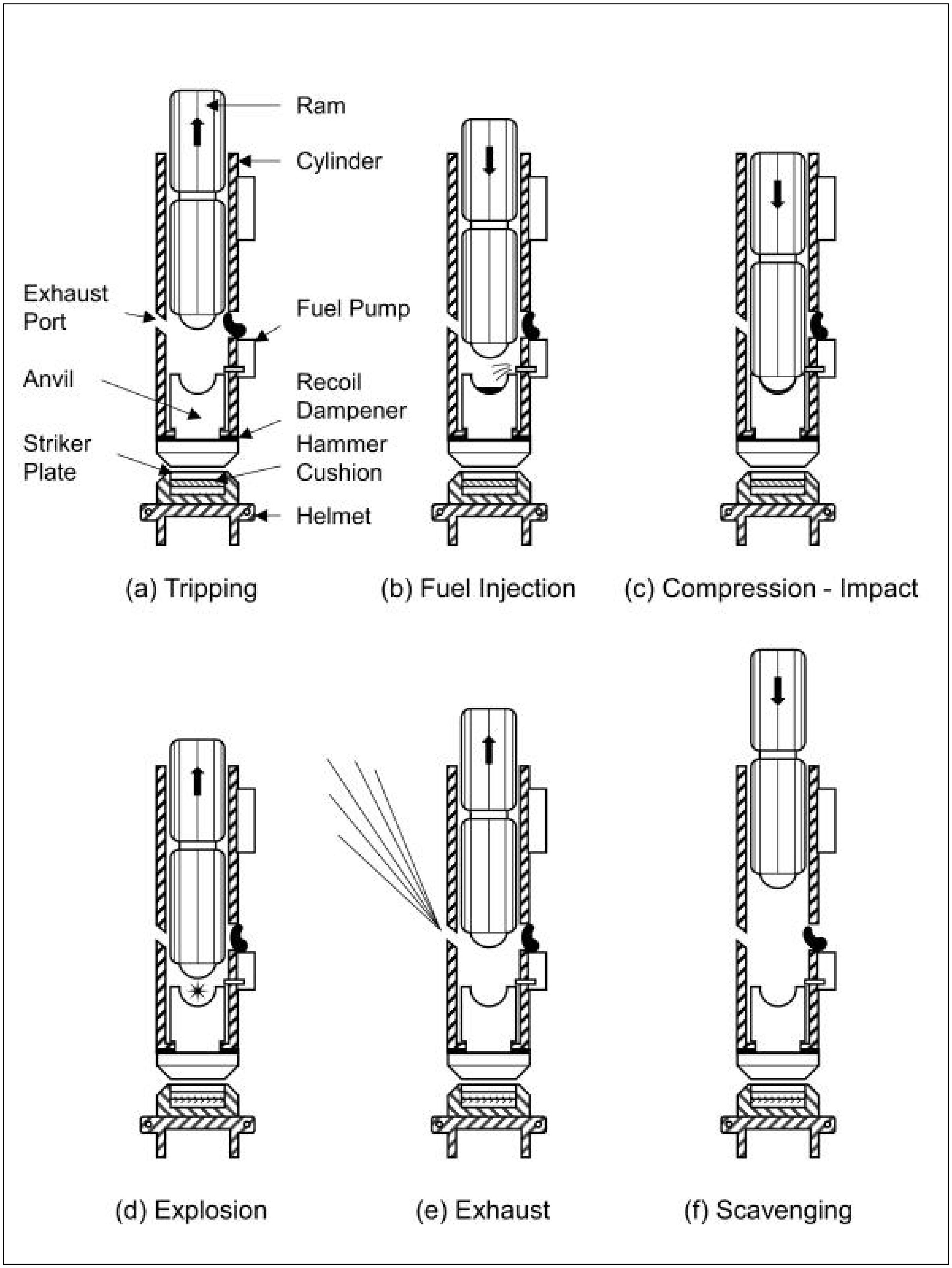


Figure 15-30 Schematic of a single acting diesel hammer.

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:

$$h = 4.01\left(\frac{60}{bpm}\right)^2 - 0.3 \qquad \text{Eq. 15-3}$$

Where:

h = ram stroke (feet).

bpm = blows per minute.

Note: This formula is only applicable for calculating the stroke of single acting diesel hammers and not correct for other hammer types.

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. To address environmental concerns, some diesel hammers can be operated using biodiesel fuel and non-petroleum lubricants. One manufacturer has also developed a smokeless diesel 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 15-31(a).

15.14 DOUBLE ACTING (CLOSED END) DIESEL HAMMERS

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 15-31(b).

Figure 15-31 (a) Single acting diesel and, (b) double acting diesel hammers.

The bounce chamber has ports so that atmospheric pressure exists as long as the ram top is below these ports, as shown in Figure 15-32. 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

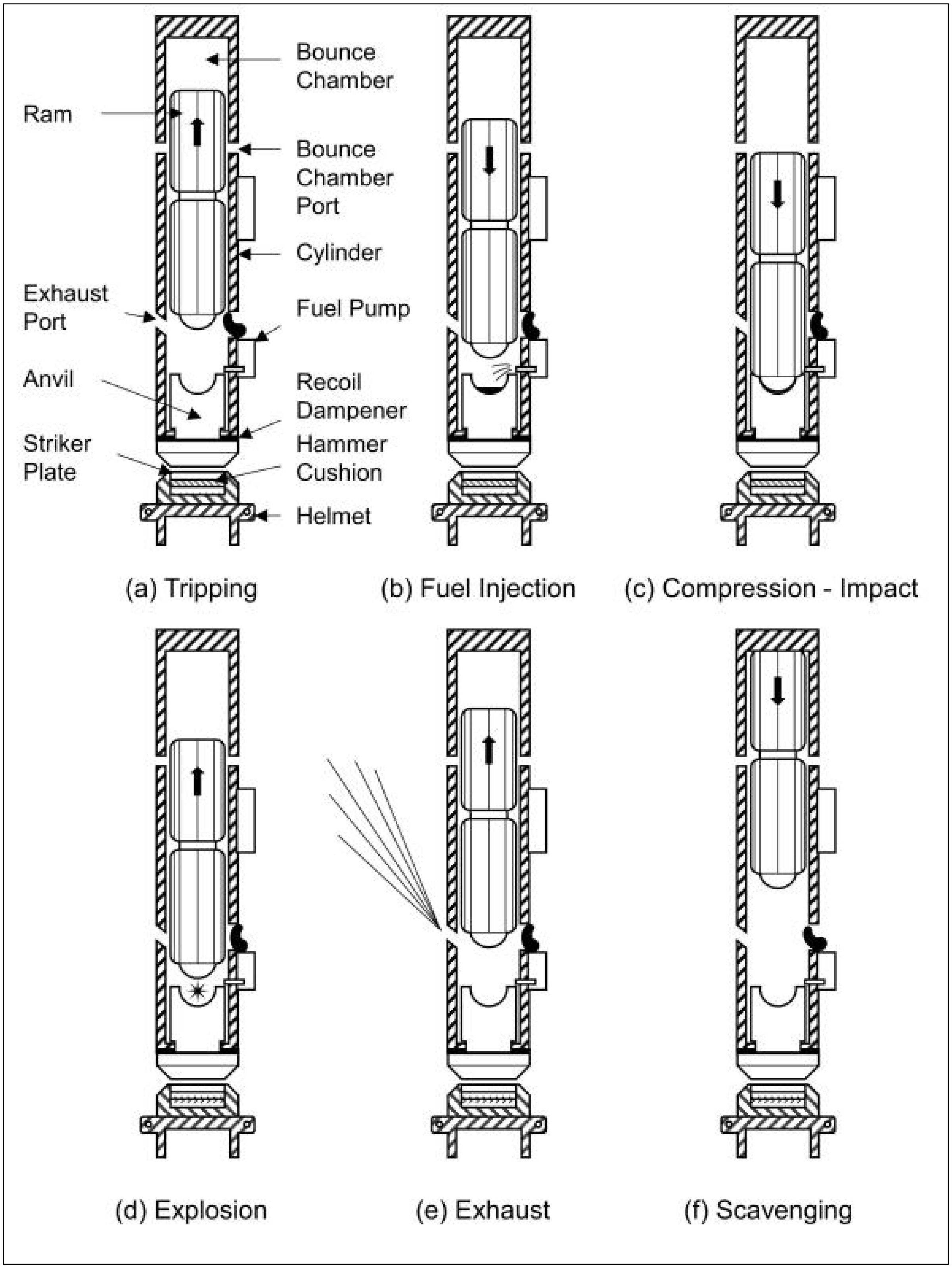


Figure 15-32 Schematic of a double acting diesel hammer.

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 combustion chamber pressure, should 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. The use of double acting diesel hammers is limited and they are infrequently encountered in practice.

15.15 SINGLE ACTING HYDRAULIC HAMMERS

Single acting hydraulic hammers use an external hydraulic power source to lift the ram. 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 size and the ram is lifted by an external power source. Low headroom models exist that can be mounted on an excavator while larger models can be mounted on specialty / universal rigs or in conventional leads. During operation, hydraulic actuators lift the ram which then retracts quickly at predetermined height. This fully releases the ram, which falls under gravity, impacting the striker plate and hammer cushion located in the helmet. The hydraulic cylinder then lifts the ram again and the cycle is repeated. Single acting hammers are classified as such because of dependence upon gravity alone to perform the work. Some hammer models include hydraulic accumulators that can provide a relatively small double acting component. A schematic of a single acting hydraulic hammer is illustrated in Figure 15-33(a), while a photograph of a single acting hydraulic hammer is presented in Figure 15-34(a).

Most single acting hydraulic hammers allow the ram stroke, blow rate, and dwell time (duration ram remains on top of pile following impact) to be continuously varied and controlled using a pendant attached to the hydraulic power pack. Very short strokes may be used during easy driving to prevent pile run or to minimize tension stresses in concrete piles. Higher strokes are available for hard driving conditions.

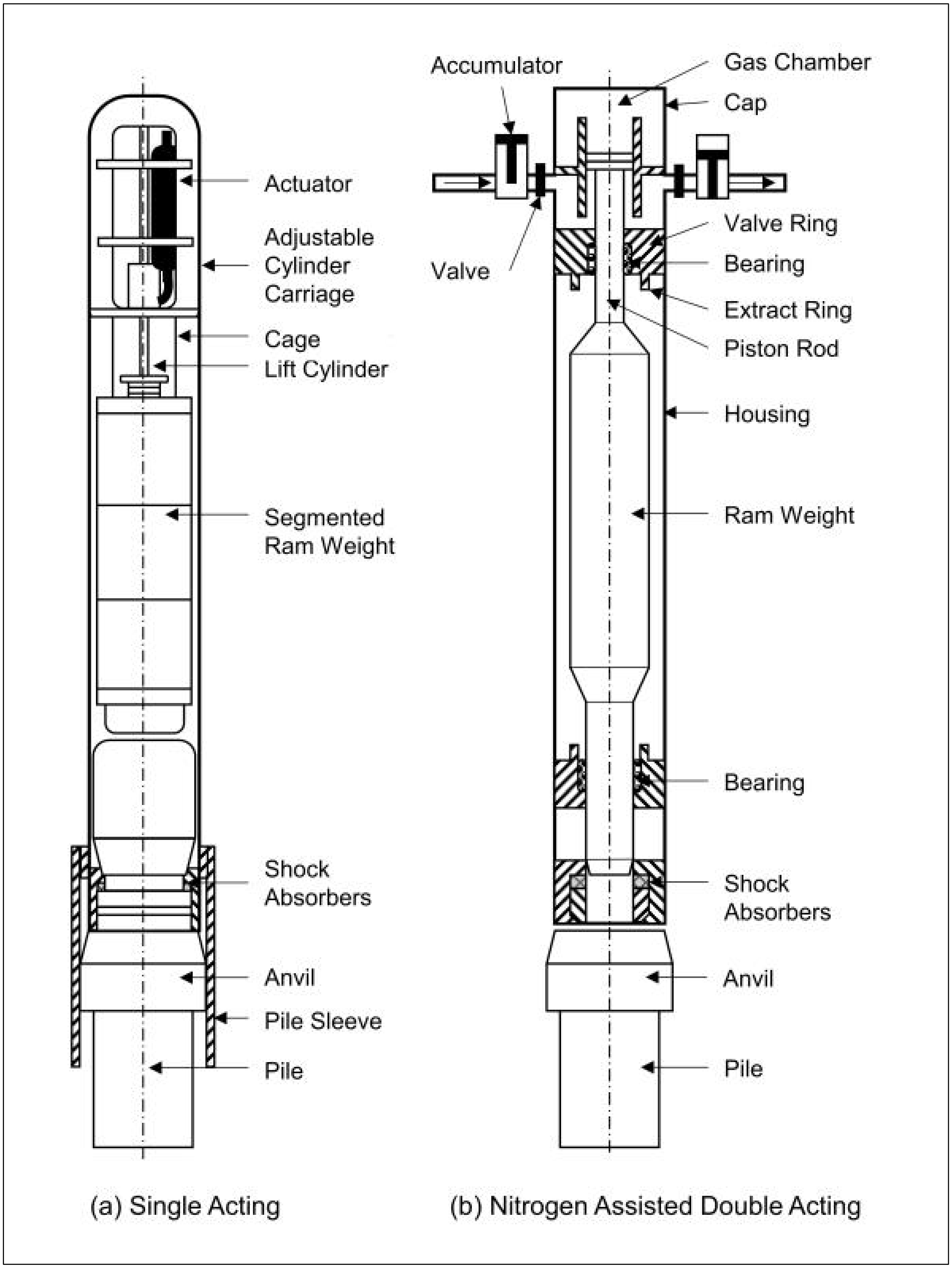


Figure 15-33 Schematic of hydraulic hammers:
(a) single acting and (b) double acting.

Figure 15-34 (a) Single acting hydraulic, and (b) double acting hydraulic hammers.

On many single acting hydraulic hammers, such as the one illustrated in Figure 15-33a, the stroke can be easily be visually estimated. Most newer 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. Hydraulic hammers require a dedicated hydraulic power pack, and can be more complex to operate and maintain compared to other hammers.

15.16 DOUBLE ACTING HYDRAULIC HAMMERS

Double acting hydraulic hammers cover the same range of manufacturer's rated energy as the single acting hydraulic hammers but can also be significantly larger. Figure 15-33(b) presents a schematic of a double acting hydraulic hammer. Double acing hammer consist of a ram attached to a piston rod which are entirely enclosed within the hammer housing and an external hydraulic source supplies the power to the hammer. Oil flows through a supply valve into the hammer and out through a return valve to the power pack. During hammer operation, the return valve closes, which causes the piston to rise. When the piston reaches a predetermined height, the supply valve closes and the return valve opens. Pressure is relieved and the ram begins its downward stroke accelerated by hydraulic pressure or compressed nitrogen gas. The ram then strikes the anvil, which in turn contacts the pile. Steel on steel impact occurs between these two parts which results in a relatively high energy transfer.

Similar to single acting hydraulic hammers, the ram stroke can be controlled to adapt to the driving conditions. However because of the enclosed housing, the stroke cannot be visually estimated. Built-in monitoring systems are therefore essential on double acting hammers to document hammer performance. A photograph of a double acting hydraulic hammer is included in Figure 15-34(b). A significant advantage of the fully enclosed double acting hydraulic hammers is that they can operate underwater. This allows piles to be driven without using a follower or extra length pile. Most double acting hydraulic hammers do not have conventional hammer cushions and thus generate steel to steel impacts with high efficiency. Some models can also be used to drive piles horizontally.

15.17 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 eccentric weights cancel). A schematic of a vibratory hammer is presented in Figure 15-35 and a photograph is included in Figure 15-36. Several types of vibratory hammers exist, variable moment vibratory hammers, high frequency vibratory hammers, standard vibratory hammers and low frequency vibratory hammers. Variable moment and high frequency hammers typically operate at up to 2300 to 2400 vibrations per minute or 40 Hz. Standard vibratory hammers operate at up to a maximum of 1600 vibrations per minute or 26 Hz and low frequency vibratory hammers operate up to about 1200 vibrations per minute or 20 Hz.. The lower the operating rate (vibrations per minute) of the vibratory hammer the greater the effect on the soil and structure. Hence, hammer start up and shut down often produce the most construction vibration concerns. Variable moment vibratory hammers are attractive in these situations as the hammer can be brought to full operating speed with the eccentric moments in neutral thus avoiding critical frequencies for potential vibration damage.

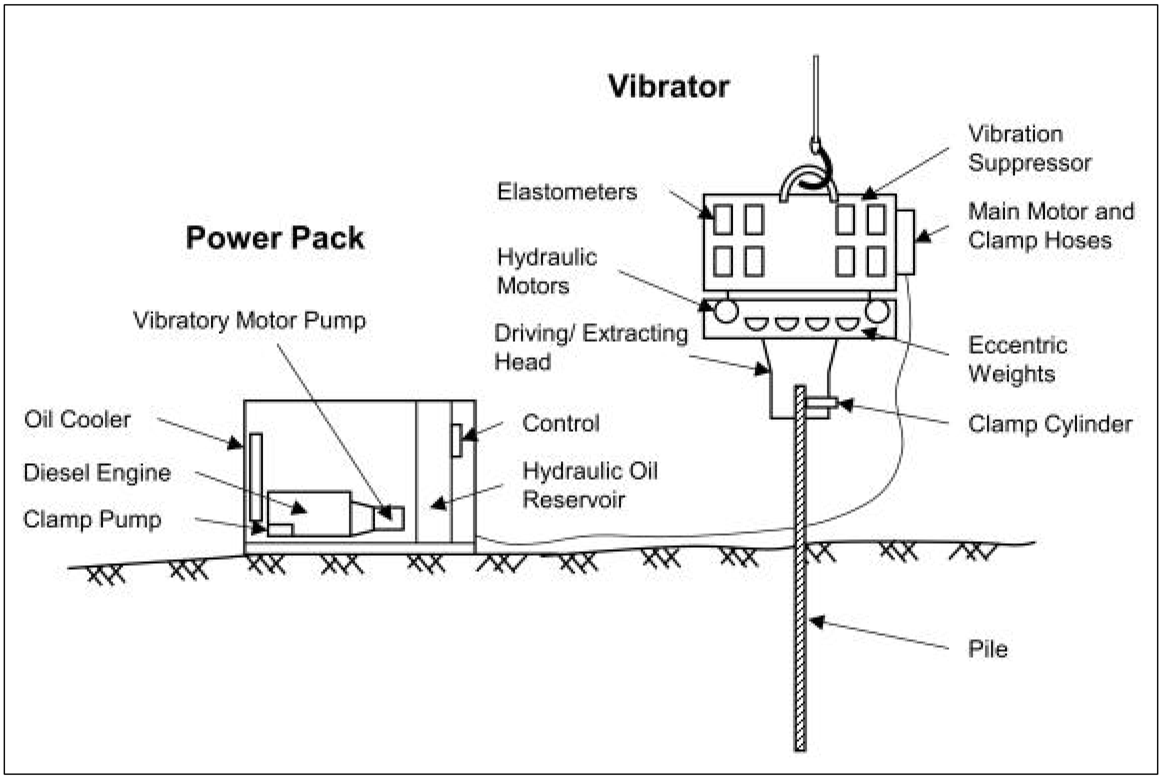


Figure 15-35 Schematic of a vibratory hammer.

Figure 15-36 Vibratory hammer installing an H-pile (courtesy ICE).

Vibratory hammers are rigidly connected by hydraulic clamps to the pile head and may be used for either pile installation or extraction. Vibratory hammers typically do not require leads, although templates are often required for sheet pile cells. These 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 nominal resistance during vibratory hammer installation has not yet been developed. If a vibratory hammer is used for pile installation, confirmation of the nominal resistance by other means is still necessary.

15.18 RESONANT HAMMERS

Resonant pile hammers use a hydraulic piston-cylinder design to generate a high magnitude, high frequency, oscillating force. The amplitude and magnitude of the force is controlled through a valve that rapidly switches hydraulic oil to alternating sides of the piston that oscillates at up to 180 Hz (10,800 vibrations per minute). Piles are advanced into the soil using the high frequency vibration and resonance. A proprietary algorithm automatically adjusts and optimizes the frequency to maintain resonance. Janes (2009) summarized projects where resonant hammers have been used to install steel H-piles and pipe piles.

Advantages of the resonant hammer include fast pile installation, very low ground vibrations, and reduced noise levels compared to conventional equipment. Typical cranes and hydraulic power packs can be used on resonant pile hammer projects. Similar to vibratory installed piles, a disadvantage of resonant hammer use is nominal resistance verification. A reliable technique for determining the nominal resistance is not available. Therefore, a conventional impact hammer and associated standard nominal resistance verification methods must be employed after resonant hammer pile installation is completed. A photograph of a resonant hammer is shown in Figure 15-37.

Figure 15-37 Resonant hammer installing an H-pile
(courtesy of Resonance Technology).

15.19 IMPACT HAMMER SIZE SELECTION

It is important that the contractor and the engineer choose the proper hammer for efficient use on a given project. An impact hammer which is too small may not be able to drive the pile to the required resistance, or may require an excessive number of blows. On the other hand, an impact hammer which is too large may damage the pile. The use of empirical dynamic pile formulas to select hammer energy and size is not recommended as this approach incorrectly assumes these formulas result in the desired nominal resistance. Results from these formulas become progressively worse as both the complexity of the hammers and the required nominal pile resistance increase.

A wave equation analysis, which considers the hammer cushion pile soil system, is the recommended method to determine the optimum hammer size. In general, a hammer having a ram weight of 1 to 2% of the required nominal resistance or nominal driving resistance, whichever is greater, often yields a good first estimate of the necessary hammer size. Table 15-2 also provides approximate minimum hammer energy sizes for preliminary equipment evaluation based on ranges of nominal resistances. These are generalizations 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 14.

Table 15-2 Preliminary Hammer Energy Requirements

Nominal Resistance (kips)	Minimum Manufacturer's Rated Energy (ft-lbs)
180 and under	12,000
181 to 300	21,000
301 to 415	28,800
416 to 540	37,600
540 to 600	42,000

15.20 HAMMER KINETIC ENERGY MONITORING

Several pile driving hammers are available from their manufacturers with kinetic energy readout devices. These devices typical monitor hard wired proximity switches built into the hammer body. The impact velocity and hammer kinetic energy are calculated based on the time it takes the ram to travel the distance between the proximity switches. These devices also typically provide the hammer blow rate, and for open end diesel hammers, the hammer stroke. Examples of hammer manufacturer provided devices are presented in Figure 15-38 for an IHC hydraulic hammer and Figure 15-39 and Figure 15-40 for a Berminghammer diesel hammer.

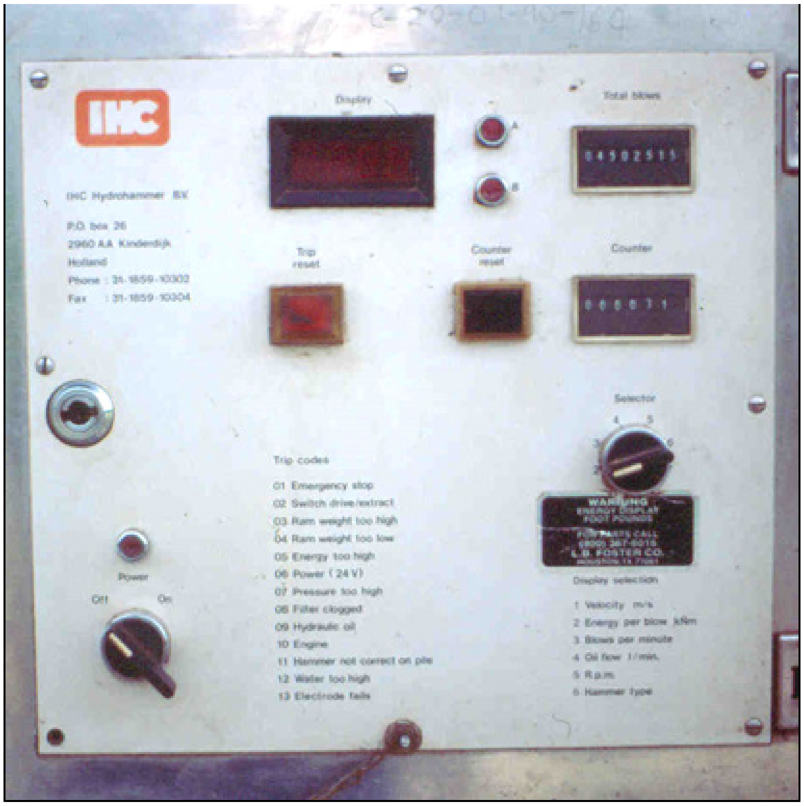


Figure 15-38 IHC hydraulic hammer kinetic energy readout panel.

Any existing hammer can also be retrofitted for these measurements by attaching proximity switches to the hammer body. Attachment procedures vary depending upon the hammer model. For a diesel hammer, proximity switches are set into two 1.2 inch diameter smooth bore drill holes in the cylinder wall above the combustion chamber. For air/steam or single acting hydraulic hammers, the proximity switches are attached to the hammer body. The proximity switches are connected to a transmitter mounted on the hammer that sends the impact velocity and kinetic energy to a wireless hand held unit. This hand held unit, called an E-Saximeter, can also be used to keep a pile driving log if the inspector presses the enter key with each passing pile penetration increment. For single acting diesel hammers, the hammer stroke can also be calculated and displayed. A photograph of a diesel hammer retrofitted for kinetic energy measurements is presented in Figure 15-41 and the readout device is in Figure 15-42.

Hammers equipped with kinetic energy readout devices provide improved quality control and are particularly attractive on large projects or with piles that require a large nominal resistance. These devices can detect changes in hammer performance over time that may necessitate adjustment to the pile installation criterion.

Figure 15-39 Proximity switch attachment for Berminghammer diesel hammer (courtesy Berminghammer).

Figure 15-40 Proximity switches, readout device and driving log trigger switch for Berminghammer diesel hammer (courtesy Berminghammer).

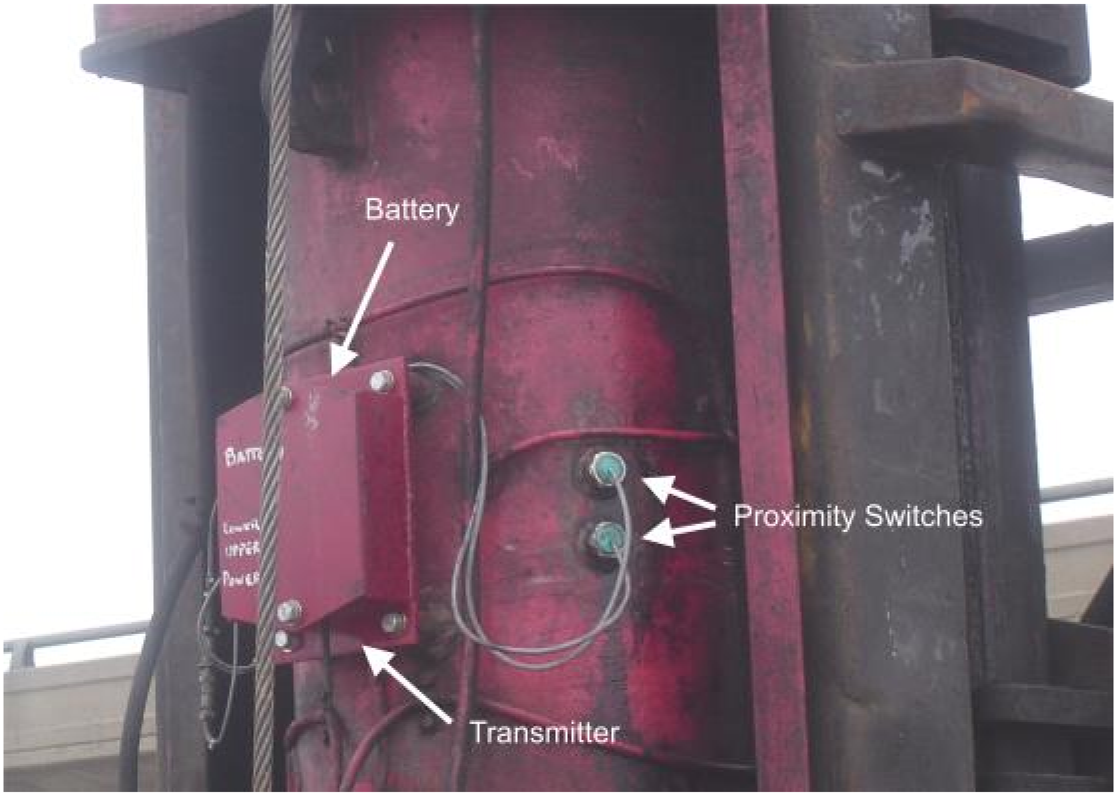


Figure 15-41 Proximity switches and transmitter on retrofitted diesel hammer.

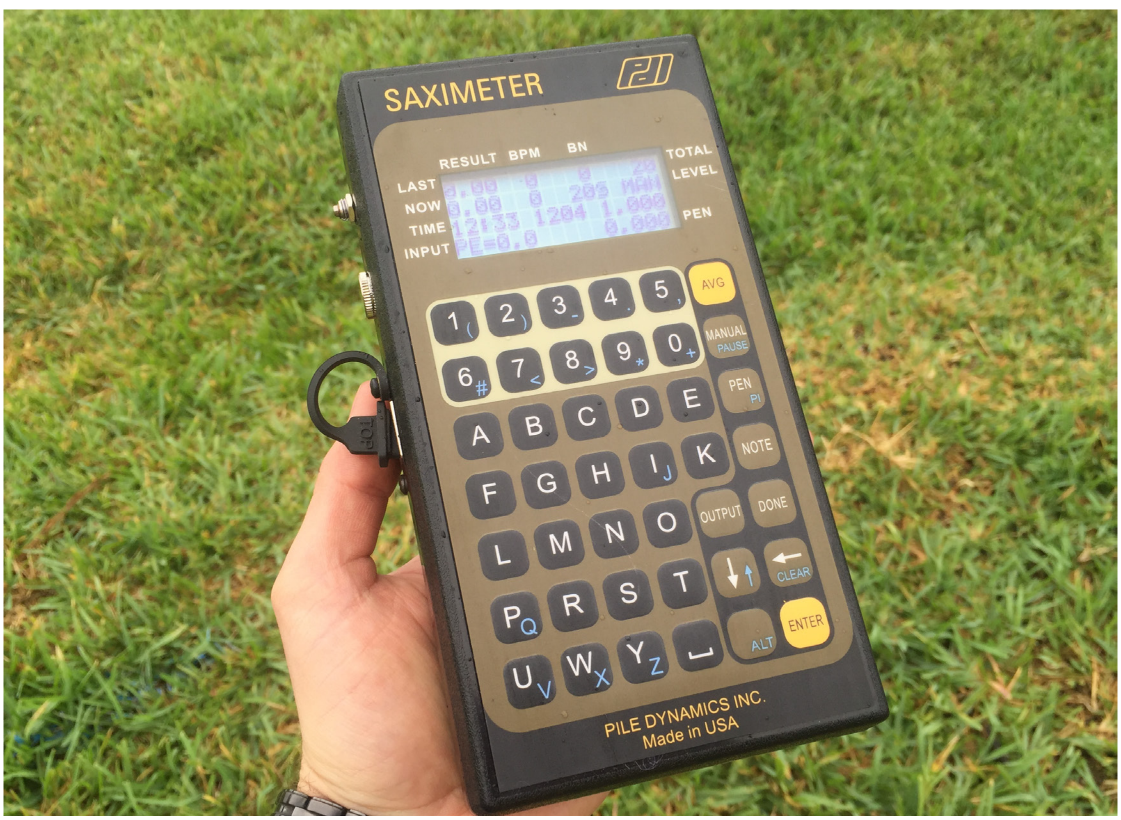


Figure 15-42 E-Saximeter wireless kinetic energy readout device.

15.21 NOISE SUPPRESION EQUIPMENT

Depending upon the hammer and pile type used, noise from impact pile driving operations can range from around 80 to 130 dBa. Local ordinances or specification may place limits on noise levels that may influence equipment selection or may dictate that pile driving noise shrouds be used in order to meet the specified noise limits. Manufacturers of a few diesel and hydraulic hammers can provide optional noise suppression devices that may reduce the pile driving generated noise by about 10 dBa. An example of a noise shroud produced by a hammer manufacturer is presented in Figure 15-43.

Greater reductions in pile driving noise have been obtained by combining noise abatement techniques on a project. A 20 to 25 dBa reduction was obtained through the combined use of shock absorbing cushion material, a hammer exhaust noise shroud, application of damping compound to the steel piles, and use of a noise shroud around the hammer-pile impact area. Hammer and pile type selection can also influence the pile driving generated noise and should be considered in the design stage of projects in noise sensitive areas.

Figure 15-43 Noise shroud for IHC hydraulic hammer.

15.22 UNDERWATER NOISE SUPPRESSION EQUIPMENT

Bubble curtains are sometimes required when driving piles through water to reduce underwater sound waves, shock waves, and overpressures that impact marine mammals and fish. In general, overpressure levels greater than 4.4 psi have been found to be detrimental. However, the detrimental overpressure level will vary depending upon the species of fish, their size, and their maturity level.

Bubble curtain devices use air bubbles to attenuate the pile driving induced pressure wave. Bubble curtains can be categorized as bubble rings or bubble walls. A bubble ring is typically used around a single pile and typically consists of a high volume air compressor a primary feed line, a primary distribution manifold, medium volume secondary feed lines, and secondary distribution manifolds. Bubble walls combine the features of bubble rings with a sound damping curtain that encapsulates the air bubbles. Bubble walls are typically placed around a complete substructure location rather than an individual pile.

For a bubble curtain to be effective, the bubble curtain must completely surround the pile driving activity. This can sometimes be difficult to accomplish with a bubble ring in areas with tides and currents, or when the foundation design includes batter piles. Bubble rings are sometimes used in conjunction with containment devices such as a large diameter pile sleeve, a turbidity curtain, or a cofferdam in these situations. A bubble wall system may be more attractive in areas where a bubble ring system requires containment devices. A photograph of a pile driven inside bubble ring used in conjunction with a containment device is presented in Figure 15-44. Longmuir and Lively (2001) presented a case history where use of a bubble ring reduced overpressures during pile driving from in excess of 22 psi with no mitigation to less than 3 psi.

The WSDOT and FHWA sponsored a full scale test of new underwater noise suppression technology developed by the University of Washington and Marine Construction Technologies, PBC. The research study developed a patented solution for mandrel bottom driving a steel pipe pile, trademarked a Reinwall pile. In the full scale field study where 30 inch pipe piles were driven, Reinwall et al. (2014) determined that the air gap between the mandrel and conventional steel pipe resulted in a 21 to 23 dBa reduction in underwater noise compared to the 3 to 6 dBa reduction achieved with a typical bubble curtain.


Figure 15-44 Bubble ring with containment device (courtesy of WSDOT).

15.23 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. A photograph of a follower for driving steel H-piles underwater is presented in Figure 15-45(a).

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 blow count to obtain the required nominal resistance. These losses can be estimated by an extensive and thorough wave equation analysis, or field evaluated by dynamic measurements. In Figure 15-45(b), the hammer-follower-pile alignment issues are apparent for the H-pile being driven on a batter.
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.

Figure 15-45 (a) Follower and (b) follower in use drving H-piles.

15.24 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 15-46 and Figure 15-47, respectively.

Figure 15-46 Dual jet system mounted on concrete pile (courtesy of Florida DOT).

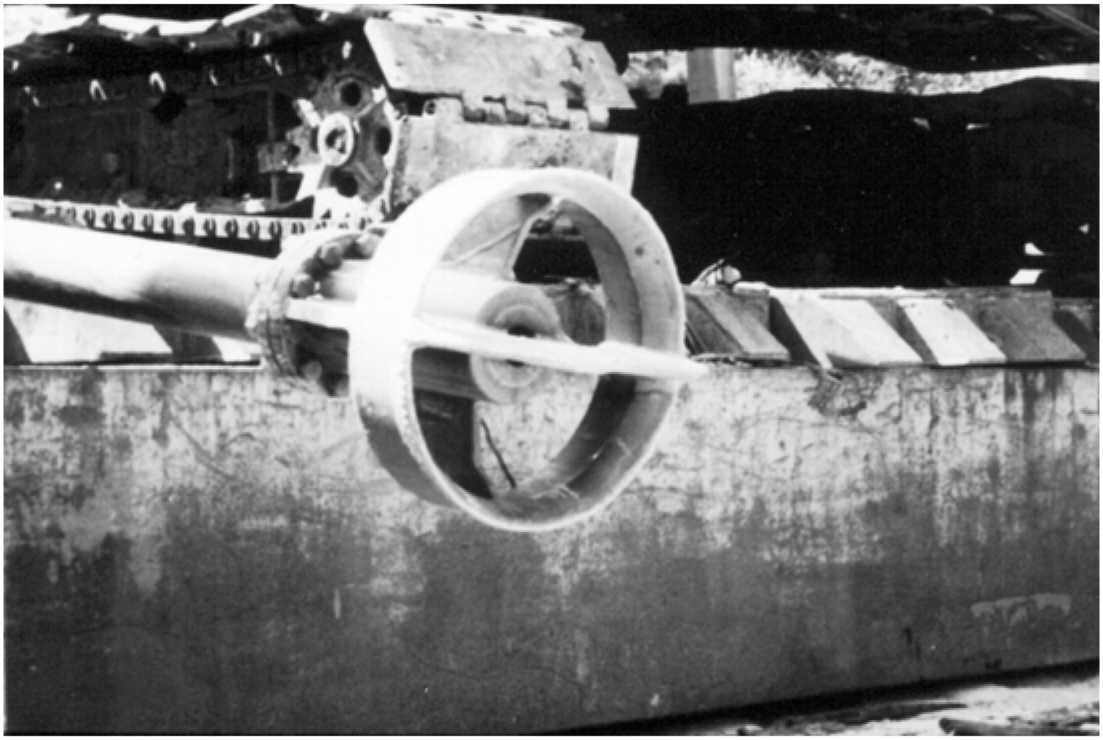


Figure 15-47 Jet/Punch system (courtesy of Florida DOT).

15.25 PREDRILLING

Soil augers or rotary drills may sometimes be used to facilitate driving or limit vibrations. 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 spacing.

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. The hole size and depth should be noted in the contract documents. In most situations, the predrilled hole diameter should be 4 inches less than the diagonal of square or steel H-piles, and 1 inch 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. When predrilled holes are used in embankments, a hole of up to 6 inches larger than the greatest pile cross sectional dimension is sometimes used. 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 15-48.

Figure 15-48 Solid flight auger predrilling system.

15.26 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. Two spudding devices are shown in Figure 15-49. The spud in Figure 15-49(a) consists of a thick walled pipe section with a conical tip formed from steel plates. The spud in Figure 15-49(b) was made by adding a wedge shaped tip and plates to a H-pile section.

15.27 EQUIPMENT SUBMITTALS

The Contractor should provide an equipment submittal of all proposed equipment as well as the proposed procedures for equipment use to the Engineer prior to pile

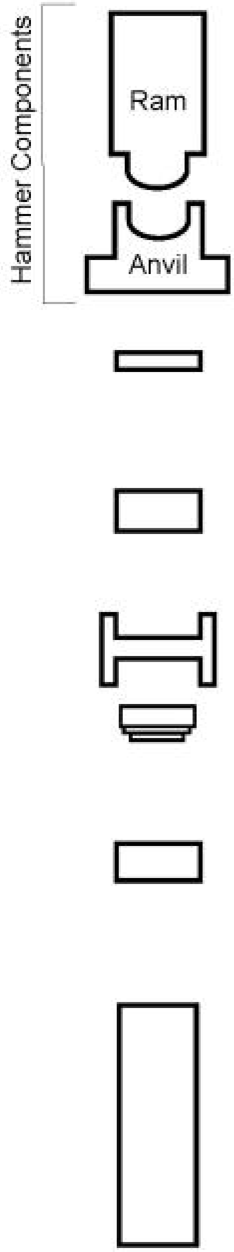
Figure 15-49 (a) Pipe pile and (b) H-pile spuds.

driving operation. In addition, a Pile Installation Plan should be prepared and followed throughout the project. This allows the Engineer to evaluate the proposed equipment, perform a wave equation analysis with appropriately modeled soil conditions, and provide preliminary or final driving criteria. Figure 15-50 presents a drive system submittal form covering hammer components and hammer accessories as well as pile type, pile section, pile splicing and pile toe details. Additional information on any jetting, predrilling, or spudding equipment should be submitted in conjunction with this hammer submittal form. Reference can be made to Section 14.2.1 and 14.2.3 of Chapter 14 respectively for specifications regarding the Pile Installation Plan and equipment used for driving.

By submitting technical information related to the pile driving equipment, the Engineer can evaluate the suitability and expected performance of the proposed system. Without the specific installation system details, the accuracy of any wave equation modeling and resulting driving criteria or drivability assessment is reduced. An in depth discussion of wave equation analyses is provided in Chapter 12 of this manual.

Contract No.: ____________________
Project: ____________________
____
County: ____________________

Structure Name and/or No.: ____________________
____
Pile Driving Contractor or Subcontractor: ____________________
____
(Piles driven by)



Hammer

Manufacturer: ____________ Model No.: ____________
Hammer Type: ____________ Serial No.: ____________
Manufacturers Maximum Rated Energy: ____________ (ft-lbs)
Stroke at Maximum Rated Energy: ____________ (ft)
Range in Operating Energy: ____________ to ____________ (ft-lbs)
Range in Operating Stroke: ____________ to ____________ (ft)
Ram Weight: ____________ (kips)
Modifications: ____________
____
____

Striker Plate

Weight: ____________ (kips) Diameter: ____________ (in)
Thickness: ____________ (in)

Hammer Cushion

Material #1	Material #2 (Composite Cushion)
Name: ____________	Name: ____________
Area: ____________ (in^2)	Area: ____________ (in^2)
Thickness/Plate: ____________ (in)	Thickness/Plate: ____________ (in)
No. of Plates: ____________	No. of Plates: ____________

Total Thickness of Hammer Cushion: ____________ (in)

Helmet

Weight: ____________ (kips)

Insert (If Any)

Weight: ____________ (kips)
Total Weight of Helmet and Insert: ____________ (kips)

Pile Cushion

Material: ____________
Area: ____________ (in^2) Thickness/Sheet: ____________ (in^2)
No. of Sheets: ____________
Total Thickness of Pile Cushion: ____________ (in)
Maximum Thickness Accommodated by Helmet : ____________ (in)

Pile

Pile Type: ____________
Wall Thickness: ____________ (in) Taper: ____________
Cross Sectional Area: ____________ (in^2) Weight/ft: ____________

Ordered Length: ____________ (ft)
Factored Resistance: ____________ (kips)
Nominal Resistance: ____________ (kips)

Description of Splice: ____________
____
Driving Shoe/Closure Plate Description: ____________
____

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

Figure 15-50 Drive system submittal form.

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CHAPTER 16

PILE ACCESSORIES

Pile accessories are frequently used for pile toe protection or to facilitate pile splicing. Accessories are available for driven piles that can make pile installation easier and faster. They can also reduce the possibility of pile damage and help provide a more dependable and permanent support for any structure. Heavier foundation loads, pile installations to sloping rock surfaces or into soils with obstructions, and longer pile lengths, are project situations where the use of pile toe attachments 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. During driving and under all applicable loading conditions, pile toe attachments and splices should develop the required structural resistance.

16.1 TIMBER PILES

Potential problems associated with driving timber piles include splitting and brooming of the pile toe and/or 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.

16.1.1 Pile Toe Attachments

Timber pile toe protection devices include steel boots or points. The trend toward larger pile hammers and higher nominal resistances may result in greater risk of damage for timber piles if obstructions are encountered. Figure 16-1 shows two types of commonly use pile toe attachments. Timber pile boots are designed to cover the entire pile toe without the need for trimming. Timber pile points involve trimming the pile toe to fit the point. Both toe protection devices can be secured quickly in the field. The toe protection device is the device over the pile toe and then nailing the shoe straps to the pile toe and anchoring the attachment. Figure 16-2 shows a pile boot attached to a timber pile.

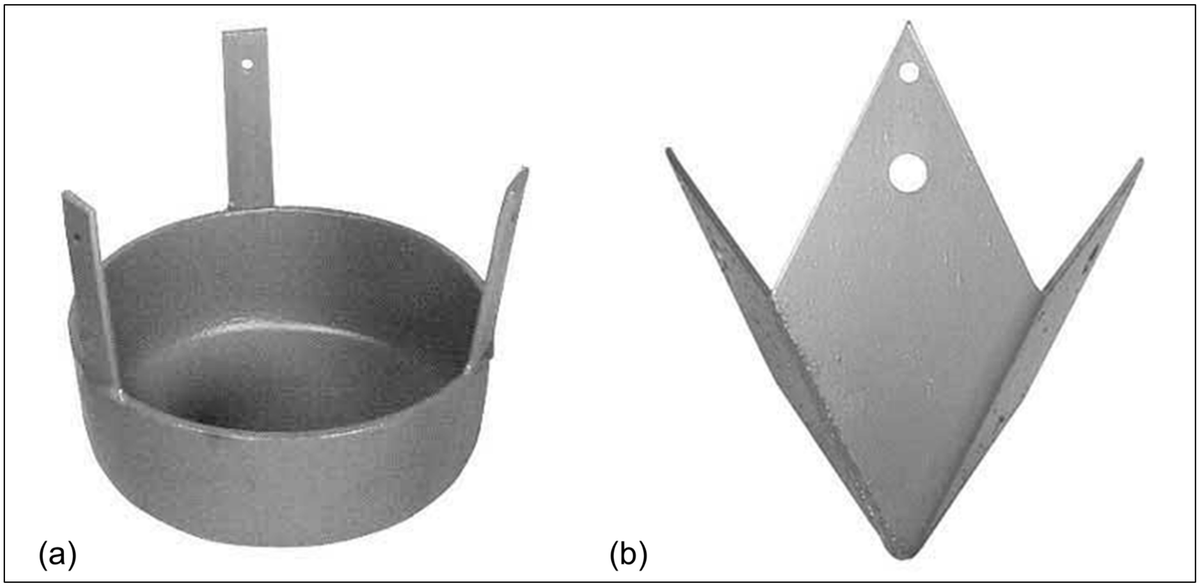

Figure 16-1 Timber pile toe attachments: (a) Pile boot and (b) Pile point (courtesy Skyline Steel).

Figure 16-2 Timber pile with pile boot attached.

16.1.2 Attachments at Pile Head

The American Wood Preservers Institute (Collin 2012), recommends banding timber piles with heavy metal strapping at the pile head prior to driving to prevent splitting. AASHTO (2010) requires collars, bands, or other devices to prevent splitting and brooming when the required nominal driving resistance is more than 200 kips on a timber pile. A photograph of a banded timber pile head is shown in Figure 16-3.

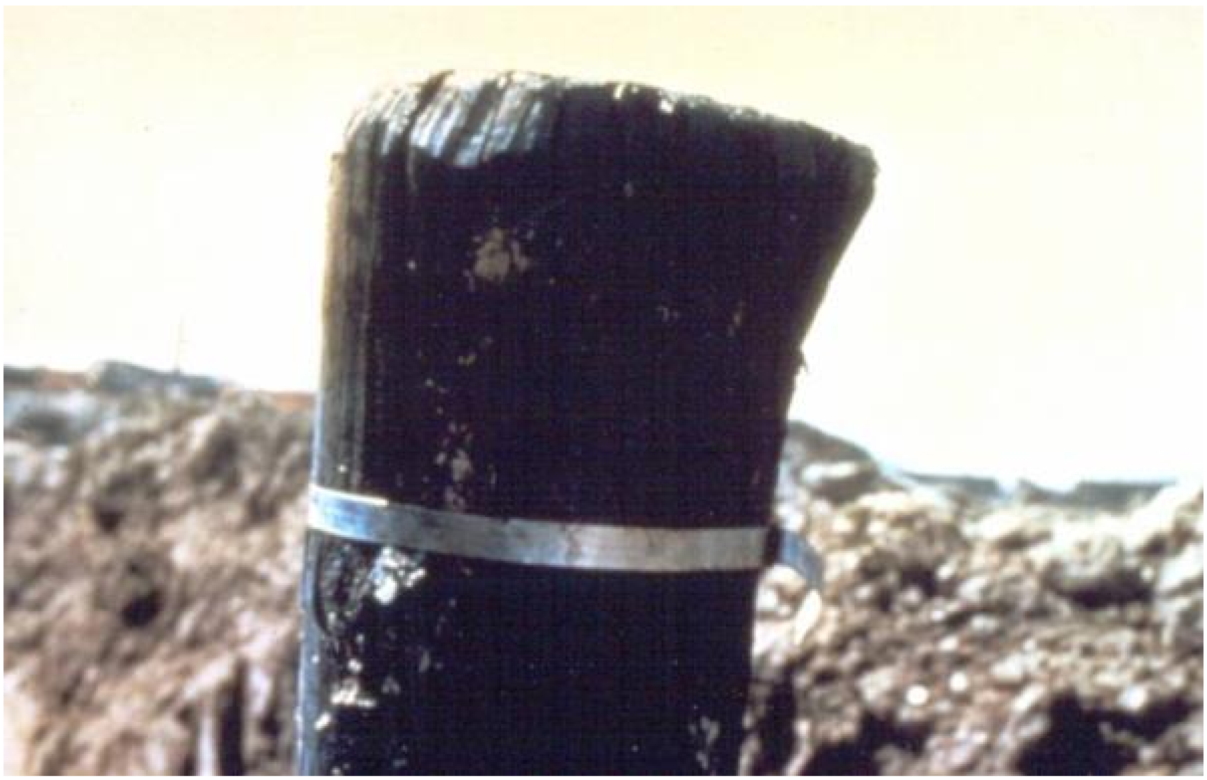

Figure 16-3   Banded timber pile head.

16.1.3 Splices

Timber pile splices are undesirable. It is virtually impossible to develop the full bending strength of a spliced timber pile. AASHTO (2010) states that timber piles should not be spliced unless specified in the contract documents or in writing by the engineer. When timber pile lengths prove insufficient for the required nominal resistance, longer piles should be ordered, or the foundation should be redesigned using additional piles of the furnished length.

16.2 STEEL H-PILES

Steel H-piles are often used to penetrate through hard or problematic upper soil layers and terminate in a very dense or hard bearing layer such as rock. The bearing layer can be at a depth that requires single or multiple H-pile splices. Depending on the nature of the surficial materials, pile toe attachments may also be needed to prevent or reduce pile toe damage. Pile toe attachments are discussed in detail in Section 16.2.1. Splicing methods for H-piles are described in Section 16.2.2. The importance of proper welding procedures and weld inspection are discussed in Section 16.5.

16.2.1 Pile Toe Attachments

Steel H-piles are generally easy to install due to the non-displacement character of the pile. Problems can 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 H-pile flanges and web may occur. Figure 16-4(a) and 16-4(b) show an extracted H-pile that was driven without a pile shoe through a deposit containing boulders. In Figure 16-4(a), the pile has just been extracted after dynamic test records indicated pile toe damage. The extracted H-pile has a boulder wedged between the flanges. In Figure 16-4(b), the H-pile has been laid on the ground and the extent of the toe damage is apparent including separation of the flange and web.

Commercially manufactured pile shoes can help minimize or prevent toe damage problems. Pile shoes are also desirable for H-piles driven to hard rock to facilitate distributing high localized contact stresses over a greater cross sectional area. This is particularly helpful when piles are to be driven onto 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 H-pile shoes are available in various shapes and styles as shown in Figure 16-5 (a through c). Pile shoes should be fabricated from cast steel conforming to ASTM A148/A148M (Grade 90-60). These shoes are attached to the H piles with fillet welds along the outside of each flange as illustrated in Figure 16-6. Most agencies have a standard detail for shoe attachment. In general, the lower portion of the H-pile flanges are beveled and a 5/16 inch fillet weld or greater depending on flange thickness used to attach the shoe.

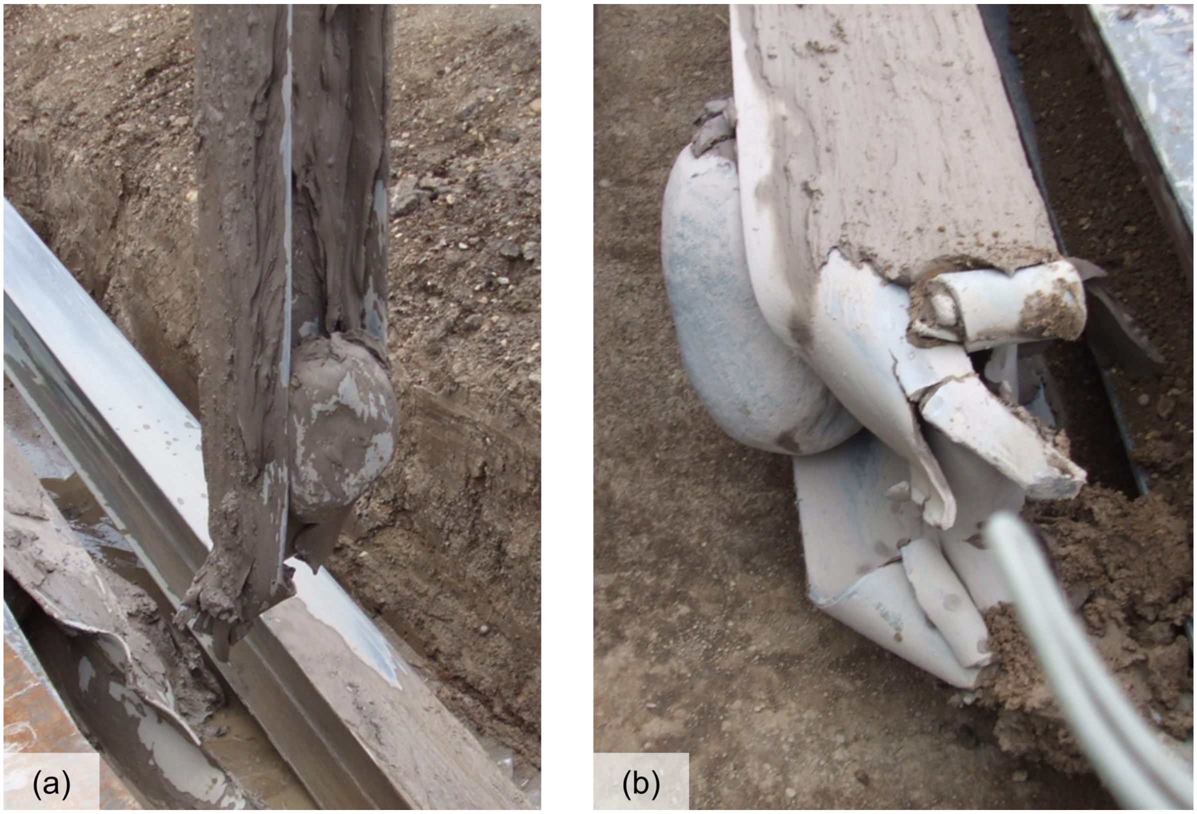

Figure 16-4 Damaged H-pile driven without pile toe protection (courtesy Wisconsin DOT).

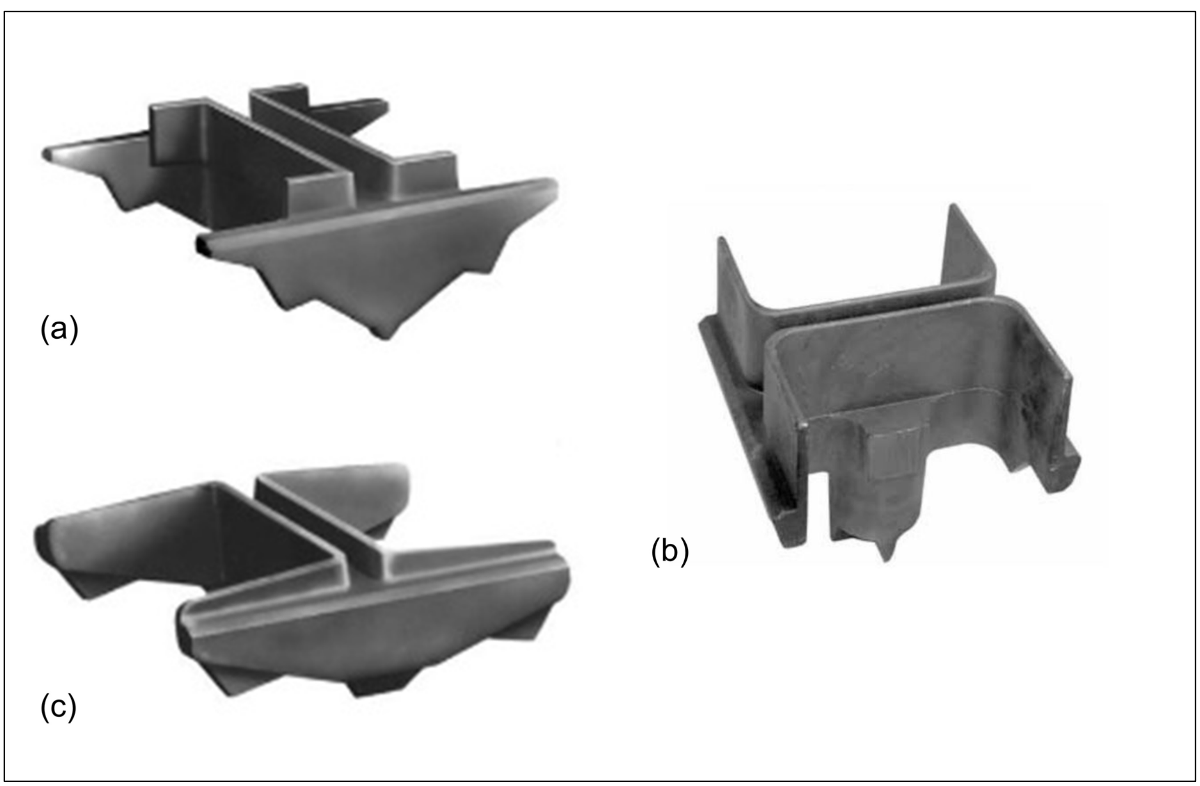

Figure 16-5 Styles of H-pile driving shoes (courtesy Skyline Steel).

Figure 16-6 Pile shoe welded to pile toe using fillet welds along exterior flanges.

Manufacturers recommend different shoe 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 provided they are manufactured from similar strength materials using 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.

16.2.2 Splices

H-pile splices are routinely made by using a full penetration groove weld along the web and both flanges as shown in Figure 16-7. Commercially manufactured H-pile splicers as shown in Figure 16-8 are also used. Some agencies also utilize a welded square or diamond backer plate detail in conjunction with a welded splice. H-pile splices must satisfy all the pile structural resistance requirements in axial compression, tension, and bending demanded by the foundation loading conditions. The appropriate splice details; a full penetration weld or an H-pile splicer, will be determined by the foundation loading conditions. Welded splice

Figure 16-7 Full penetration groove weld with backer bars.


Figure 16-8 H-pile splicer.

strength, beyond that required to survive pile driving induced stresses, is necessary for piles subjected to bending stresses and cyclic loading. Hence, it is essential that H-pile splices be performed by certified welders following approved procedures. Further discussion on field welds is discussed in Section 16.5.

For the manufactured H-pile splicer, a notch is cut into the web of the driven pile section and the splicer is slipped over that pile. A 5/16 inch fillet weld extending vertically from the four corner of the H-pile splicer is then made between the edge of the H-pile splicer and the H-pile flanges. The length of each fillet weld is typically on the order of 2.5 to 4 inches. The flanges on the add-on H-pile section are chamfered at 45 degrees to facilitate flange welding. Typically the add-on section of pile is positioned, aligned, and held while a 5/16 inch fillet weld is made between the edge of the H-pile splicer plate and the H-pile flanges. The length of each fillet weld is again on the order of 2.5 to 4 inches. After completion of the upper welds, either a full penetration or partial penetration groove weld (depending on design) is made along both flanges. A partially completed splice is shown in Figure 16-9.

Figure 16-9 Partially completed splice using H-pile splicer.

Most H-piles produced today are typically made from ASTM A-572, Grade 50 steel or higher. When H-pile splicers are used they should also be made from the same high strength steel as the H-pile.

16.3 STEEL PIPE PILES

Steel pipe piles can be designed to be driven either closed end or open end. Pile toe attachments are designed for both installation situations. Toe attachments can be used to minimize the potential for toe damage in difficult subsurface conditions as well as to help limit pile deflection. Steel pipe piles lengths can be easily increased in the field using welded or friction splices. Pile toe attachments are discussed in detail in Section 16.3.1. Splicing methods for pipe piles are described in Section 16.3.2. The importance of proper welding procedures and weld inspection are discussed in Section 16.5.

16.3.1 Pile Toe Attachments

Pile toe attachments on pipe piles are used to reduce the possibilities of pile toe damage and limit pile deflection. Problems during installation of closed end pipe piles may arise when driving through materials containing obstructions. In this case, piles may deflect and deviate from their design alignment to an unacceptable extent. When driving open end pipe piles through or into very dense materials, the toe of the pile may be deformed.

Closed end pipe piles are most frequently installed with a flat plate welded to the pile toe as shown in Figure 16-10. The flat plate thickness typically ranges from 0.75 to 1.0 inch thick for closed end pipe piles having an outside diameter of 18 inches or less. A thicker closure plate may be required by the engineer for larger diameter piles. The diameter of the closure plate is typically 5/8 inch larger in diameter than the outside diameter of the pipe pile in order to allow a 5/16 inch fillet weld to attach the closure plate to the pile toe. The closure plate can also be the same diameter as the pile in which case either the pile toe or the closure plate must be beveled so that a partial penetration weld can be made. End plates should be made of ASTM A36/A36M steel or better.

A conical toe attachment can also be used as a pile toe closure device for closed end pipe piles. Conical toe attachments can have a rounded shape as shown in Figure 16-11(a) or a 60 degree point as shown in Figure 16-11(b). These conical

Figure 16-10 Closure plate typical of closed end steel pipe pile.

attachments generally cost more than flat closure plate devices. In addition, flat closure plates generally develop a higher unit toe resistance. In the same soil profile, a closed end pipe pile with a conical toe attachment may drive slightly longer than a pipe with a flat closure plate for the same nominal resistance. A conical toe attachment is typically welded to the pile toe using a full or partial penetration groove weld. Conical toe attachments should be made from cast steel conforming to ASTM A148/A148M (Grade 90-60). A photograph of a conical toe attachment welded to a pipe pile is presented in Figure 16-12.

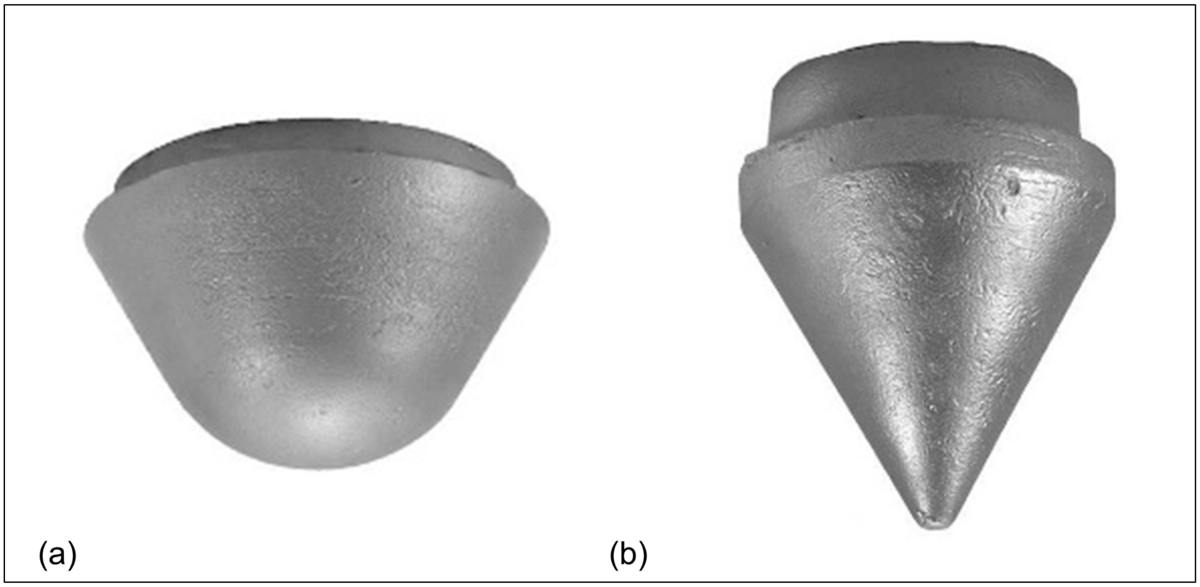

Figure 16-11 Conical toe attachments for closed end pipe piles: (a) Rounded (b) Pointed conical tip (courtesy Skyline Steel).

Figure 16-12 Conical toe attachment welded on a closed end steel pipe pile.

A “rock crusher” driving shoe is a preferred pipe pile closure device where sloping bedrock is encountered. This pile toe attachment consists of a thick flat plate with heavy steel plates forming a 45 degree point. A photograph of the rock crusher toe protection device is included in Figure 16-13.

Figure 16-13 Rock crusher driving shoe welded on a closed end steel pipe pile.

When installing open end pipe piles in dense gravel or to rock, the use of cutting shoes will help protect the piles and distribute high localized contact stresses over a larger pile area. Cutting shoes are made from cast steel conforming to ASTM A148/A148M (Grade 90-60). Both inside flange and external flange cutting shoe designs are available as shown in Figure 16-14.

Both cutting shoes have a bearing ring where the open end pipe pile sits on the cutting shoe. Cutting shoes with an outside flange can make drilling through the pile toe easier, if needed. However, the shoe perimeter is larger in diameter than the pile section and can therefore reduce shaft resistance. Both inside and outside flange cutting shoes can be welded to pile toe, however, the outside flange is sometimes used with only a friction fit. Photographs of inside flange and outside flange cutting shoes on open end pipe piles are presented in 16-15 and 16-16 respectively.

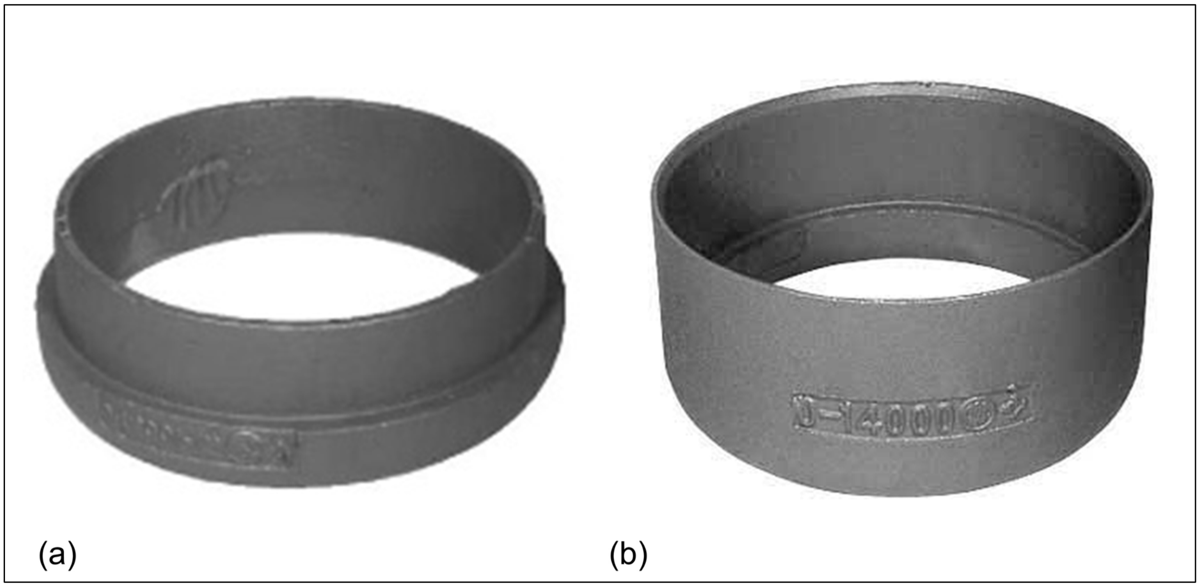

Figure 16-14 Cutting shoes for open end pipe piles:
(a) Inside flange and (b) Outside flange (courtesy Skyline Steel).

Figure 16-15 Inside flange cutting shoe welded to open end steel pipe pile.

Figure 16-16 Outside flange cutting shoe on open end steel pipe pile.

Large diameter open end pipe piles, 36 inches in diameter or greater, are frequently used in heavily loaded foundations having a limited number of piles. In these situations, use of a thicker wall bottom section over the lower two pile diameters is the preferred toe protection mechanism. The thicker wall toe section functions as an inside cutting shoe by having the same outside diameter as the design pile section. The thicker wall section helps distribute localized stresses and reduce the potential for pile toe damage that may otherwise develop due to partial pile toe contact with boulders, sloping bedrock, or on batter pile installations.

16.3.2 Splices

Steel pipe pile sections can be spliced by full penetration welding, or by using a mechanical drive-fit or friction splicer. Full penetration groove welds are depicted in Figure 16-17(a). A schematic of a friction splicer is shown in Figure 16-17(b). When a friction splicer is used in cases where the full bending strength of the pile is needed, fillet welds are required. However, the friction splicer must be fully seated into both top and bottom pile sections before performing the fillet weld to avoid subsequent cracking of the weld. Therefore, a full penetration weld is preferred in cases where the full bending strength is required.

For the full penetration weld, a backing ring, shown in Figure 16-18, is designed to sit between both pile sections and aides in pile alignment. Pins extending from the

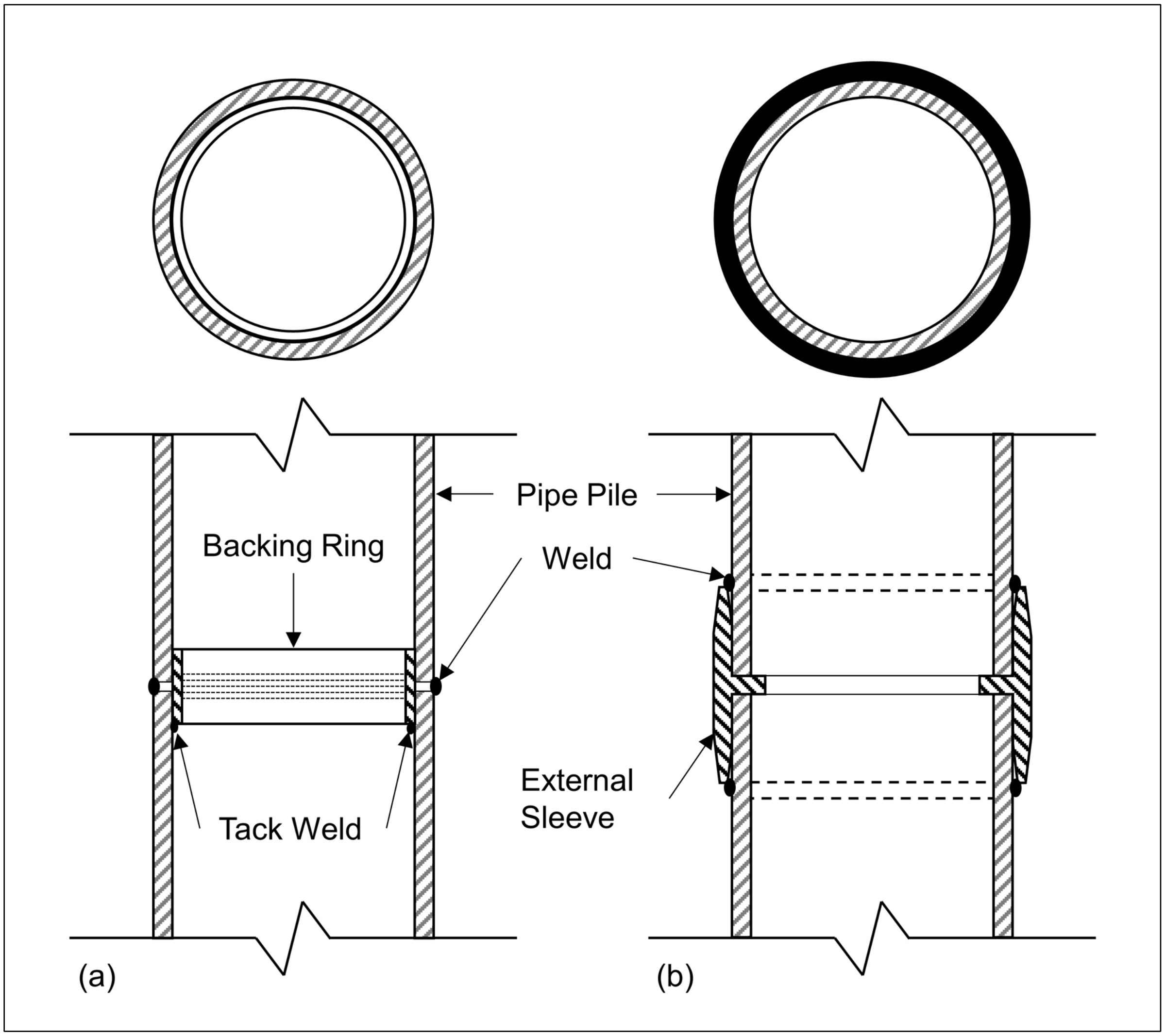

Figure 16-17 Splices for pipe piles.

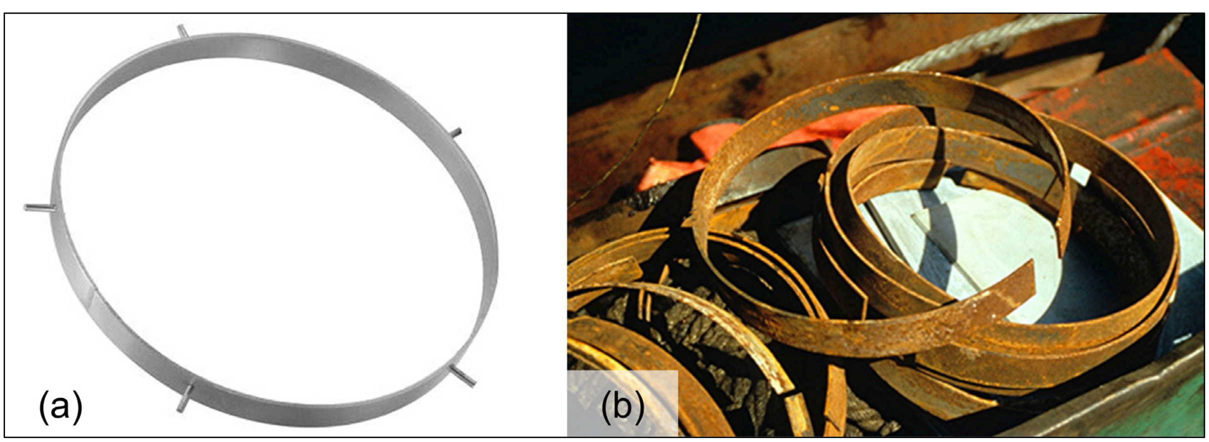

Figure 16-18 Backing rings for pipe pile splicing
(a) with pins (courtesy Skyline Steel) and (b) without.

backing ring provide a gap to facilitate welding the root pass. A photograph of the backup ring in place on a lower pile section is shown in Figure 16-19. Following completion of the root pass, additional passes are made around the splice to complete the full penetration weld and connect the pipe pile sections. Figure 16-20 shows a completed splice.

Figure 16-19 Backing ring tack welded to bottom pile.

Figure 16-20 Welded pipe pile sections.

In applicable situations, pipe piles can also be spliced with manufactured drive-fit or friction splicers similar to the one shown in Figure 16-21. These splicers are typically cast from ASTM A27 (Grade 65-35) or ASTM A148/A148M (Grade 90-60) steel and are designed with a taper for a frictional connection to eliminate the need for welding. A 3/8 inch bearing is located at the midpoint of the friction splicer for load transfer from the top pipe section to lower pile section. Little advanced preparation is required for this splice; however the adjoining pile sections must be square. If the initial pile section has some pile top damage following driving, the damaged section must be cut off and made square for a suitable friction connection with the drive-fit splicer. Unless a friction splicer is fillet welded to the top and bottom pile sections, the full pile strength in bending will not be provided. The suitability and location of friction splicers on piles subject to uplift loads should also be determined by the design engineer. If friction splicers are used on spiral welded pipe piles, fillet welds are also often necessary to provide a barrier to prevent ground water from leaking into the pile. A completed splice using the drive-fit friction splicer is shown in Figure 16-22.

Figure 16-21 Drive-fit mechanical friction splicer for pipe pile (courtesy Skyline Steel).

Figure 16-22 Drive-fit mechanical friction splicer on pipe pile.

16.3.3 Constrictor Plates

As discussed in Section 6.4.2 and Section 7.10 7, large diameter open ended pipe (LDOEP) piles can "core" or remain unplugged during driving. To facilitate plugging at a targeted depth in the soil stratigraphy, a constrictor plate is sometimes designed and welded inside the pile. Figure 16-23 shows the top surface of a constrictor plate with stiffeners welded inside a 60 inch O.D. pipe pile. A different constrictor plate detail is presented in Figure 6-14 of Section 6.4.2. Constrictor plates are often used inside large diameter open end pipe piles to force plugged behavior and a larger nominal resistance. Research on LDOEP pile design including constrictor plates is ongoing.

Figure 16-23 Constrictor plate installed inside pipe pile.

16.4 PRECAST CONCRETE PILES

16.4.1 Pile Toe Attachment

The toe of precast concrete piles may be damaged in compression under hard driving. In hard driving conditions and for toe bearing piles on rock, special cast steel toe attachments can be added to the pile during casting. However, toe attachments are not routinely used for concrete piles.

When necessary, a flat cast steel shoe as shown in Figure 16-24(a), or an "Oslo Point" or “Rock Injector Point” as shown in Figure 16-24(b), can be cast into a concrete pile for toe protection. The characteristics of Oslo or Rock Injector points are such that the pile can be “chiseled into” most rock types to ensure proper seating. These points generally have a 3.5 inch diameter hardened steel point housed in the steel casting attached to the concrete pile. All toe attachments for precast concrete piles must be attached during casting of the piles and not in the field. A photograph of an Oslo point with and without the hardened steel point in the housing is presented in Figure 16-25.

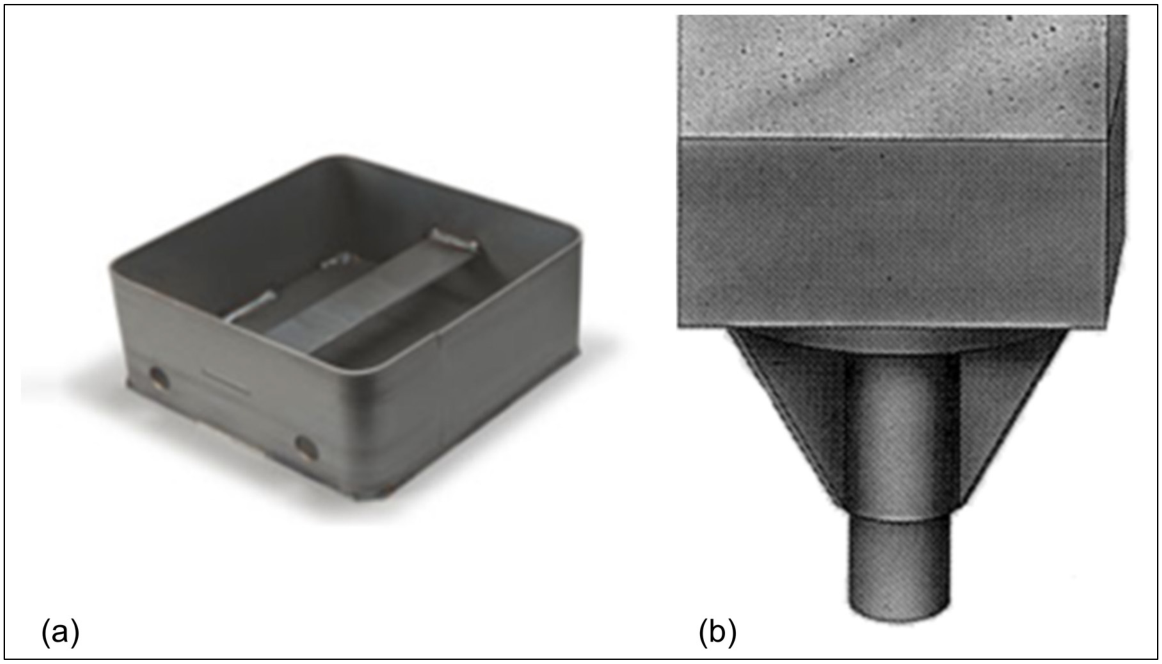

Figure 16-24 Pile toe attachments on concrete pile:
(a) Flat shoe (courtesy Titus Steel) and (b) Oslo point.

Figure 16-25 Oslo point for concrete pile.

Another toe attachment sometimes used with a prestressed concrete pile is an H-pile or pipe pile section cast into the pile toe. These devices are not typically used for toe protection but more to facilitate overall pile penetration depths in very dense or hard materials where it may be difficult to achieve the required pile penetration depth with a displacement pile. Depending on the characteristics of the materials to be penetrated, the embedded steel sections may or may not be equipped with its' own toe protection attachment. A photograph of a square prestressed concrete pile section with an embedded H-pile section at the pile toe is presented in Figure 16-26. In this situation the steel section must be appropriately sized for the concrete section, foundation loading conditions, and subsurface conditions to prevent overstressing.

Figure 16-26 Steel H-pile cast into toe of prestressed concrete pile.

16.4.2 Splices

Virtually all 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. 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. Illustrations of these splice types are provided in Figure 16-27. The wedge and pinned connectors can be classified as mechanical splices while the connector ring is a sleeve splice.

As part of a FDOT research effort on development of a new post tensioned concrete pile splice, Mullins and Sen (2015) summarized the available splicing systems for prestressed concrete piles including the capacity, failure type, durability, installation, and production aspects. The results of this summary are presented in Table 16-1.

Table 16-1 Summary of Precast Concrete Pile Splices (after Mullins and Sen 2015)

Name of Splice	Type	Capacity	Failure Type	Durability	Installation	U.S. Produced
Epoxy Dowel	Dowel	Poor	Ductile	Good	Moderate / Poor	Yes
Kie-Lock	Mechanical	Good	Ductile	Moderate	Good	No
ICP	Welded	Good	Brittle	Moderate	Moderate	No
NU Chuck	Bolted	Good	Brittle	Moderate	Untested / Moderate	Yes
UF Tube	Grouted Tube	Good	Ductile	Good	Poor	Yes
GYA	Mechanical	Good	Ductile	Moderate	Good	No
Macalloy	Post Tensioned	Good	Ductile	Good	Moderate / Poor	No

In the above table, capacity was defined as how well the splice met the flexural and tensile strength requirements for driving forces and pile loads. The failure type identified how the splice fails. Ductile failure is preferable to brittle failure. Durability noted the splice resistance to reinforcement and strand corrosion. Installation identified the time and labor requirements to complete the splice and U.S. produced identifies whether the splice is made in the U.S. Foreign made splices require special exceptions on some Federal funded projects. Some splices are only applicable for a specific pile type such as the UF Tube which is designed for voided concrete piles.

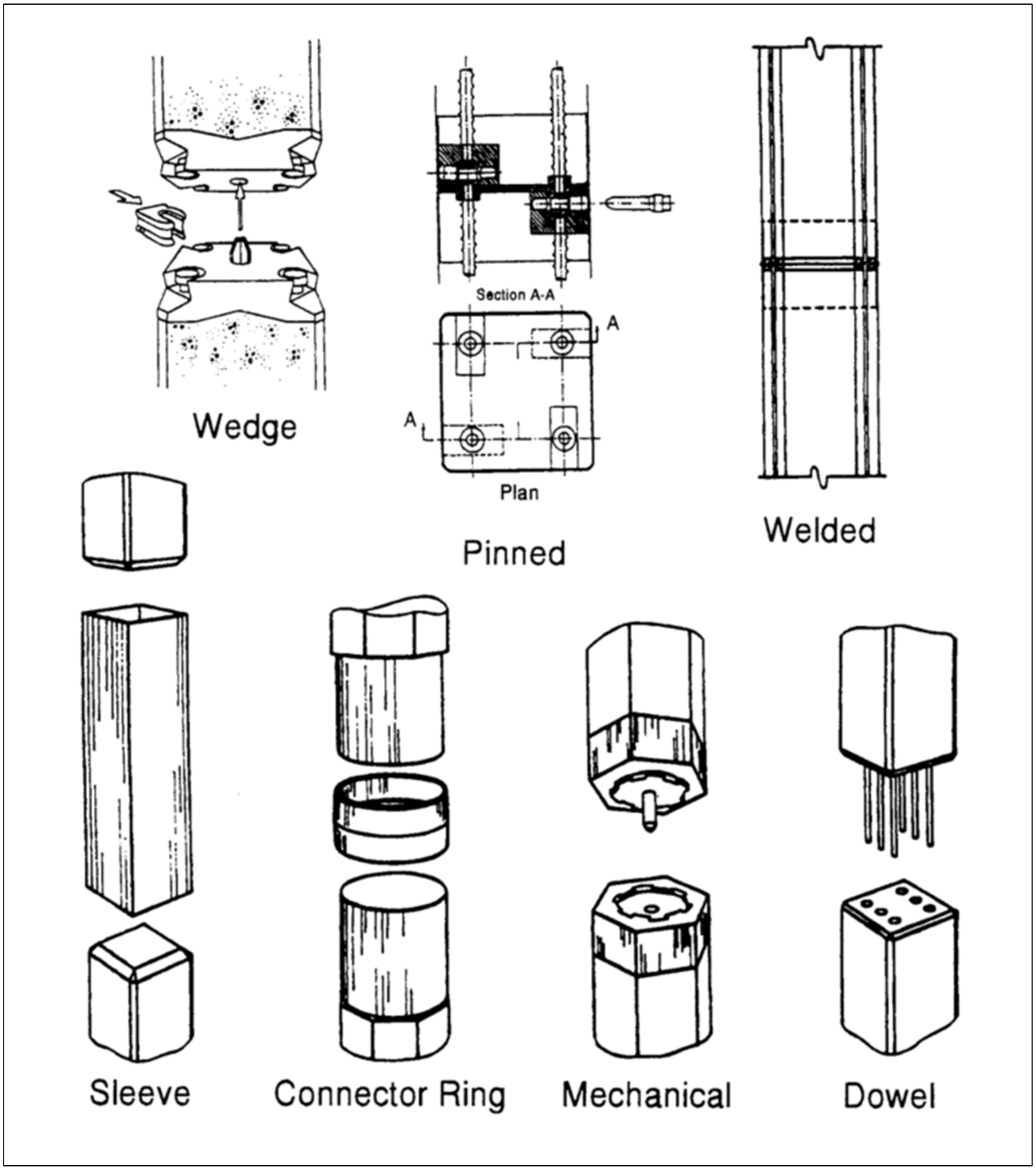

Figure 16-27 Commonly used prestressed concrete pile splices (after PCI 1993).

A schematic of an epoxy dowel splice for prestressed concrete pile is presented in Figure 16-28 while photographs of the splice in progress are presented in Figure 16-29(a) and 16-29(b). The bottom pile section to be spliced requires holes to 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. Pile driving can typically resume on epoxy dowel splices one to two days after epoxy placement. Epoxy dowel splices can be time consuming but are comparatively inexpensive. These splices have been 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. As noted by Mullins and Sen (2015), the splice has limitations meeting all the flexural and tensile strength requirements for driving forces and foundation loads.

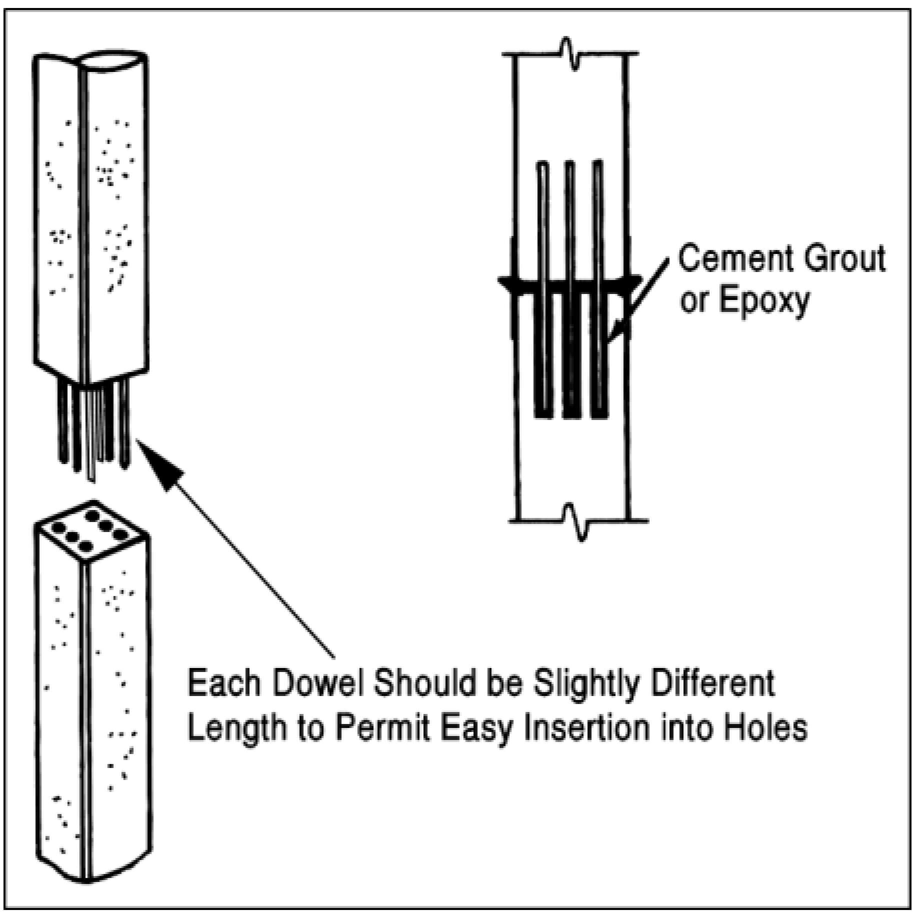

Figure 16-28 Cement/Epoxy-dowel splice (after Bruce and Herbert 1974).

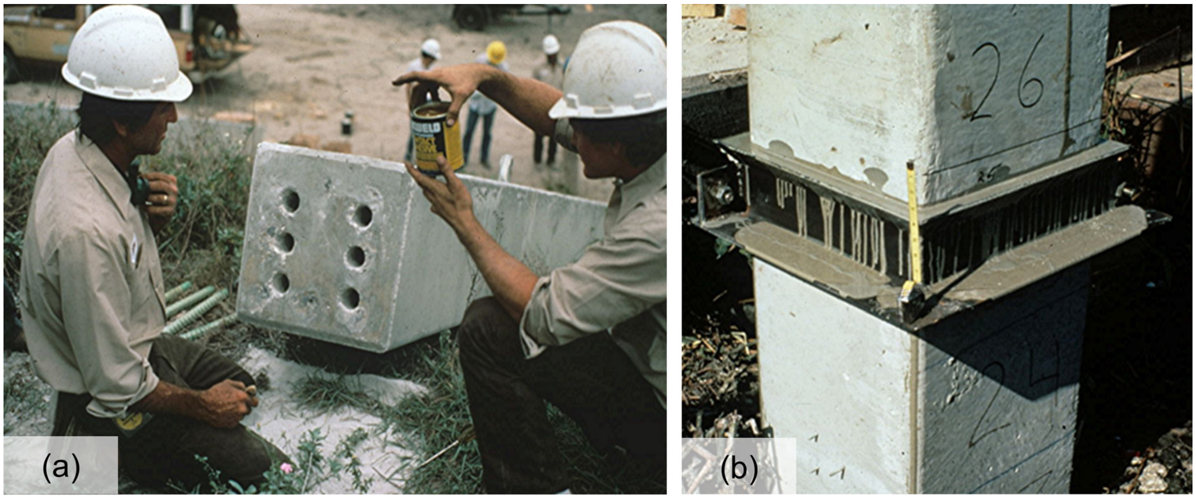

Figure 16-29 Epoxy-dowel splice: (a) pile with core holes and (b) splice in progress.

Welded splices require having steel fittings cast into the end sections of the concrete pile when manufactured. Sections are welded around the entire perimeter. Figure 16-30(a) shows the beveled steel fitting at the head of the driven pile section and Figure 16-30(b) shows the completed ICP pile welded splice.

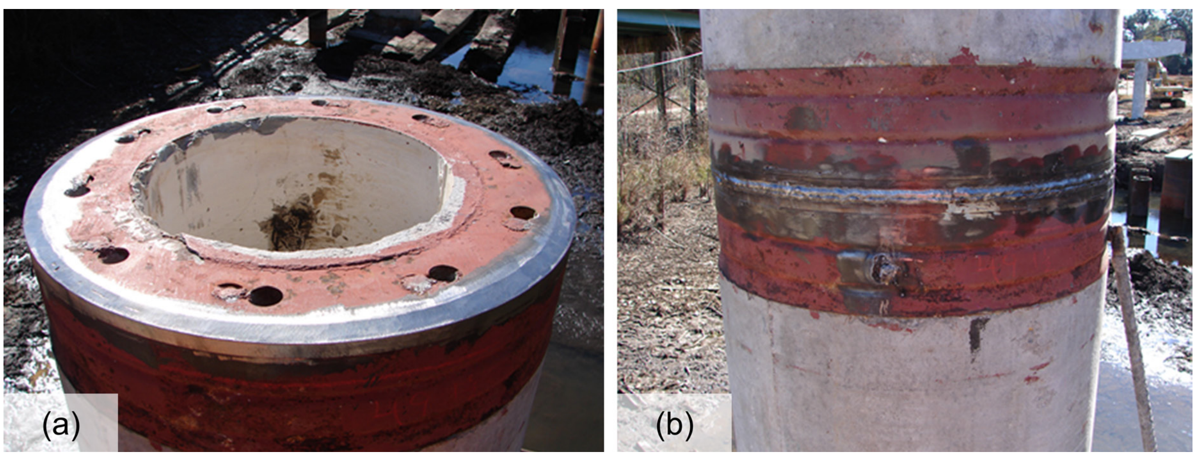

Figure 16-30 Welded ICP splice: (a) beveled steel fitting and (b) welded splice.

Most mechanical splices for prestressed concrete piles, such as the Kie-Lock, Herkules, and ABB, among others, are made of steel castings and are available for square, octagonal, hexagonal, and/or round sectional shapes. The steel castings are installed in the formwork of a prestressed concrete pile prior to concrete placement. The Kie-Lock (Figure 16-31) and Hercules splices require mating both male and female castings, while most other mechanical splices are gender neutral. All mechanical splices are then locked by inserting bars, 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.

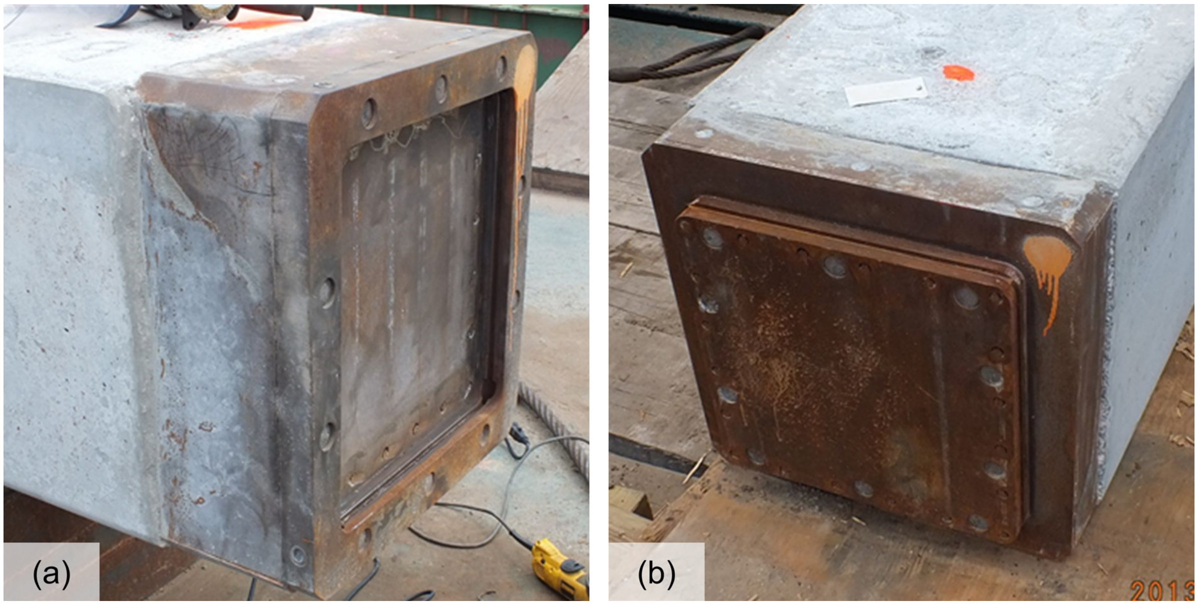

Figure 16-31 Kie-Lock splice.

A new concrete pile splice is under development for the Florida DOT in research by Mullins and Sen (2015). The splice is shown in a horizontal position in Figure 16-32. The splice is post-tensioned after joining the two concrete pile sections with the full prestress level restored across the splice and adjacent pile sections. For this splice, anchorages for the strands are cast into the lower section and open ducts are cast into the upper pile section. Steel strands are then locked into the lower anchor blocks and inserted though the upper pile section. After the two pile sections are mated, the steel strands are post-tensioned and the remaining duct voids grouted.

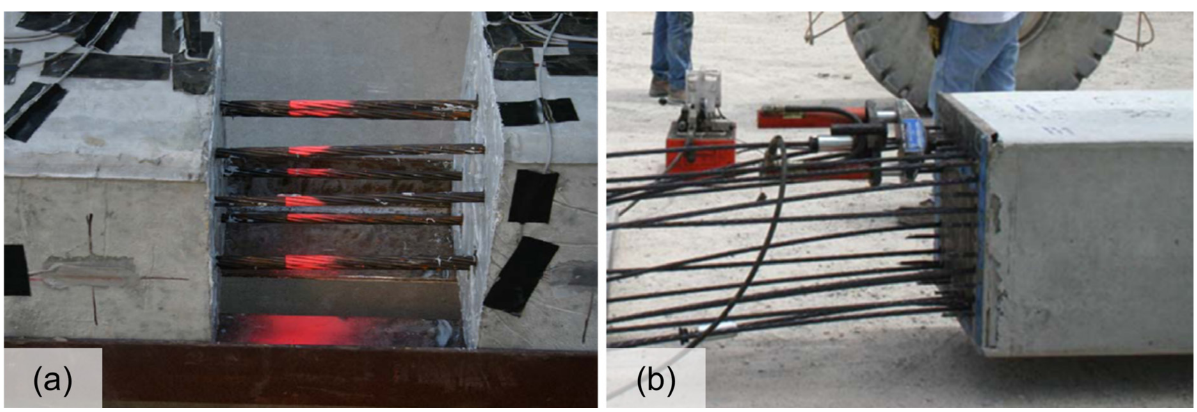

Figure 16-32 Post-tensioned splice (a) splice joint and (b) post tensioning strands in upper section (courtesy University of South Florida).

Sleeve type concrete splices are illustrated in Figure 16-33. These concrete pile splices can be rapidly applied and are very effective in reducing tension driving stresses. However, they cannot be used where static uplift loading will be required.

A Hawaiian can sleeve splice is shown in Figure 16-33(a). The sleeve must have sufficient length and strength if lateral or bending loads are anticipated. A Bruns connector ring splice is shown in Figure 16-33(b). The Bruns splice requires steel castings at the ends of the concrete piles. The shorter connector ring design has limited tensile and flexural strength and is therefore not recommended for designs with those loading considerations.

Figure 16-33 Sleeve splices: (a) Hawaiian can and (b) Bruns splice.

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.

16.5 WELDED SPLICES AND TOE ATTACHMENTS

Field welding is required on many pile splices and pile toe attachments. Weld quality is paramount in maintaining the full pile material strength. Due to the importance of field welding quality, the Michigan Department of Transportation developed a detailed document “Field Manual for Pile Welding” (2012). The New York State Department of Transportation also devoted twelve pages in their “Pile Driving Inspection Manual” (2012) to pile splicing and toe attachments. The NYSDOT document includes minimum splice times for weld preparation and welding as a function of pile section and splice type.

If not performed properly, welds at splice locations can crack or break during driving. Examples of welded pile splices that failed during driving are presented in Figures 16-34 and 16-35. The pipe pile weld in Figure 16-34 cracked as a result of bending stresses occurring during hammer-pile alignment. The weld shown in Figure 16-35 broke a few hammer blows following splicing. The H-pile splice had been completed in low temperatures without properly preheating the pile metal. The main sources of welded splice problems, when they occur, can be traced to poor pile surface preparation, insufficient bevel or root opening, or general non-compliance with approved welding procedures or splice details.

Figure 16-34 Cracked weld on pipe pile splice.

Figure 16-35 Broken weld on H-pile splice.

16.5.1 Welding Surface Preparation

Prior to commencing welding, the pile joint to be welded must be properly cleaned. If the ambient temperature is below 32 degrees Fahrenheit the weld area must also be preheated. Figure 16-36 illustrates a steel pipe pile head which has been properly cut and beveled at the required angle to accommodate a welded splice. Without the appropriate bevel, weld penetration will be reduced, and it will not be possible to complete a full penetration groove weld. In Figure 16-37, a grinder is being used to prepare the pile head surface for welding. Typically, the add on H-pile section will have a 45 degree bevel with a 1/8 or 1/4 inch root opening to complete a full penetration weld. In Figure 16-38, two H-pile sections have been positioned for splicing. The specified splice detail required a full penetration weld. However, a full penetration weld is clearly impossible given the lack of a root opening and bevel shown in the figure.

Figure 16-36 Beveled edges on pipe pile in preparation for weld.

Figure 16-37 Grinding edges of web and flange in preparation for weld.

Figure 16-38 H-pile splice preparation without adequate root opening or bevel.

Proper pile section alignment is required prior to splicing. Figure 16-39 illustrates poor section alignment, a poor quality groove weld on the flange, and poor quality and insufficient fillet weld lengths on a splice with an H-pile splicer. When H-pile splicers are used, section alignment and root opening should be checked prior to welding. Figure 16-40 shows an extracted H-pile from the same project where dynamic testing indicated a cracked weld during driving. The H-pile was extracted after driving; however only the upper pile section returned. The pile separated at the poorly welded splice with the H-pile splicer and lower pile section lost below ground.

Figure 16-39 Misalignment between pile sections and poor quality weld.

Figure 16-40 Extracted H-pile upper section. Weld cracked during driving and extracted section retrieved without H-pile splicer or lower pile section.

16.5.2 Temperature Requirements During Welding and Splicing

In colder climates where the ambient air temperature is below 32 degrees Fahrenheit, the steel in the vicinity of the splice should be preheated to a minimum temperature of 70 degrees Fahrenheit within 3 inches of the weld immediately prior to welding and maintained at a minimum of 50 degrees Fahrenheit until the welding is complete. No welding should be performed when the ambient air temperature is below 0 degrees Fahrenheit. Figure 16-41 illustrates an H-pile being preheated at the splice location prior to welding.

Figure 16-41 Heating pile splice location prior to welding.

16.5.3 Welding Pile Toe Accessories

Pile toe attachments should be welded to the toes of H-piles and pipe piles following the approved welding procedure by approved welders. In Figure 16-42, small spot welds; one on the web and one at the corner of each flange have been used to attach the driving shoe to the pile toe. The limited welding can facilitate the loss of the driving shoe when difficult driving conditions are encountered. A partial penetration groove weld along the full length of the H-pile flange is shown attaching the driving shoe to the H-pile in Figure 16-43. This detail greatly reduces the possibility of driving shoe loss or the shoe not performing as intended when cobbles or bedrock is encountered.

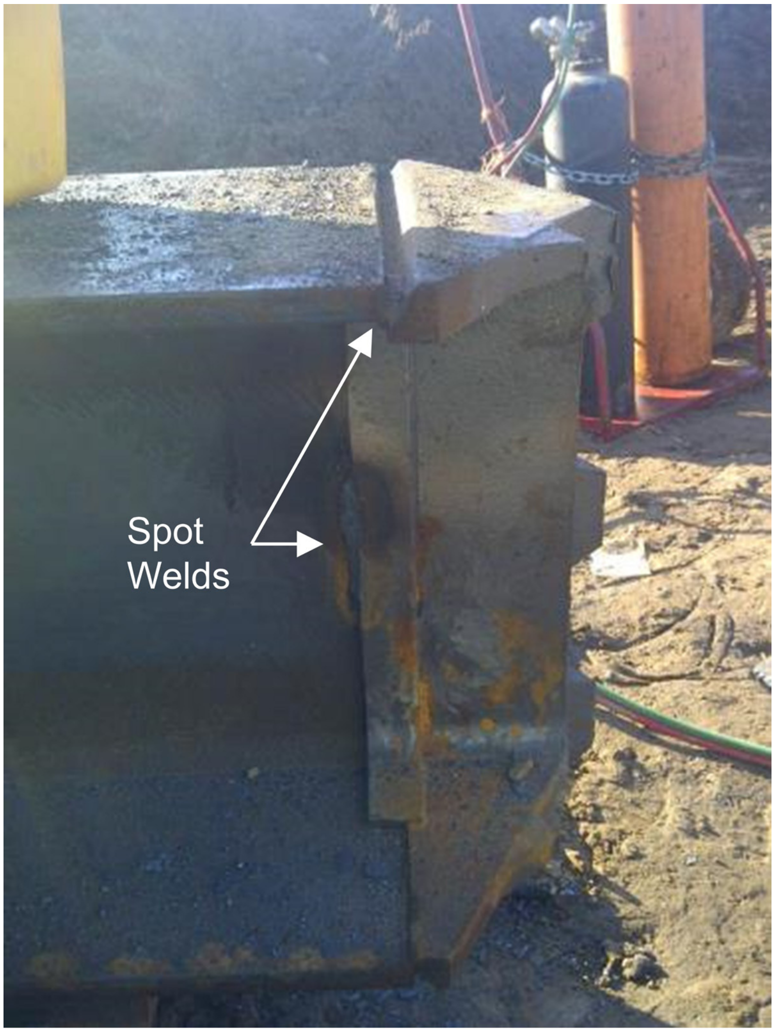

Figure 16-42 Unsatisfactory spot welds used for shoe attachment.


Figure 16-43 Full groove weld between pile and toe attachment.

16.5.4 Welded Splice Checklist

The following checklist is from the Michigan Department of Transportation Field Manual for Pile Welding (2012). It highlights several key items to be addressed for successful field welded splices.

16.5.4.1 Preparation

- Welders should be certified by an authorized agency to perform the type of work, and produce the type of weld to be used.
- A Weld Procedure Specification (WPS) should be developed and approved prior to work commencement.
- Joint preparation, pile alignment, root openings, bevels and fit up should be completed according to contract plans.
- Weld surfaces should be ground and cleaned. Coatings, oil, grease, rust, dirt, moisture and other contaminants within the weld zone should be removed.
- When ambient air temperature is below 32 degrees Fahrenheit, the pile splice area should be preheated, and maintained at 70 degrees. Welding should not be performed when ambient temperature is below 0 degrees Fahrenheit. Heating and housing may be approved by an engineer.
- The contractor should have an approved quality control plan, and perform welding with respect to the established quality control plan.

16.5.4.2 During Welding

- Electrodes should be properly stored. This includes:
    - Electrodes should be stored in a hermetically sealed container or hot box with a minimum temperature of 250 degrees Fahrenheit.
    - One exposed to the atmosphere, electrodes should be either used within two-hours, or redried for two hours at a minimum of 500 degrees.
    - Electrodes that become wet should be discarded and not used.
    - Electrodes should be discarded if they are dropped on the ground, exposed to rain or not properly stored.
- Tack welds used during fit up should not be part of the final weld.
- The welder(s) should follow the approved WPS, contractor quality control plan and should inspect his/her own work.
- The welder(s) should clean between passes, removing all slag and repairing discontinuities between passes.
- The welder(s) should back gouge to sound metal for full penetration groove welds before welding the backside (if applicable).

16.5.4.3 Final Weld Inspection

- All slag, spatter, and debris should be removed from weld surface.
- The weld size, length and profile should meet project requirements.
- Arc strikes should be ground smooth.
- A final inspection for cracks or other discontinuities, porosity, undercut, underfill, overlap, lack of penetration, or lack of fusion should be performed before accepting the weld.
- Repairs or replacement of weld should be performed before accepting the weld.

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CHAPTER 17

DRIVING CRITERIA

Pile foundations must be installed to meet limit state requirements for compressive, tensile, and lateral resistances as well as to satisfy serviceability requirements. Driven pile foundations are therefore used to satisfy nominal resistance requirements, pile penetration requirements, or both. As the pile is driven into the soil or rock, observations of the pile penetration resistance or blow count provide an indication of the nominal geotechnical resistance. Several methods have been developed that correlate the observed blow count and the associated hammer operational performance with the nominal geotechnical resistance achieved.

Driven pile foundations are generally installed using a driving criterion that equates the blow count and hammer stroke to the nominal geotechnical resistance. A minimum pile penetration depth may often be included in the driving criterion in cases when scour, foundation settlement, uplift or lateral loading demands impact design performance. The correlation of the observed blow count and hammer stroke to the nominal geotechnical resistance can be established from one or a combination of two of the following methods: static load test (Chapter 9), dynamic testing with signal matching (Chapter 10), rapid load testing (Chapter 11), wave equation analysis (Chapter 12) or dynamic formulas (Chapter 13).

17.1 DEVELOPMENT OF THE PILE DRIVING CRITERIA

The foundation designer should specify the method to be used for determination of the driving criteria. Furthermore, construction personnel should clearly understand the method being used and its proper implementation on the project.

Brown and Thompson (2011) summarized current state transportation agency practices on driving criteria in the NCHRP Synthesis 418, “Developing Production Pile Driving Criteria from Test Pile Data.” This report surveyed 42 transportation agencies covering a wide range of pile types and sections, hammer types and sizes, project sizes, as well as soil and rock conditions. The agencies reported their predominant method of establishing driving criteria were, from most frequent to least frequent:

- Drive piles to a blow count determined from another pile that had previously been subjected to dynamic testing and analysis.
- Drive piles to a blow count determined from a dynamic formula.
- Drive piles to a blow count determined from a wave equation analysis.
- Drive piles to practical refusal.
- Drive piles to a specified tip elevation.
- Drive piles to a blow count determined from another pile that had previously been statically load tested.

The resistance verification methods mentioned above each have their own resistance factor in AASHTO (2014). Hence, a change in the resistance verification method during construction will necessitate driving production piles to either a higher or lower nominal resistance than shown on the original contract documents. For example, a change from dynamic testing with signal matching (ϕ_{dyn} = 0.65) to the FHWA modified Gates formula (ϕ_{dyn} = 0.40) will result in a 63% increase in the required nominal resistance. This can create problems if the pile driving system was not originally sized for the higher nominal resistance. Conversely, a switch from the FHWA modified Gates formula (ϕ_{dyn} = 0.40) to either a static load test (ϕ_{dyn} = 0.75) on one pile, or to 100% dynamic testing with signal matching will result in an 87% reduction in the required nominal resistance. In this scenario, reduced nominal resistance piles may allow the use of a smaller pile hammer, or may solve a problematic pile installation condition such as avoiding a damage causing boulder layer since piles with a reduced nominal resistance may terminate at a higher, and therefore, safer installation elevation.

When developing driving criteria, consideration should also be given to the effect of time dependent changes in the nominal geotechnical resistance. In conditions exhibiting soil setup, pile penetration resistances at the end of driving less than that required for the nominal resistance may be acceptable when setup is confirmed by later restrikes. Conversely in geomaterials that exhibit relaxation, pile penetration resistances at the end of driving should be greater than those needed for the required nominal resistance to account for the future loss of nominal resistance.

Restrike tests are typically performed in soils with time dependent soil strength changes to confirm the expected change in nominal resistance. Restrike tests are also performed on projects in the event unexpected changes in nominal resistance occur with time. In cases where time dependent soil strength changes are anticipated, static or dynamic restrike tests should be delayed by an appropriate waiting period until the anticipated soil strength changes have occurred. Additional information on time dependent soil resistance effects is presented in Section 7.2.4 and approximate waiting periods for various soil types is noted in Section 14.5.2.1. When time dependent soil strength changes occur, care has to be taken to assess the driving criterion for the soil condition at the end of driving. For example, if soil setup results in a 50% increase of nominal resistance from the end of driving to the time of an acceptable setup period, then piles may be driven to a blow count that produces the required nominal resistance divided by 1.5. Obviously, this reduced driving criterion may result in substantial savings compared to driving the piles to the full nominal resistance. Restrike tests should be performed on a representative percentage of production piles to substantiate that the anticipated soil setup occurs over the site. A contingency plan should also be in place in the event restrike results indicate less than required nominal resistance, such as performing a second longer term restrike or driving piles to a greater penetration depth.

17.2 PRACTICAL AND ABSOLUTE REFUSAL

As noted above, agencies sometimes use practical refusal as a driving criteria. Definitions for practical and absolute refusal are based on an approved hammer system operating properly at its maximum fuel or stroke setting unless hammer approval was established based on hammer operation at a reduced fuel or stroke setting. If refusal driving conditions develop in combination with suspect hammer performance, further evaluation of hammer energy transfer and the source of the refusal driving conditions are appropriate.

Practical refusal is defined as a pile penetration resistance (blow count) of 10 blows per inch for a maximum of 3 consecutive inches of pile penetration. Practical refusal is often used as a criterion for piles driven to a consistent and hard bearing layer. Blow counts greater than 10 blows per inch should be used with care for concrete piles and should be avoided for timber piles. Absolute refusal is defined as 20 blows for one inch or less of pile penetration. Driving should terminate immediately once either criteria are achieved with a properly sized and properly working hammer.

Practical and absolute refusal criteria should be used to avoid driving for an extended duration at excessively high and unreasonable blow count requirements. When seating a pile on hard rock, an absolute refusal criterion of 5 blows per ¼ inch or 10 blows per ½ inch may be preferred to 20 blows per inch to reduce the risk of pile toe or driving equipment damage.

When the required pile penetration depths cannot be achieved by driving without exceeding practical or absolute refusal criteria with the approved hammer, use of other pile penetration aids should be evaluated. Predrilling, jetting, and spudding equipment are discussed in Chapter 15.

17.3 PRACTICAL ISSUES AND CONSIDERATIONS

Prior to the onset of test pile or production pile driving, numerous practical issues and considerations should be clearly understood by the parties responsible for developing the driving criteria, approving a submitted criteria, or responsible for implementing the criteria in the field. Some of the more common issues encountered include:

- The lack of, or an incomplete or conflicting definition of, pile driving acceptance criteria within contract documents.

  Specifications should clearly identify the pile acceptance requirements (nominal resistance requirements in compression or uplift; required minimum pile penetration depth, if any, to meet lateral resistance or serviceability requirements, to avoid premature pile acceptance in competent layers overlying compressible ones, or due to high driving resistance caused by obstructions or poor hammer performance, etc.) as well as the resistance verification method.

- Clear identification of who determines when driving is terminated on test or production piles.

  On test pile projects, numerous parties may be present on the job site each with different responsibilities. A pile "inspector" will likely be present for the agency or for the DB team to record the pile penetration resistance or blow count as well as the associated hammer performance during test pile installation. The agency's project engineer or its design consultant may be present to observe test pile driving, another agency or contractor retained engineer may be present performing a dynamic pile test during driving, and the contractor's foreman may be documenting pile installation. Who and what (resistance verification method) determines when test pile driving is terminated should be clearly established in the contract documents and revisited prior to test pile installation. A detailed, manually recorded, pile driving log should always be maintained as part of a test pile installation.

During production driving, the pile inspector often serves as the agencies lone representative during driving and determines when the driving criteria established from the test pile program or by other means is achieved.

- Definition, or lack thereof, of time considerations in pile acceptance criterion.

  Pile acceptance criteria are based on achieving a nominal resistance at a specific time. Therefore, the pile acceptance criterion must clearly state when the criterion is to be applied such as at the end of initial driving or at the beginning of restrike. If based on restrike conditions, the time window when restrike tests are to be performed must be incorporated into the criterion. Oftentimes a time dependent nominal resistance correlation is established that requires restrike tests within a set time period. Restrike tests performed too early or too late may invalidate this correlation. Restrike tests on piles having limited drivability in soil setup environments may also need to be performed within a set time window to avoid refusal condition during restrike.

- Definition, or lack thereof, of driving criteria applicability and refinement as necessary to various substructure locations, subsurface conditions, or nominal resistance demands.

  Contract documents should identify test pile locations, the substructure units covered by those test pile locations, and that the resultant driving criterion is applicable to those substructure units. In some cases developing an appropriate driving criterion may necessitate the extrapolation of the test pile results to higher or lower nominal resistances, shorter or longer pile lengths, different batter angles, or slightly variable soil stratigraphy within the identified substructure locations by using refined and then modified wave equation analysis.

- The purpose of specifying and achieving a minimum pile penetration depth or required pile toe elevation.

  Minimum pile penetration requirements may be specified due to the depth of scour, the compressibility of soil layers, the presence of liquefaction susceptible layers, satisfying lateral or uplift loading demands, and other factors besides the nominal geotechnical resistance in axial compression. A minimum penetration depth should generally not be specified only to achieve an axial compression resistance.

- The difference between estimated and minimum pile toe elevations and when a minimum pile toe elevation should be specified.

  The estimated pile toe elevation defines the estimated pile length needed to achieve the nominal geotechnical resistance in axial compression. It is used to establish contract length estimates for bidding purposes. A minimum pile toe elevation is used to satisfy performance requirements.

- The difference between the required nominal geotechnical resistance based on design requirements and the resistance required to be overcome during initial driving (greater or lesser) based on unsuitable layers or time dependent changes in soil resistance.

  The required nominal driving resistance, R_{ndr}, is the soil resistance that must be overcome at the time of pile driving in order to provide the nominal geotechnical resistance, R_n, needed to satisfy loading and performance requirements. The nominal driving resistance can be significantly greater than the nominal resistance in cases where scourable, liquefiable, highly compressible, or otherwise unsuitable soil layers must be penetrated and/or where toe resistance may decrease after initial driving due to relaxation. The nominal driving resistance can also be less than the nominal resistance in cases where these conditions are not present and/or the suitable support layers exhibit soil setup.

- Unreasonable definitions of practical and absolute refusal criteria in terms of either an excessive driving resistance (blow count) or a hard to achieve minimum pile penetration length.

  Definitions of practical and absolute refusal with regard to blow count aspects were provided in Section 17.2. These criteria should be reviewed to avoid excessively high and unreasonable blow count requirements. When piles must penetrate into very hard geomaterials, weathered bedrock, or hard bedrock to satisfy any pile penetration depth requirements, predrilled holes may be required as it may not be possible to achieve the required penetration depth through driving alone.

- Pile termination and acceptance criteria in soft and hard rock.

Oftentimes piles can be driven into soft rock at considerable distances with a gradual buildup of both the pile penetration resistance as well as the nominal geotechnical resistance. In general, this condition is not too problematic unless the driving occurs at high blow counts for an extended duration of driving resulting in a reduction in hammer performance. In the case of hard rock, driving should be terminated relatively quickly once hard rock is encountered to reduce the risk of pile toe or hammer damage as discussed in Section 17.2.

- Understanding the limitations of dynamic tests and wave equation analyses in easy driving (less than 24 blows per foot) and hard driving (more than 120 blows per foot) situations.

  At pile penetration resistances less than 24 blows per foot and above 120 blows per foot, dynamic test and analysis methods can overpredict and underpredict the nominal resistance, respectively. At low blow counts (high set per blow), it is difficult for dynamic methods to easily separate the static and dynamic soil resistance effects resulting in a tendency to overpredict the static resistance. Use of a reduced hammer stroke or lower fuel setting can help improve the accuracy of dynamic methods in low blow count situations. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound nominal resistance estimates as not all of the resistance (particularly at and near the toe) is fully activated. In these high blow count situations, use of a larger hammer stroke, higher fuel setting, pile hammer with a greater rated energy, or variable stroke drop hammer can help improve dynamic method accuracy.

- Understanding that static load testing, dynamic testing with signal matching, wave equation analysis results, and dynamic formula results will give different values of the nominal resistance on the same pile.

  All of the above methods for nominal resistance verification have a different resistance factor, indicating they all differ in reliability. Therefore, it should be surprising if an identical nominal resistance were determined from each method. Driving criteria should be developed using the most reliable of the resistance verification methods available provided that the results from that resistance verification method are checked and appear reasonable.

  In the case of a static load test, the correlating data is only the pile penetration depth, blow count at end of driving, and the associated hammer stroke. Other tools must therefore be used to determine what other stroke and blow count combinations (the load test stroke and blow count values) are suitable for the required nominal resistance at the same penetration depth (accomplished by refined wave equation analysis) or if the same blow count and stroke are suitable for the nominal resistance at other locations and pile penetration depths (accomplished by a review of subsurface conditions and static analysis). A procedure for calibrating wave equation generated driving criteria to the dynamic testing signal matching results was described in Section 12.6.9. A similar approach may be used to calibrate wave equation based driving criteria to the static load test as described in Section 17.4.

- An understanding of how to perform and evaluate restrike dynamic test results.

  Ideally, the hammer stroke or fuel setting is selected such that the penetration resistance at the beginning of restrike falls between 2 to 3 and 10 blows per inch. In this situation, the test record to select for signal matching analysis is readily apparent. An early, high energy blow, with good data quality should be selected and analyzed for the nominal resistance. The restrike blow count should be carefully recorded over the full restrike event as the rate by which the blow count decreases from inch to inch can be helpful. When the restrike blow count is near or less than 24 blows per foot, a lower energy restrike blow should be chosen for signal matching analysis to reduce the potential for overpredicting the nominal resistance.

  In more difficult situations, limited pile movement may occur during restrike and several records may need to be analyzed with signal matching. Superposition of the activated shaft resistances under various restrike hammer blows may be used to assess the nominal geotechnical resistance. Initial restrike blows may mobilize the shaft resistance along the upper portion of the pile shaft. Later restrike blows may indicate more shaft resistance on the lower portion of the pile once the upper shaft resistance has started to breakdown. The toe resistance and shaft resistance on the lower portion of the pile from the end of drive analysis should also be reviewed and, if appropriate, used in a superposition case. When using the toe resistance from an end of drive situation, the analyst should be confident that relaxation in the bearing layer is not a consideration or overestimation of the nominal resistance could result by using superposition.

- Failure to make timely decision regarding the acceptance of production piles during installation.

The acceptability of production piles must be determined in a timely manner following completion of driving. Delay claims can result if decisions to splice and drive deeper, the need to add piles, or the need to drive replacement piles, cannot be made in a reasonable amount of time.

- Difficulty to accept that, in some cases, dynamic methods of estimating nominal resistance (dynamic formulas, wave equation and dynamic measurements) may yield conservative predictions of the true geotechnical resistance and thus, correlations and extrapolation between dynamic and static load test results are necessary.

  Dynamic methods can yield conservative estimates of the true geotechnical resistance in some situations. For example, an undersized hammer will not be able to mobilize the full soil resistance. Occasionally also, open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loading conditions, the soil inside a pipe pile or between H-pile flanges may slip and produce internal shaft resistances. Under static loading conditions, this soil may plug and move with the pile resulting in toe resistance over the full pile cross section. Hence both shaft and toe resistances may be different in open profile pile sections under static and dynamic loading conditions. Plugging behavior can also vary in different geomaterials. Careful interpretation and extrapolation of dynamic results is required in these situations. Additional commentary on pile driving of open pile sections is provided in Section 7.10.7.

17.4 EXAMPLES FOR ESTABLISHING DRIVING CRITERIA

The following simplified examples illustrate the considerations necessary to establish both economical and safe installation criteria (a) for 2% dynamic testing or (b) for one static test plus 2% dynamic testing. Various other ways of determining a driving criterion could be envisioned. For example, based on initial testing results, a refined wave equation bearing graph could be established, providing required blow counts for different nominal resistance values. Another example would be performing restrike dynamic testing after all piles had been installed to a wave equation calculated required blow count. That approach may be satisfactory if the bearing layer would be well documented and competent. However, if the final restrike tests would indicate insufficient nominal resistance, the piles would have to be redriven to a greater depth, or additional piles would be necessary, or 100% testing may be necessary. The latter remedy would then allow for an increased resistance factor and, hopefully make the reduced nominal resistance values acceptable.

17.4.1 Driving Criterion – Example 1

For a small bridge project with less than 100 piles, the foundation designer determined that dynamic testing with signal matching should be used for the nominal resistance verification method and driving criteria. Therefore dynamic testing with signal matching was specified on 2% of the production piles using the AASHTO resistance factor, ϕ_{dyn}, of 0.65. The bridge project involved pile driving at three pier and two abutment locations. Soil borings indicated a relatively uniform site consisting of sandy soils.

All dynamic tests were performed prior to production pile driving began at each substructure location. One pile at each abutment or pier was tested for a total of five test piles. This quantity exceeded the requirement of a minimum of two test piles for the uniform site condition. The factored loads were 117 kips at the abutments and 150 kips at the piers. This meant the required nominal resistance per pile was 117kips / 0.65 or 180 kips at the abutments and 150 kips / 0.65 or 230 kips at the piers. Piles were dynamically tested during initial driving and again during restrike the next day as applicable for the sandy soil conditions. Signal matching was performed on the dynamic test data acquired at the end-of-driving and beginning of restrike for each of the five test piles. A summary of the test results is presented in Table 17-1.

Table 17-1 Example 1 Summary of Requirements and Results

Test Pile Location	Factored Load (kips)	Required Nominal Resistance (kips)	Test Pile Blow Count (bl/ft)	Nominal Resistance from Signal Matching at EOD (kips)	Nominal Resistance from Signal Matching at BOR (kips)	Soil Setup Factor
N Abut.	117	180	40	185	215	1.16
Pier 1	150	230	47	235	275	1.17
Pier 2	150	230	49	240	290	1.21
Pier 3	150	230	50	245	290	1.18
S Abut.	117	180	42	188	215	1.14

At the abutments, the average test pile nominal resistances from signal matching at the end of driving and restrike were 187 and 215 kips respectively; the average setup factor was, therefore, 1.15. The average driving resistance at the end of driving varied between 40 and 42 blows/ft.

Using the nominal resistances calculated by signal matching, it was conservatively decided that for the production piles, the EOD nominal resistances could be 10% less than the required nominal resistance of 180 kips. The associated required penetration resistance, i.e., the driving criterion, was determined through refined wave equation analysis to be 36 blows/ft. This yielded a 164 kip nominal resistance at end of driving which with setup provided the 180 kip nominal resistance.

At the piers, the average test pile capacity from signal matching at the end of driving and restrike were 240 and 285 kips respectively; the average setup factor was, therefore, 1.19. The average driving resistance at the end of driving varied between 47 and 50 blows/ft.

Using the nominal resistances calculated by signal matching, it was once again conservatively decided that for the production piles, the EOD nominal resistances could be 10% less than the required nominal resistance of 230 kips. The associated required penetration resistance was determined through refined wave equation analysis to be 43 blows/ft. This yielded a 209 kip nominal resistance at end of driving which with setup provided the 230 kip nominal resistance.

Note: The test piles were driven to nominal resistance values in excess of those required, and were, therefore, acceptable as production piles.

17.4.2 Driving Criterion – Example 2

For a relatively small bridge project with two abutments, three piers and less than 120 piles, the foundation designer determined that static load testing and dynamic testing of 2% of the production piles (AASHTO resistance factor ϕ_{dyn}, of 0.80) should be used for the nominal resistance verification method and driving criteria. Therefore a static load test was planned at one pier along with dynamic testing and signal matching on 2% of the production piles. The bridge project involved pile driving at three pier and 2 abutment locations. Subsurface conditions are relatively uniform across the bridge site and consist primarily of medium dense fine sands.

Based on factored loads of 144 kips at the abutments and 184 kips at the piers, the required nominal resistances per pile were 180 kips at the abutments and 230 kips at the piers. The static load test frame and reaction system was set up at a pier location for a target loading capacity of 230 kips, but with a maximum test load capability of 350 kips.

The static test loading was preceded by dynamically monitoring the installation of first the reaction piles and then the static load test pile. The reaction piles were driven to a penetration depth determined by static analysis as required for their uplift load (350 kips/number of reaction piles) and to a corresponding driving resistance determined by wave equation analysis. The static load test pile was driven to a depth determined by static analysis for the required 230-kip nominal resistance and this was achieved at a final penetration resistance of 60 blows/ft. The load test frame was then constructed and one week after installation, the static load test was performed. This was followed by dynamic restrike tests of the static load test pile and all reaction piles.

The signal matching result at the end of driving on the load test pile indicated a nominal resistance of 220 kips. The static load test evaluated by the Davisson Criterion, failed at a nominal resistance of 260 kips. Signal matching on the restrike dynamic test data indicated a nominal resistance of 290 kips. While this 30 kip difference in nominal resistance between the dynamic test restrike and static load test was likely caused by the preloading of the granular soils by the static load test performed before the restrike, the geotechnical engineer attributed it to a systemic difference between the test methods. For conservatism, a correlation factor of 260 kips / 290 kips or 0.9 was recommended to be applied to all dynamic test results. For the load test pile this meant that the agreed upon EOD nominal resistance was 90% of 220 kips or 198 kips which yielded a soil setup factor of 260 kips / 198 kips or 1.31. The geotechnical engineer also recommended a conservative soil setup factor of 1.2 be used across the relatively uniform site which then required that the piles be driven to EOD nominal resistances confirmed by signal matching of 167 kips (180 kips / (1.2* 0.9) = 167 kips) at the abutments and 212 kips (230 kips / (1.2*0.9) = 212 kips) at the piers.

One production pile in each pier or abutment location was then dynamically tested and analyzed in this manner. When the test pile reached the required EOD nominal resistance, the associated penetration resistance or blow count was established as the driving criterion for the remainder of the production piles at that pier or abutment.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). (2014). AASHTO LRFD Bridge Design Specifications, US Customary Units, Seventh Edition, with 2015 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, D.C., 1960 p.

Brown, D.A., and Thompson, W.R. (2011). Developing Production Pile Criteria from Test Pile Data. National Cooperative Highway Research Program (NCHRP) Synthesis 418, Washington, D.C., 54 p.

CHAPTER 18

CONSTRUCTION MONITORING OF PILE INSTALLATION

Knowledgeable construction monitoring and inspection play a critical role in the proper installation of pile foundations. The general trend in cost effective pile foundation design is to use fewer piles each with a higher nominal resistance. This often requires the use of larger pile sections and bigger installation equipment. The construction monitoring of these pile installations becomes critical because of less redundancy (fewer piles required), smaller tolerances, and higher resistance factors.

Construction monitoring is only as good as the knowledge, experience and qualifications of the “inspector”. The role and duties of inspection personnel will vary depending upon the contract delivery method. However, pile inspection is still required for foundation acceptance regardless of how the necessary tasks are contractually assigned. Inspection personnel must understand their role on the project as well as the operation of the hammer and its accessories, the pile behavior, the soil conditions, and how these components interact. Most pile installation problems are avoidable if the 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 owner and engineer. 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 construction monitoring procedures and maintenance of pile driving records is provided in this chapter. Further details on the inspection of piles and pile driving systems may be found in the Deep Foundations Institute (1995); (1997), provided at the end of the chapter. The FHWA document “Performance of Pile Driving Systems” by Rausche et al. (1986), as well as hammer and equipment manufacturer’s literature are also good sources for information and details on hammer operation. The Florida Department of Transportation also developed and continues to update an excellent inspection manual for driven pile foundations as part of their inspector training program, Passe (1994). The pile installation monitoring procedures and record keeping methods presented herein should be refined as needed based on the project delivery method and agency practice.

18.1 MONITORING NEEDS BASED ON THE PROJECT DELIVERY METHOD

Transportation projects have historically been designed and constructed using the design-bid-build (DBB) delivery method. Design-Build (DB) and Construction Manager / General Contractor (CMGC) project delivery methods are increasingly being used due to the reduced project development and delivery time associated with these methods. An overview of the key construction monitoring items on driven pile foundations projects and how the typical duties associated with that item vary depending on DBB or DB contract delivery is summarized in Table 18-1. Construction monitoring duties for the driven pile foundations items noted in Table 18-1 for CMGC project delivery contracts may fall under either DBB or DB depending on the individual project.

18.2 ITEMS TO BE MONITORED

Regardless of the project delivery method there are several items to be monitored by the "inspector" on every pile foundation project. These include test piles driven to establish order lengths or for load testing, as well as for production piles. Items to be inspected 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. Review contractor's pile installation plan.

4. Inspection of pile driving equipment both before and during operation.

5. Inspection of test or indicator piles installation.

6. Inspection during production pile driving and maintenance of driving records.

Table 18-1 Overview of Key Construction Monitoring and Inspection Items and Agency Inspection Duties on Design-Bid-Build and Design Build Contracts.

Item	Design-Bid-Build	Design-Build
Pile Installation Plan including Hammer Submittal	Review pile installation plan. Perform or review required analyses for hammer approval if hammer approval performed.	Submit DB team pile installation plan to Engineer for conformance check with contract documents and requirements.
Test Pile Installation	Inspect piles, hammer and appurtenances before driving. Inspect and record test pile driving, pile splices, pile alignment, location, and reference elevation.	Observe test pile installation and documentation by others. Communicate any concerns to Engineer.
Nominal Resistance Verification	Perform resistance verification observations (blow count and stroke) or tests (static, dynamic).	Review results of resistance verification method test.
Driving Criteria and Production Pile Order Lengths	Analyze test pile and resistance verification results. Establish driving criteria and determine order lengths.	Review driving criteria established by design-build team.
Production Pile Installation	Inspect piles, inspect pile splices, inspect and record production pile driving, document final alignment, location, and elevations.	Check all piles met the established driving criteria and associated plan requirements.
Foundation Certification	Required documentation completed as part of overall inspection process.	Review foundation certification package from design-build team with Engineer.
Verification Testing	Not applicable.	Select piles for verification testing in coordination with Engineer.
Piling Problems and Resolution	Identify, document, and evaluate any piling problems. Coordinate problem resolution with Engineer, Designer, and Contractor.	Document resolution of any noted deficiencies.

A flow chart identifying the key components of the pile inspection process is presented in Figure 18-1. On DBB projects the Contact Engineer corresponds to the agency engineer. On DB projects, the Contact Engineer is the design-build team foundation engineer of record.

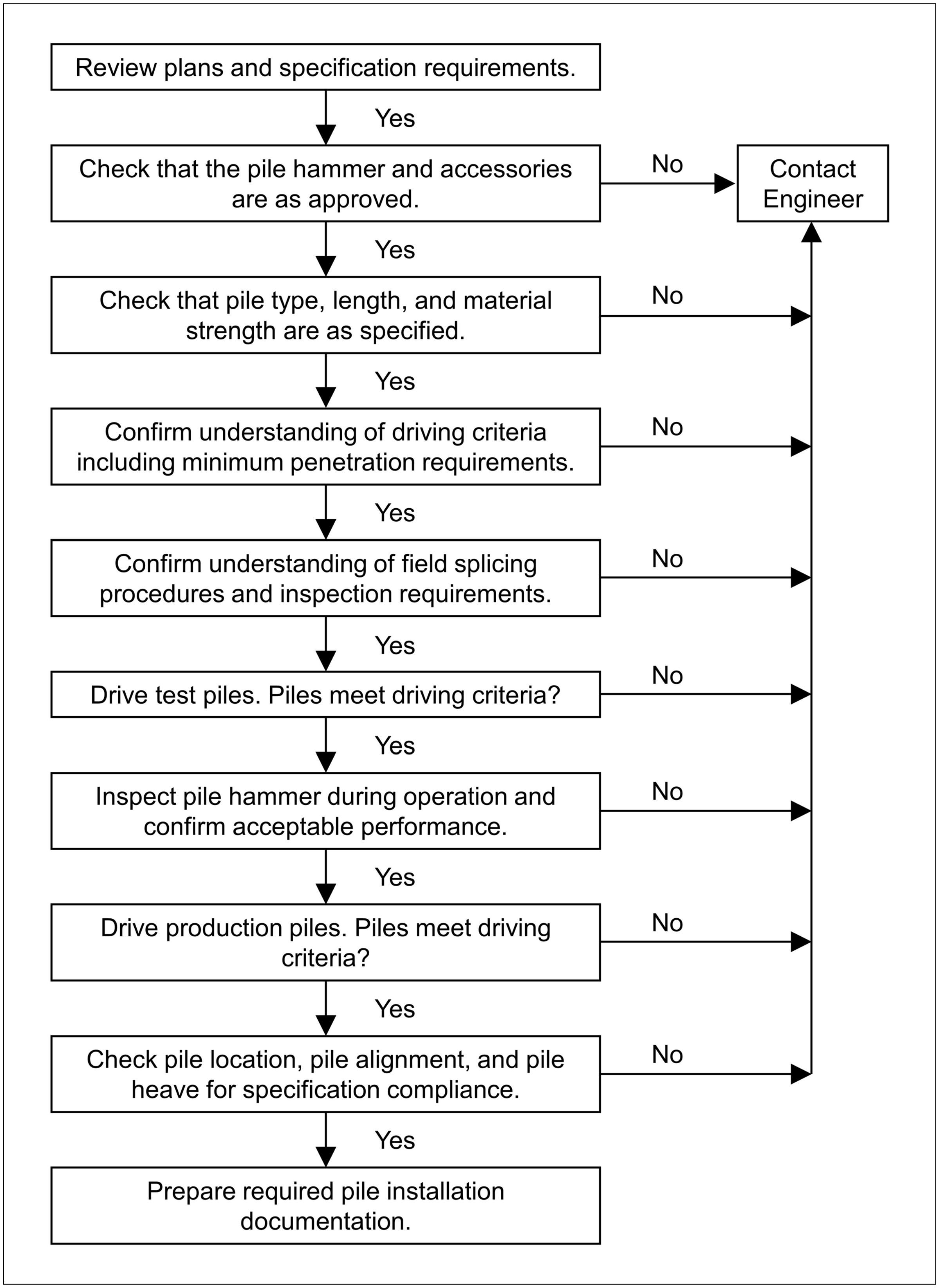


Figure 18-1 Key components of the pile installation inspection process.

18.3 REVIEW OF PROJECT PLANS AND SPECIFICATIONS

The first task in construction monitoring or inspection 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 the criteria to site specific hammer performance and soil conditions. The pile inspector should fully understand the responsibility of his/her organization in the DB or DBB project 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

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 construction monitoring reports during pile driving activities and upon completion of the project.

4. How to inform contractor when work deviates from contract documents without directing contractor's work.

18.4 INSPECTORS TOOLS

The checklist shown in Figure 18-2 is modified from Williams Earth Science (1995) and 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 (FHWA GEC-12)
- ☐ Performance of Pile Driving Systems Inspectors Manual (FHWA/RD-86/160)

Figure 18-2 Key components of the pile installation process (modified from Williams Earth Science 1995).

18.5 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. Section 18.7 provides similar sections for each major hammer type. A detailed pile driving inspection list for a project can be obtained by combining the check list for that projects pile type in Section 18.5, and hammer type in Section 18.7.

18.5.1 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 16.

g. Steel shoes which may be specified must be properly attached. Details are provided in Chapter 16.

h. Pile splices, if allowed by plans and specifications, must meet the project requirements.

18.5.2 Precast Concrete Piles

On many projects, inspection and supervision of casting operations for precast concrete piles is provided by the transportation agency. 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 tension release, 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.

18.5.3 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 16.

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 16.

d. Pile splices and pile toe attachments 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.

18.5.4 Steel Pipe Piles

The following should be inspected at the construction site:

a. The piles should be of specified steel grade, length, and 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 16.

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.

18.6 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 15, 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 pile driving equipment before driving:

1. The pile hammer should be the approved make and model as submitted or should meet specification requirements if no submittal is required.

Usually the specifications require certain hammer types and/or specify minimum and/or maximum energy ratings. The inspector should make sure for single acting air/steam 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, 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. For single acting and double acting hydraulic hammers, the contractor should supply a system for measuring and displaying the hammer energy or impact velocity.

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 ready for inspection prior to insertion into the helmet is presented in Figure 18-3. The Blue Nylon hammer cushion disks are shown in the lower right corner of the photograph. The hammer cushion thickness and diameter, the diameter of the helmet cushion pot, as well as the dimensions of the striker plate should all be measured by the inspector during a hammer cushion inspection. A damaged aluminum plate, found during a cushion check of an aluminum and micarta hammer cushion, is displayed on the left hand side of Figure 18-4.

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. A poorly fitting helmet often results in pile head damage. 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.

Figure 18-3 Check of Blue Nylon hammer cushion material before use.

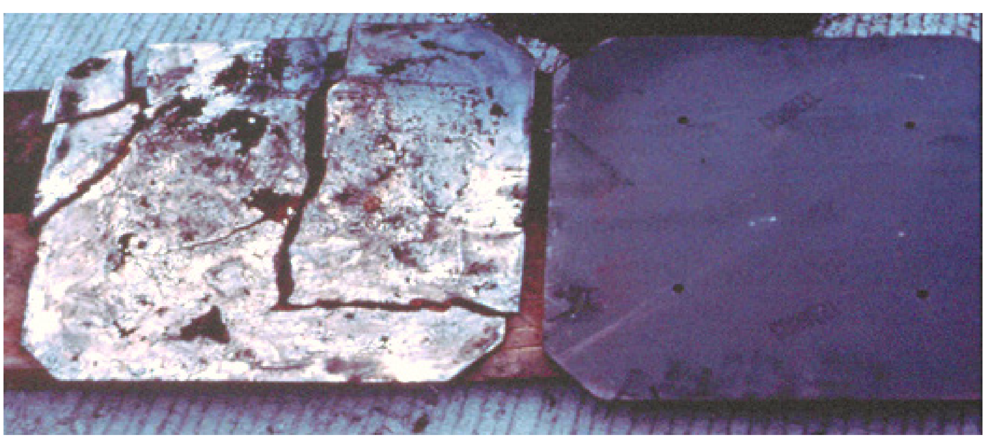


Figure 18-4 Damaged aluminum and micarta hammer cushion.

4. The pile cushion should be of correct type material and thickness for concrete piles.

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. Pile cushions for concrete piles should have the required thickness determined from a wave equation analysis but not less than 4 inches. A new plywood, hardwood, or composite wood pile cushion, which is not water soaked, should be used for every pile. In Figure 18-5, a new, 22 inch thick plywood pile cushion is being inserted into a helmet. The helmet with pile cushion inserted is shown in Figure 18-6. Note that after cushion insertion, minimal depth remains in the helmet to accommodate and adequately restrain the pile head.

Figure 18-5 New pile cushion being inserted into helmet.

Figure 18-6 Helmet with new pile cushion installed.

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.

5. Predrilling, jetting or spudding equipment, if specified or permitted, should be available for use and meet the requirements.

The depth of predrilling, jetting or spudding should be very carefully controlled so that it does not exceed the allowable limits. Predrilling, jetting, or spudding below the allowed depths will generally result in a reduced nominal geotechnical resistance, and the pile acceptance may become questionable. Additional details on predrilling, jetting, and spudding equipment are presented in Chapter 15.

6. The lead system being used must conform to the requirements, if any, in the specifications. Lead system details are discussed in detail in Chapter 15.

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 blow counts are 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 many 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 15-50 and obtain prior approval from the 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.

18.7 INSPECTION OF DRIVING EQUIPMENT DURING INSTALLATION

The main purpose of construction monitoring and inspection is to assure that piles are installed so that they meet the driving criteria and are undamaged. Driving criteria are often defined, in part, by a minimum pile penetration resistance or blow count that is measured in blows per foot or blows per inch. Driving criteria assure that piles have the required nominal resistance. The blow count, however, is dependent upon the performance of the pile driving hammer. The blow count will generally be lower when the hammer imparts higher energy and force to the pile, while the blow count will be higher if the hammer imparts lower energy and force to the pile. High blow counts can be due either to soil resistance or to a poorly performing hammer. Thus, the inspector must evaluate if the hammer is performing properly to assure that the driving criteria has been met and therefore the nominal resistance is achieved.

Each hammer has its own operating characteristics; the inspector should not blindly assume that the hammer on the project is in good working condition. Two different types of hammers with identical manufacturer's rated energy will not drive the same pile in the same soil with the same blow count. 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, ring condition, 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 15. The following checklists briefly summarize key hammer inspection issues.

18.7.1 Drop Hammers

a. Determine/confirm the ram weight. Ram weight can be calculated from the ram volume and steel density of 492 lb/ft^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.

18.7.2 Single Acting Air/Steam Hammers

a. Determine/confirm the ram weight. Ram weight can be calculated from the ram volume and steel density of 492 lb/ft^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 18-7 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.

Figure 18-7 Air compressor display panel.

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 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 blow count 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 10 blows per inch 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 18-2.

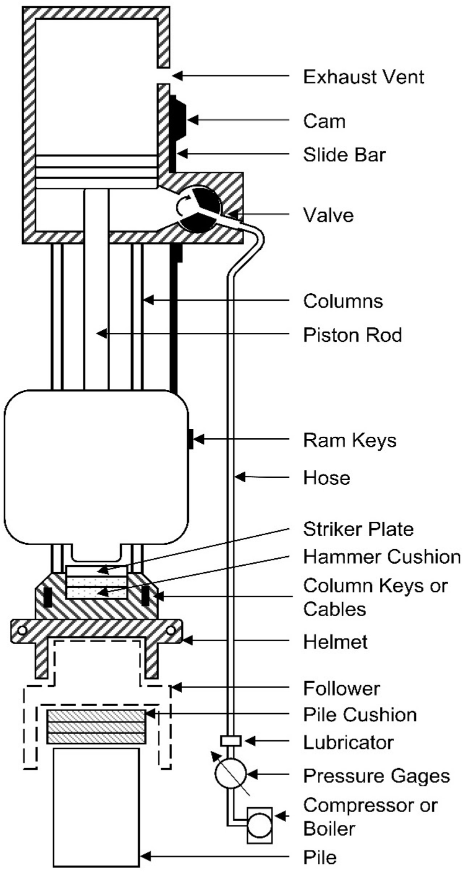
An inspection form for single and differential acting air/steam hammers is provided in Figure 18-8. The primary feature of this form is the three column area in the middle of the form. The left column illustrates the key objects of the driving system. The middle column contains the manufacturer's requirements 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.

Table 18-2 Common Problems and Indicators for Air/Steam Hammers
(after 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.

Project/Pile:	____________	Hammer Name	____________
Date:	____________	Serial No.	____________
Conditions:	____________		

Object	Requirements	Observations
	Slide Bars/ Cams Greased? Tight?	Yes / No Remarks ____________
	Columns Greased?	Yes / No
	Ram Keys Tight?	Yes / No
	Hose	I.D. Size ______ Length ______ Leaks? ______ Obstructions? ______
	Striker Plate	t= ______ D= ______
	Hammer Cushion	t= ______ D= ______ Material ______ How long in use? ______
	Column Keys or Cables Tight?	Yes / No
	Helmet	Type or Weight? ______
	Follower?	Yes / No Type? ______
	Pile Cushion	Material ______ t= ______ Size ______ How long in use? ______
	Lubricator Filled	Yes / No
	Pressure at Hammer ______ psi	Measured ______ psi at ______ feet from Hammer
	Fluctuating During Driving?	Yes / No How much? ______
	Check Compressor and Boiler?	Size ______ ft^3/min Make ______
	Pile	Material ______ Length ______ Size ______ Batter ______

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/ft ______
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
Piston Rod Lubricated		______
Exhaust Description		Freezing? Condensing? Lubricant Apparent?

Figure 18-8 Inspector’s form for single and differential acting air/steam hammers.

18.7.3 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 492 lbs/ft^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.

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 penetration 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 10 blows per inch 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.

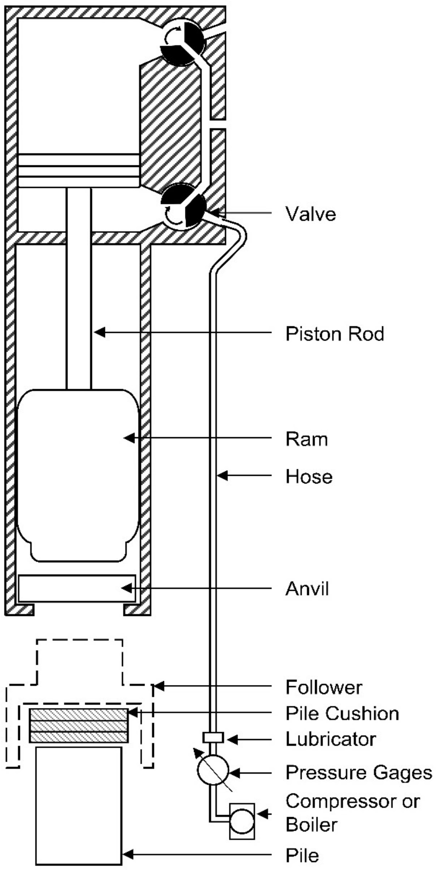
m. Common problems and problem indicators for air/steam hammers are summarized in Table 18-2.

An inspection form for enclosed double acting air/steam hammers is provided in Figure 18-9. 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

Project/Pile: ____________________ Hammer Name ____________________

Date: ____________________ Serial No. ____________________

Conditions: ____________________

Object	Requirements	Observations
	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? ________
	Hose	I.D. Size ______ Length ______ Leaks? Obstructions?
	Lubricator Filled	Yes / No
	Pressure at Hammer ________ psi	Measured ________ psi at ________ feet from Hammer
	Fluctuating During Driving?	Yes / No How much? ______ ________
	Check Compressor and Boiler?	Size ________ ft^3/min Make ________
	Pile	Material ________ Length ______ Size ______ Batter ________

Manufacturer's Hammer Data		Observation when Bearing is Confirmed	
Ram Weight	________	Full Ram Stroke	Yes/No ________ %
Max Stroke	________	Blows/min ________	Blows/ft ________
Rated Energy	________	High Pile Rebound? Yes/No	Pile Whipping? Yes/No
Blows/min in Hard Driving	________	Pile- Hammer Alignment	Front/Back ____ Sides ____
		Crane Size and Make	________
Attached Saximeter Printout		Lead Type	________
		Hammer Lead Guides Lubricated	Yes/No
		Piston Rod Lubricated	________
		Exhaust Description	Freezing? Condensing? Lubricant Apparent?

Figure 18-9 Inspector’s form for enclosed double acting air/steam hammers.

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.

18.7.4 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.

b. Make sure <u>all</u> exhaust ports are open with all plugs removed.

c. Inspect the recoil dampener for condition and thickness. If this is excessively worn or of an improper thickness (consult manufacturer) it should be replaced. If the recoil dampener is too thin, the stroke will be reduced. Conversely, if it is too thick, or if cylinder does not rest on the 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 ram stroke height, h, can also be calculated from this formula using the number of blows per minute (bpm) recorded:

$$\mathrm{h} = 4.01\left(\frac{60}{\mathrm{bpm}}\right)^2 - 0.3 \qquad \text{Eq. 18-1}$$

Where:

h = ram stroke (feet).
bpm = blow per minute (dimensionless).

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 blow count 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 18-10(a), others are continuously variable as shown in Figure 18-10(b), and some use a pressure pump as shown in Figure 18-11. 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.

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 pre-ignition. 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 pre-ignition occurs, less energy is transferred and the blow count rises, giving a false indication of high nominal resistance. If piles driven with a cold hammer drive deeper or with less hammer blows, or if the blow count decreases after short breaks, pre-ignition could be the cause and should be investigated. Dynamic testing is the preferable method to check for pre-ignition.

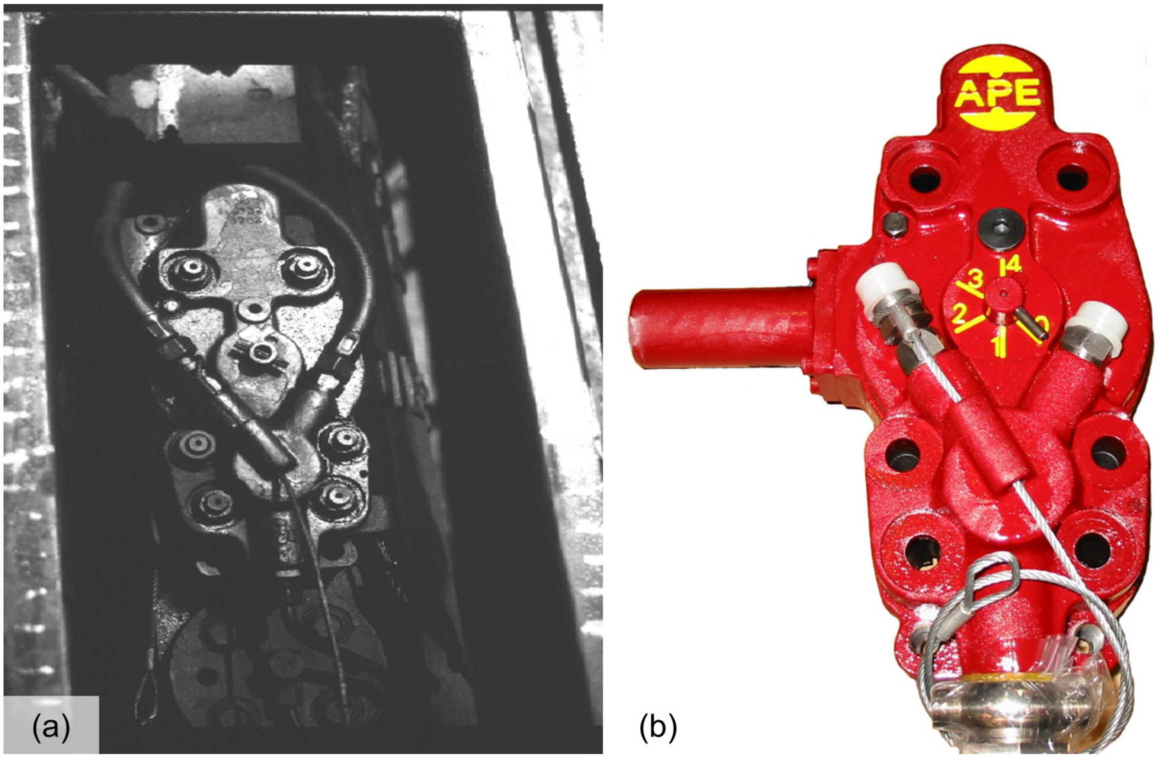


Figure 18-10 Fuel Pumps: (a) fixed four step pump and (b) variable fuel pump.


Figure 18-11 Hydraulic pump for fuel pump adjustments.

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 10 blows per inch 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 18-3.

An inspection form for single acting diesel hammers is provided in Figure 18-12. 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.

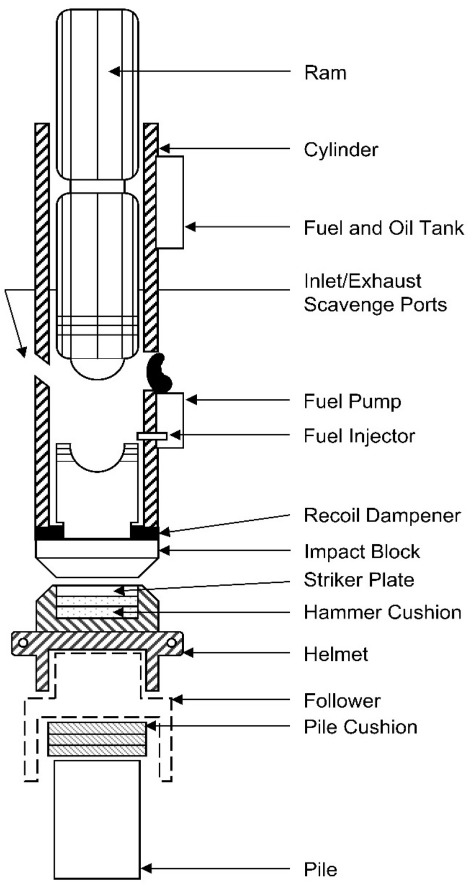
The bottom portion of the hammer inspection form contains an area where observations at final driving should be recorded. This information may be particularly interesting to an engineer who has performed a wave equation analysis as the actual situation can then be compared to the analyzed one. Therefore, it is recommended that a copy of the completed hammer inspection form be provided to appropriate design and construction personnel.

Project/Pile: ____________________ Hammer Name ____________________

Date: ____________________ Serial No. ____________________

Conditions: ____________________

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 ft-kips	Rated Stroke, feet
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 18-12 Inspector’s form for single acting diesel hammers.

Table 18-3 Common Problems and Indicators for Single Acting Diesel Hammers (after 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.

18.7.5 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 all 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 18-13. 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 penetration resistance increases, the stroke and bounce chamber pressure should also increase. If the ram fails to achieve the correct stroke or bounce chamber pressure (part of the driving criteria from a wave equation analysis) at final driving, 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.

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.

Figure 18-13 Typical external bounce chamber pressure gauge.

j. Watch for pre-ignition. When a hammer preignites, the fuel burns before impact requiring extra energy to compress the gas and reducing energy transferred to the pile. When pre-ignition occurs, the blow count increases giving a false indication of high nominal resistance. 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 blow count decreases after short breaks, investigate for pre-ignition, 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 10 blows per inch 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 18-4.

An inspection form for double acting diesel hammers is provided in Figure 18-14. 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.

Table 18-4 Common Problems and Indicators for Double Acting Diesel Hammers (after Williams Earth Sciences 1995)

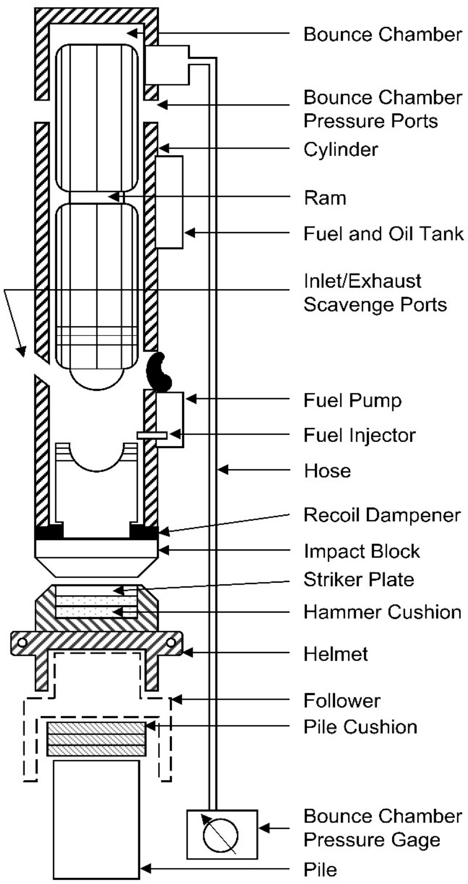
Common Problems	Indicators
Water in fuel.	Hollow sound, white smoke.
Fuel lines clogged.	No smoke or little gray smoke.
Fuel pump malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Fuel injectors malfunctioning.	Inconsistent ram strokes, little gray smoke or black smoke.
Oil 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.

Project/Pile: ______________________ Hammer Name ______________________

Date: ______________________ Serial No. ______________________

Conditions: ______________________

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
Bounce Chamber Hose	Length ________
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 ________

Max Stroke ________

Rated Energy ________

Bounce Chamber Pressure, psi	Rated Energy ft-kips

Attached Saximeter Printout

Observation when Bearing is Confirmed

Bouce Chamber Pressure ________

Cylinder Lift-off Type 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 18-14 Inspector’s form for double acting diesel hammers.

18.7.6 Single Acting Hydraulic Hammers

a. Determine/confirm the ram weight. If necessary, the ram weight can be calculated from the ram volume and steel density of 492 lbs/ft^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.

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. Allow the hammer to warm up before operation, and do not turn off power pack immediately after driving.

f. For single acting 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.

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 penetration 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 10 blows per inch 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.

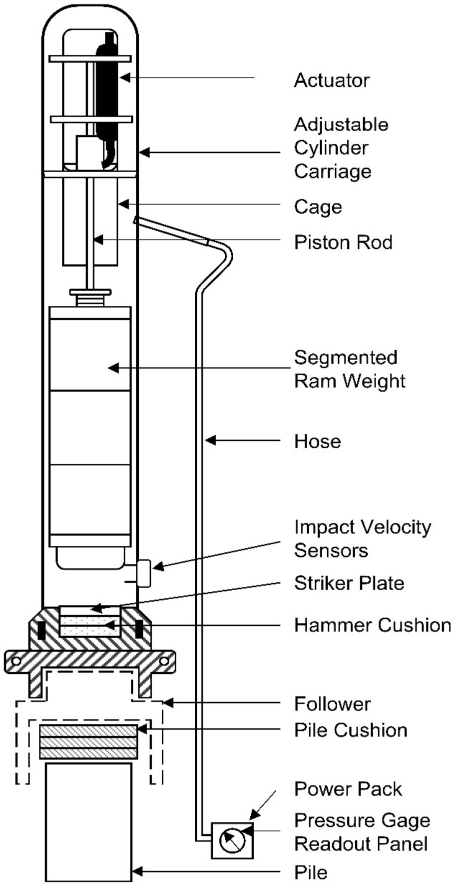
n. Common problems and problem indicators for hydraulic hammers are summarized in Table 18-5.

Table 18-5 Common Problems and Indicators for Single Acting Hydraulic Hammers (after 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.

Project/Pile: ____________________ Hammer Name ____________________
Date: ____________________ Serial No. ____________________
Conditions: ____________________

Object



Requirements	Observations
Ram Visible	Yes / No Observed Ram Stoke ______ ft
Ram Downward Pressure Provided?	Yes / No Hyd. Pressure Rated ______ psi Hyd. Pressure Actual ______ psi
Impact Velocity Measurement? If No, then	Yes / No Freefall? Yes / No Observed Fall Height ______ ft Pressure under ram during fall ______ Preadmission Possible? Yes / No
Striker Plate	t= D=
Hammer Cushion	t= ______ D= ______ Material ______ How long in use? ______
Helmet	Type or Weight? ______
Power Pack	Make ______ Model ______
Pressure Gage?	Yes / No Reading ______
Computer Readout?	Yes / No Reading ______
Follower	Yes / No Type? ______
Pile Cushion	Material ______ t= ______ Size ______ How long in use? ______
Pile	Material ______ Length ______ Size ______ Batter ______

Manufacturer's Hammer Data

Ram Weight ______
Max Stroke ______
Min Stroke ______
Max Energy ______
Min Energy ______

Attached Saximeter Printout

Observation when Bearing is Confirmed

Hammer Uplifting? Yes/No
Reduced Pressure? Yes/No ______ %
Blows/min ______ Blows/ft ______
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

Figure 18-15 Inspector’s form for single acting hydraulic hammers.

18.7.7 Double Acting Hydraulic Hammers

a. Determine/confirm the ram weight. There may also be identifying labels as to hammer make, model, and serial number which should be recorded.

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. Allow the hammer to warm up before operation, and do not turn off power pack immediately after driving.

e. Most double acting 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 blow count. 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 mounted on the power pack is presented in Figure 18-16. Hand held displays are also available on some hammers.

f. Most double acting hydraulic hammers are fully enclosed and therefore do not have observable rams. On these hammers it is impossible to measure the stroke being attained and confirm that it meets specification. Properly working energy readout devices are therefore mandatory for inspection.

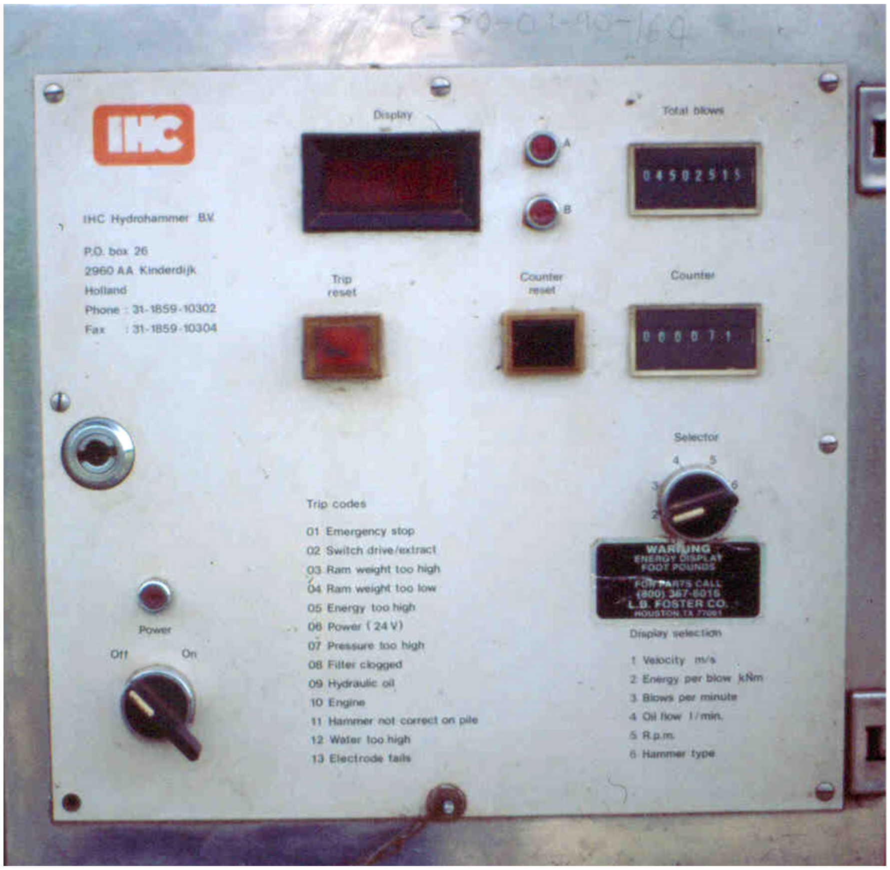


Figure 18-16 IHC hydraulic hammer read-out panel.

g. Most double acting hydraulic hammers are steel ram on steel anvil impact and do not use a striker plate or hammer cushion. One manufacturer uses an aluminum stack for a hammer cushion. Double acting hydraulic hammer models designed for use on concrete piles have a synthetic anvil block instead of a steel one. Document the anvil or cushioning mechanism, dimensions used in the double acting hammer model.

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 penetration 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.

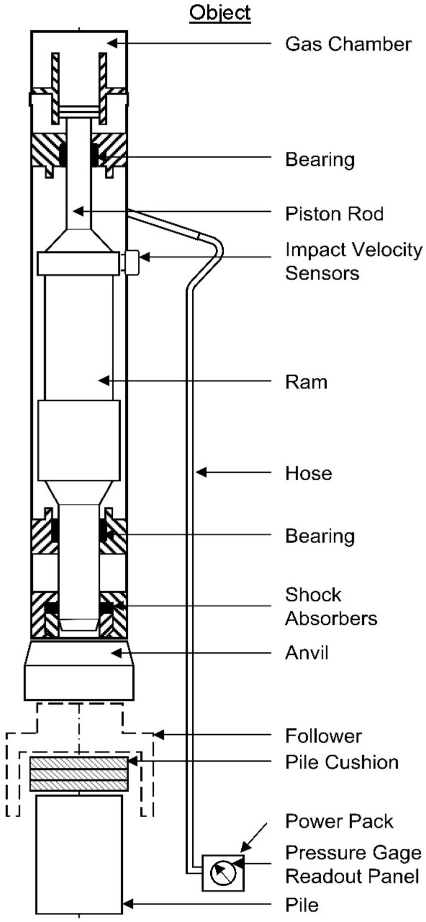
m. Some manufacturers void their warranty if the hammer is consistently operated above 10 blows per inch of penetration beyond short periods such as those required when toe bearing piles are driven to rock. Therefore, in prolonged hard driving situations, it may be more desirable to use a larger hammer or stiffer pile section.

n. Common problems and problem indicators for double acting hydraulic hammers with nitrogen caps are summarized in Table 18-6.

Table 18-6 Common Problems and Indicators for Double Acting Hydraulic Hammers

Common Problems	Indicators
Hydraulic hoses crossed.	Jumping hydraulic hoses.
Incorrect pressure in accumulators.	Jumping hydraulic hoses.
Cap pressure too high.	Hammer jumping on pile.
Low oil supply.	Erratic hammer operation.
Pressure or return valve malfunctioning.	Erratic or no hammer operation.
Hammer inoperable.	Control cable not connected.
Hammer inoperable.	Bulb A or B not lit on control panel.
Hoses getting caught in leads.	Visually evident.
Fittings leaking.	Hydraulic fluid dripping.
Electrical connections.	Erratic performance.
Sensors.	Erratic performance.

Project/Pile: ____________________ Hammer Name ____________________
Date: ____________________ Serial No. ____________________
Conditions: ____________________



Requirements	Observations
Ram Visible	Yes / No Observed Ram Stoke ______ ft
Ram Downward Pressure	Hyd. Pressure Rated ______ psi Hyd. Pressure Actual ______ psi
Impact Velocity Measurement	Measured Impact Velocity ______ ft/s Fall ______ ft
Anvil	Type or Weight? ______
Power Pack	Make ______ Model ______
Pressure Gage?	Yes / No Reading ______
Follower	Yes / No Type? ______
Pile Cushion	Material ______ t= ______ Size ______ How long in use? ______
Pile	Material ______ Length ______ Size ______ Batter ______

Manufacturer's Hammer Data	Observation when Bearing is Confirmed
Ram Weight ______	Hammer Uplifting? Yes/No
Max Stroke ______	Reduced Pressure? Yes/No ______ %
Min Stroke ______	Blows/min ______ Blows/ft ______
Max Energy ______	High Pile Rebound? Yes/No
Min Energy ______	Pile Whipping? Yes/No
	Pile- Hammer Alignment Front/Back ___ Sides ___
Attached Saximeter Printout	Crane Size and Make ______
	Lead Type ______
	Hammer Lead Guides Lubricated Yes/No

Figure 18-17 Inspector’s form for hydraulic hammers.

18.7.8 Resonant 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. Resonant 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 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 the resonant 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 may or may not be used.

18.7.9 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 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 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.

18.8 INSPECTION OF TEST OR INDICATOR PILES

Most pile foundation projects required verification of the foundation design and nominal resistance through the testing of some selected piles. The size of the foundation and relative costs of testing often dictate the type and number, if any, of verification tests performed. 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 low resistance factors and greater pile lengths. On these projects, test piles for verification testing are frequently not required. All piles are then production piles, and the entire pile foundation is usually installed in one or two days. Information on the piles, hammers, and other observations are recorded by the inspector and 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 level of verification testing is specified. Small to mid-size projects may have only a single static test (Chapter 9) on one pile at a specific depth, or there may be a few dynamic test piles (Chapter 10). The dynamic tests may include either testing during driving to assess hammer performance and driving stresses, or testing during restrike to assess nominal resistance, or both. The static or dynamic tests should be performed and reviewed by personnel having appropriate knowledge of the test method and proper procedures. 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 as conditions warrant. The goal is to determine driving criteria which will lead to a safe and 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. 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 nominal resistance 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 appropriate agency, design, or construction personnel.

The answers to the following questions should be known before driving test piles. It is often beneficial to have a preconstruction pile driving meeting between the contractor, project administrator, and the inspector to clarify these items.

1. Who determines test pile locations?

2. Who determines the test pile driving criteria?

3. Who stops the driving when the driving criterion 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. What documentation is required from test pile installations (test pile driving record, dynamic test results, static test report) and who produces what documentation?

11. Who produces the production pile driving criteria and how quickly will it be available?

12. Who has authority to allow production pile installation to begin?

13. When is the authorization to proceed to production pile driving given relative to test pile driving?

14. Can production piles (initial pile section or entire piles) be driven in advance of the production pile driving criteria and if so at whose risk?

18.9 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 pile 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 authorities in a timely manner.

The following items should be checked frequently (e.g. for each production pile):

1. Is the pile the specified 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. Have all pipe piles 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?

Previous sections of this chapter provide material which relate to inspection of production piles and offer detailed answers to the 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 using Equation 18-1. Check and record the bounce chamber pressure for double acting diesel hammers. The stroke of most single acting hydraulic hammers can be observed. Record the energy from the built-in energy monitor in addition to hammer stroke for each pile. Double hydraulic hammers must have a built-in energy monitor, 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 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 pile penetration resistance or blow count for nominal resistance verification should only be determined following the first 100 blows after cushion replacement. A new pile cushion reduces energy transfer and therefore produces an inflated blow count compared to a used cushion.

Inspection of pile splices is important to assure 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 remain 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 blow counts, above 10 blows per inch, 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 high strain dynamic pile testing, or for concrete piles, low strain integrity testing methods.   These methods are discussed in Chapter 10. Alternatively, the pile could be replaced or repaired, if possible.

Check for water leakage and soil inflow into 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. A mirror can also be used to reflect sunlight inside a pipe pile for internal inspection.

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 18-18 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).

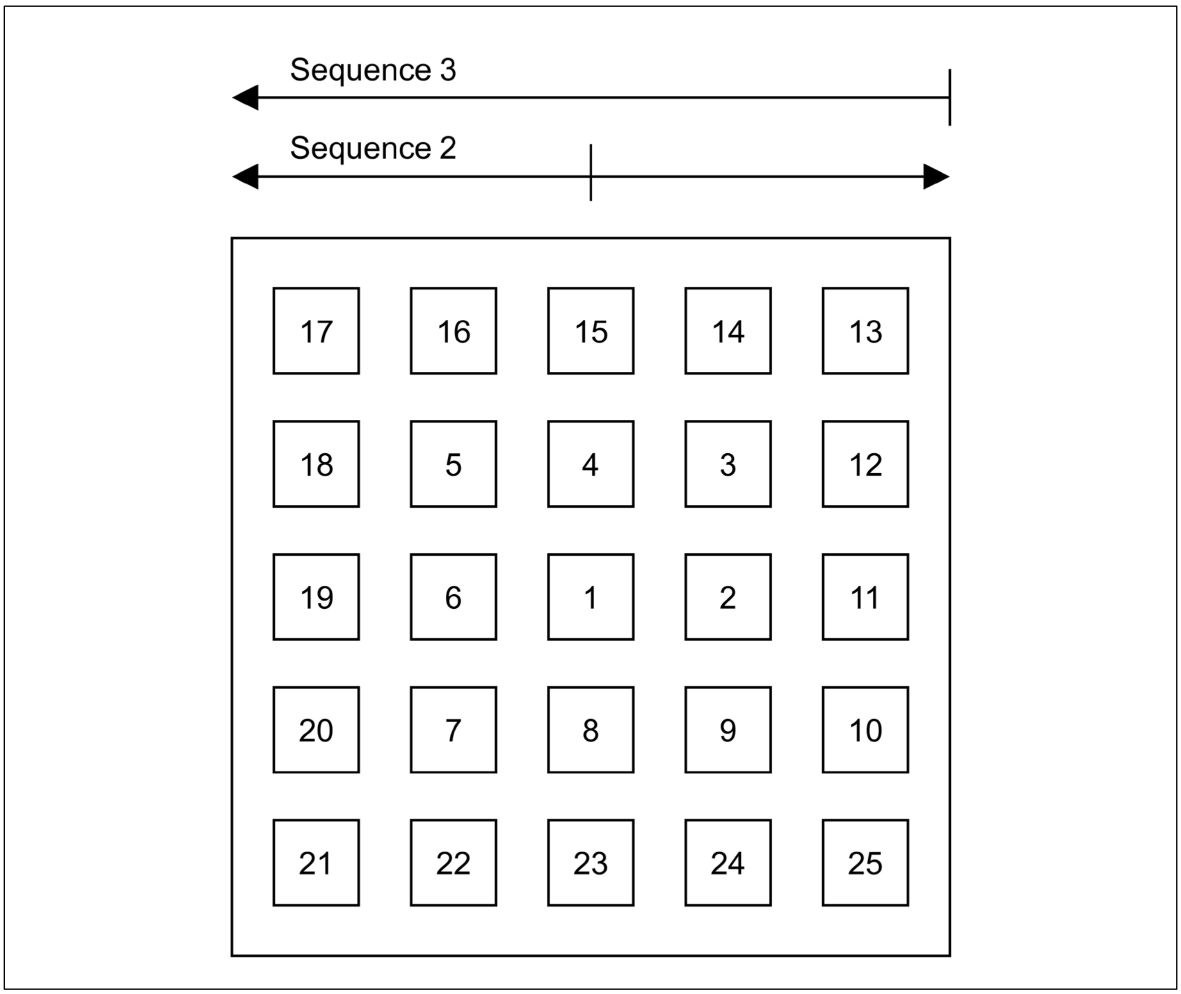


Figure 18-18 Driving sequence of displacement pile groups (after Passe 1994).

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 and prevent those piles from meeting minimum pile penetration depths for scour.

When driving close to an existing structure or utility, it is generally preferable to drive the piles nearest the existing structure or utility first and work away. For example, if a structure was located on the right side of the pile group shown in Figure 18-18, 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 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 1300 feet away from driving. Prior to the start of construction, a survey of structures and utilities within 400 feet of pile driving activities is often performed to document their existing condition. Some specifications tie the preconstruction survey limits to the hammer energy and require a preconstruction survey of all structures located within a distance in feet of 0.25 times the square root of the hammer energy in foot-pounds. The preconstruction survey generally consists of photographing or videotaping existing damage, as well as affixing crack gages to existing cracks in some cases.

Woods (1997) 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 penetration resistance for each 1 inch interval during the restrike.

A summary of common pile installation problems and possible solutions is presented in Table 18-7.

Table 18-7 Common Pile Installation Problems and Possible Solutions

Common Problems	Possible Solutions
Piles encountering refusal blow count above minimum pile penetration requirements.	Have wave equation analysis performed and check that pile has sufficient drivability 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 pre-ignition, 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 nominal resistance 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 nominal resistance based on restrike blow count is still low, check drive system performance and restrike pile with dynamic measurements. If drive system performance is as assumed and restrike nominal resistance 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 18-7 Common Pile Installation Problems and Possible Solutions (Continued)

Common Problems	Possible Solutions
Abrupt change or decrease in 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 nominal resistance 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.
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 18-7 Common Pile Installation Problems and Possible Solutions (Continued)

Common Problems	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 10 feet 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 18-7 Common Pile Installation Problems and Possible Solutions (Continued)

Common Problems	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. Nominal resistance 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 material stress limits. 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 18-7 Common Pile Installation Problems and Possible Solutions (Continued)

Common Problems	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 nominal resistance. If the calculated stress is high at high blow counts, a different hammer or pile type may be required. Nominal resistance 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 nominal resistance. Nominal resistance determination should not be made using blow counts obtained when driving with a mushroomed pile head.

18.10 DRIVING RECORDS AND REPORTS

Pile driving records vary depending upon the organization performing the monitoring or inspection service. A typical pile driving record is presented in Figure 18-19. 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.

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. Penetration resistance data in blows per foot with the final 1 foot normally recorded in blows per inch.

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 cannot 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 foundation cost 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 18-20 shows an example of a daily inspection report.

PROJECT NO.: ________________ DATE: ________________

JOB LOCATION: ________________ BENT / PIER NO.: ________________ PILE NO.: ________________

PILE TYPE: ________________ LENGTH: ________________ COATING: ________________ NOMINAL RESISTANCE: ________________

HAMMER: ________________ RATED ENERGY: ________________ OPERATING RATE: ________________ HELMET WT.: ________________

HAMMER CUSHION TYPE: ________________ t= ________________ PILE CUSHION TYPE: ________________ t= ________________

REF. ELEV.: ________________ MIN TOE ELEV.: ________________ FINAL TOE ELEV.: ________________ CUTOFF ELEV.: ________________

WEATHER: ________________ TEMP.: ________________ START TIME: ________________ END TIME: ________________

FEET	BLOWS	STROKE / PRESSURE	REMARKS	FEET	BLOWS	STROKE / PRESSURE	REMARKS
0 to 1				35 to 36			
1 to 2				36 to 37			
2 to 3				37 to 38			
3 to 4				38 to 39			
4 to 5				39 to 40			
5 to 6				40 to 41			
6 to 7				41 to 42			
7 to 8				42 to 43			
8 to 9				43 to 44			
9 to 10				44 to 45			
10 to 11				45 to 46			
11 to 12				46 to 47			
12 to 13				47 to 48			
13 to 14				48 to 49			
14 to 15				49 to 50			
15 to 16				50 to 51			
16 to 17				51 to 52			
17 to 18				52 to 53			
18 to 19				53 to 54			
19 to 20				54 to 55			
20 to 21				55 to 56			
21 to 22				56 to 57			
22 to 23				57 to 58			
23 to 24				58 to 59			
24 to 25				59 to 60			
25 to 26				60 to 61			
26 to 27				61 to 62			
27 to 28				62 to 63			
28 to 29				63 to 64			
29 to 30				64 to 65			
30 to 31				65 to 66			
31 to 32				66 to 67			
32 to 33				67 to 68			
33 to 34				68 to 69			
34 to 35				69 to 70			

PILE SUPPLIER: ________________ PILE CAST OR ROLLING DATE: ________________

PILE TOE ATTACHMENTS: ________________ FINAL PILE HEAD CONDITION: ________________

FINAL ALIGNMENT: ________________ FINAL PLUMBNESS: ________________ INTERNAL INSPECTION: ________________

INSPECTORS SIGNATURE: ________________

Figure 18-19 Pile driving log.

Daily Inspection Report

Project No.: ____________________

Date: ____________________

Project: ____________________

Weather Conditions: ____________________

Contractor: ____________________

Contractor's Presonnel Present: ____________________

____

____

Equipment Working: ____________________

____

Description of Work: ____________________

____

____

____

Special Person's Visiting Job: ____________________

____

Test Performed: ____________________

____

Special Coments: ____________________

____

____

____

____

Inspector's Signature: ____________________

Figure 18-20 Daily inspection report.

18.11 SAFETY

Pile driving involves the use of heavy equipment and heavy loads. The pile driving inspector should be cognizant of these activities and position his or herself accordingly. One of the more dangerous operations during pile driving is the lifting of the pile and the positioning of it under the hammer for driving. The inspector should remember that a 100 feet pile can fall 100 feet from the pile location during positioning. It is better to have a planned escape route prior to pile positioning rather than attempt to quickly determine one should difficulties arise during the pile lifting and positioning process.

The area beneath a suspended load should be avoided. If the hoisting device fails or slips, serious injury could occur. The inspector should also avoid the area behind the crane and be cognizant whenever the crane is moving or swinging.

The inspector should select a position to maintain the pile driving record that is a sufficient distance away from the pile location during driving. The area immediately in front of the pile should be avoided. Heavy pieces sometimes fall from a pile hammer or helmet during operation that could cause serious injury if the inspector were positioned under or near the hammer. All pile types can be also damaged during driving. Concrete and timber piles can break suddenly, and long steel piles can buckle. A sudden pile break or buckling can make the area around the pile location quite dangerous due to the broken or buckled section. A sudden loss of resistance beneath an operating pile hammer can also overload the hammer line causing it to break and the hammer to fall. Damage to the head of a concrete pile during driving can also result in heavy concrete pieces falling due to hammer-pile alignment problems or due to pile cushion deterioration. Standing beneath the hammer and monitoring the final pile penetration resistance by placing marks on the pile every 10 or 20 hammer blows should be avoided for the above reasons. The final blow count can be determined instead by marking the pile prior to driving with 1 inch marks over the anticipated final penetration depth.

Safety devices such as a hard hat, ear protection, steel toed work boots, eye protection, safety vest, fall protection harness, and life jacket should be worn as job conditions dictate. Additional site specific, state, or federal safety rules may also apply and these should be reviewed.

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Appendix A

LIST OF FHWA/ NHI RESOURCES RELEVANT TO DEEP FOUNDATIONS

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Abu-Hejleh, N., Kramer, W.M., Mohamed, K., Long, J.H., and Zaheer, M.A. (2013). Implementation of AASHTO LRFD Design Specifications for Driven Piles, FHWA-RC-13-001. U.S. Dept. of Transportation, Federal Highway Administration, Washington, D.C., 71 p.

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Appendix B

LIST OF ASTM AND AASHTO PILE DESIGN AND TESTING SPECIFICATIONS

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Appendix C

PILE HAMMER INFORMATION

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
1	DELMAG	D 5	OED	10.51	1.10	9.55
2	DELMAG	D 8-22	OED	20.10	1.76	11.42
3	DELMAG	D 12	OED	22.61	2.75	8.22
4	DELMAG	D 15	OED	27.09	3.30	8.21
5	DELMAG	D 16-32	OED	40.20	3.52	11.42
6	DELMAG	D 22	OED	40.61	4.91	8.27
7	DELMAG	D 22-02	OED	48.50	4.85	10.00
8	DELMAG	D 22-13	OED	48.50	4.85	10.00
9	DELMAG	D 22-23	OED	51.22	4.85	10.56
10	DELMAG	D 25-32	OED	66.34	5.51	12.04
11	DELMAG	D 30	OED	59.73	6.60	9.05
12	DELMAG	D 30-02	OED	66.20	6.60	10.03
13	DELMAG	D 30-13	OED	66.20	6.60	10.03
14	DELMAG	D 30-23	OED	73.79	6.60	11.18
15	DELMAG	D 30-32	OED	75.44	6.60	11.43
16	DELMAG	D 36	OED	83.82	7.93	10.57
17	DELMAG	D 36-02	OED	83.82	7.93	10.57
18	DELMAG	D 36-13	OED	83.82	7.93	10.57
19	DELMAG	D 36-23	OED	88.50	7.93	11.16
20	DELMAG	D 36-32	OED	90.56	7.93	11.42
21	DELMAG	D 44	OED	90.16	9.50	9.49
22	DELMAG	D 46	OED	107.08	10.14	10.56
23	DELMAG	D 46-02	OED	107.08	10.14	10.56
24	DELMAG	D 46-13	OED	96.53	10.14	9.52
25	DELMAG	D 46-23	OED	107.08	10.14	10.56
26	DELMAG	D 46-32	OED	122.19	10.14	12.05
27	DELMAG	D 55	OED	125.00	11.86	10.54
28	DELMAG	D 62-02	OED	152.45	13.66	11.16
29	DELMAG	D 62-12	OED	152.45	13.66	11.16
30	DELMAG	D 62-22	OED	164.60	13.66	12.05
31	DELMAG	D 80-12	OED	186.24	17.62	10.57
32	DELMAG	D 80-23	OED	212.50	17.62	12.06
33	DELMAG	D100-13	OED	265.68	22.07	12.04
35	DELMAG	D 19-52	OED	43.20	4.00	10.80
36	DELMAG	D 6-32	OED	13.52	1.32	10.23

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
37	DELMAG	D 12-32	OED	31.33	2.82	11.11
38	DELMAG	D 12-42	OED	33.30	2.82	11.81
39	DELMAG	D 14-42	OED	34.50	3.09	11.18
40	DELMAG	D 19-32	OED	42.44	4.00	10.61
41	DELMAG	D 19-42	OED	43.24	4.00	10.81
42	DELMAG	D200-42	OED	492.04	44.09	11.16
43	DELMAG	D120-42	OED	301.79	26.45	11.41
44	DELMAG	D150-42	OED	377.33	33.07	11.41
45	DELMAG	D125-42	OED	313.63	27.56	11.38
46	DELMAG	D 21-42	OED	55.75	4.63	12.04
47	DELMAG	D 5-42	OED	10.56	1.10	9.60
48	DELMAG	D160-32	OED	393.45	35.27	11.16
49	DELMAG	D260-32	OED	639.36	57.32	11.16
50	FEC	1200	OED	22.50	2.75	8.18
51	FEC	1500	OED	27.09	3.30	8.21
52	FEC	2500	OED	50.00	5.50	9.09
53	FEC	2800	OED	55.99	6.16	9.09
54	FEC	3000	OED	63.03	6.60	9.55
55	FEC	3400	OED	73.01	7.48	9.76
56	FEC	D-18	OED	39.70	3.97	10.00
61	MITSUBIS	M 14	OED	25.25	2.97	8.50
62	MITSUBIS	MH 15	OED	28.14	3.31	8.50
63	MITSUBIS	M 23	OED	43.01	5.06	8.50
64	MITSUBIS	MH 25	OED	46.84	5.51	8.50
65	MITSUBIS	M 33	OED	61.71	7.26	8.50
66	MITSUBIS	MH 35	OED	65.62	7.72	8.50
67	MITSUBIS	M 43	OED	80.41	9.46	8.50
68	MITSUBIS	MH 45	OED	85.43	10.05	8.50
70	MITSUBIS	MH 72B	OED	135.15	15.90	8.50
71	MITSUBIS	MH 80B	OED	149.60	17.60	8.50
81	LINKBELT	LB 180	CED	8.10	1.73	4.68
82	LINKBELT	LB 312	CED	15.02	3.86	3.89
83	LINKBELT	LB 440	CED	18.20	4.00	4.55
84	LINKBELT	LB 520	CED	26.31	5.07	5.19
85	LINKBELT	LB 660	CED	51.63	7.57	6.82
90	HITACHI	HNC65	ECH	56.42	14.33	3.94
91	HITACHI	HNC80	ECH	69.43	17.64	3.94
92	HITACHI	HNC100	ECH	86.79	22.05	3.94
93	HITACHI	HNC125	ECH	108.49	27.56	3.94
101	KOBE	K 13	OED	25.43	2.87	8.86
103	KOBE	K22-Est	OED	45.35	4.85	9.35
104	KOBE	K 25	OED	51.52	5.51	9.35
107	KOBE	K 35	OED	72.18	7.72	9.35

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
110	KOBE	K 45	OED	92.75	9.92	9.35
112	KOBE	KB 60	OED	130.18	13.23	9.84
113	KOBE	KB 80	OED	173.58	17.64	9.84
120	ICE	180	CED	8.13	1.73	4.70
121	ICE	422	CED	23.12	4.00	5.78
122	ICE	440	CED	18.56	4.00	4.64
123	ICE	520	CED	30.37	5.07	5.99
124	ICE	640	CED	40.62	6.00	6.77
125	ICE	660	CED	51.63	7.57	6.82
126	ICE	1070	CED	72.60	10.00	7.26
127	ICE	30-S	OED	22.50	3.00	7.50
128	ICE	40-S	OED	40.00	4.00	10.00
129	ICE	42-S	OED	42.00	4.09	10.27
130	ICE	60-S	OED	59.99	7.00	8.57
131	ICE	70-S	OED	70.00	7.00	10.00
132	ICE	80-S	OED	80.00	8.00	10.00
133	ICE	90-S	OED	90.00	9.00	10.00
134	ICE	100-S	OED	100.00	10.00	10.00
135	ICE	120-S	OED	120.00	12.00	10.00
136	ICE	200-S	OED	100.00	20.00	5.00
137	ICE	205-S	OED	170.00	20.00	8.50
139	ICE	32-S	OED	26.01	3.00	8.67
140	ICE	120S-15	OED	132.45	15.00	8.83
142	MKT	DE-20C	OED	20.00	2.00	10.00
143	MKT	DE-30C	OED	28.00	2.80	10.00
144	MKT	DE-33C	OED	33.00	3.30	10.00
145	MKT	DE333020	OED	40.00	4.00	10.00
146	MKT	DE 10	OED	8.80	1.10	8.00
147	MKT	DE 20	OED	16.00	2.00	8.00
148	MKT	DE 30	OED	22.40	2.80	8.00
149	MKT	DA35B SA	OED	23.80	2.80	8.50
150	MKT	DE 30B	OED	23.80	2.80	8.50
151	MKT	DA 35B	CED	21.00	2.80	7.50
152	MKT	DA 45	CED	30.72	4.00	7.68
153	MKT	DE 40	OED	32.00	4.00	8.00
154	MKT	DE 35	OED	35.00	3.50	10.00
155	MKT	DE 42	OED	42.00	4.20	10.00
157	MKT	DE 50C	OED	50.00	5.00	10.00
158	MKT	DE 70C	OED	70.00	7.00	10.00
159	MKT	DE 50B	OED	42.50	5.00	8.50
160	MKT	DA55B SA	OED	40.00	5.00	8.00
161	MKT	DA 55B	CED	38.20	5.00	7.64
162	MKT	DE 70B	OED	59.50	7.00	8.50

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
163	MKT	DE-50B	OED	50.00	5.00	10.00
164	MKT	DE-70B	OED	70.00	7.00	10.00
165	MKT	DE-110C	OED	110.00	11.00	10.00
166	MKT	DE-150C	OED	150.00	15.00	10.00
167	MKT	DA 35C	CED	21.00	2.80	7.50
168	MKT	DA 55C	CED	38.20	5.00	7.64
171	CONMACO	C 50	ECH	15.00	5.00	3.00
172	CONMACO	C 65	ECH	19.50	6.50	3.00
173	CONMACO	C 550	ECH	25.00	5.00	5.00
174	CONMACO	C 565	ECH	32.50	6.50	5.00
175	CONMACO	C 80	ECH	26.00	8.00	3.25
176	CONMACO	C 100	ECH	32.50	10.00	3.25
177	CONMACO	C 115	ECH	37.38	11.50	3.25
178	CONMACO	C 80E5	ECH	40.00	8.00	5.00
179	CONMACO	C 100E5	ECH	50.00	10.00	5.00
180	CONMACO	C 115E5	ECH	57.50	11.50	5.00
181	CONMACO	C 125E5	ECH	62.50	12.50	5.00
182	CONMACO	C 140	ECH	42.00	14.00	3.00
183	CONMACO	C 160	ECH	48.75	16.25	3.00
184	CONMACO	C 200	ECH	60.00	20.00	3.00
185	CONMACO	C 300	ECH	90.00	30.00	3.00
186	CONMACO	C 5200	ECH	100.00	20.00	5.00
187	CONMACO	C 5300	ECH	150.00	30.00	5.00
188	CONMACO	C 5450	ECH	225.00	45.00	5.00
189	CONMACO	C 5700	ECH	350.00	70.00	5.00
190	CONMACO	C 6850	ECH	510.00	85.00	6.00
191	CONMACO	C 160 **	ECH	51.78	17.26	3.00
192	CONMACO	C 50E5	ECH	25.00	5.00	5.00
193	CONMACO	C 65E5	ECH	32.50	6.50	5.00
194	CONMACO	C 200E5	ECH	100.00	20.00	5.00
195	CONMACO	C 300E5	ECH	150.00	30.00	5.00
196	CONMACO	C 1750	ECH	1050.00	175.00	6.00
204	VULCAN	VUL 01	ECH	15.00	5.00	3.00
205	VULCAN	VUL 02	ECH	7.26	3.00	2.42
206	VULCAN	VUL 06	ECH	19.50	6.50	3.00
207	VULCAN	VUL 08	ECH	26.00	8.00	3.25
208	VULCAN	VUL 010	ECH	32.50	10.00	3.25
209	VULCAN	VUL 012	ECH	39.00	12.00	3.25
210	VULCAN	VUL 014	ECH	42.00	14.00	3.00
211	VULCAN	VUL 016	ECH	48.75	16.25	3.00
212	VULCAN	VUL 020	ECH	60.00	20.00	3.00
213	VULCAN	VUL 030	ECH	90.00	30.00	3.00
214	VULCAN	VUL 040	ECH	120.00	40.00	3.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
215	VULCAN	VUL 060	ECH	180.00	60.00	3.00
220	VULCAN	VUL 30C	ECH	7.26	3.00	2.42
221	VULCAN	VUL 50C	ECH	15.10	5.00	3.02
222	VULCAN	VUL 65C	ECH	19.18	6.50	2.95
223	VULCAN	VUL 65CA	ECH	19.57	6.50	3.01
224	VULCAN	VUL 80C	ECH	24.48	8.00	3.06
225	VULCAN	VUL 85C	ECH	25.99	8.52	3.05
226	VULCAN	VUL 100C	ECH	32.90	10.00	3.29
227	VULCAN	VUL 140C	ECH	35.98	14.00	2.57
228	VULCAN	VUL 200C	ECH	50.20	20.00	2.51
229	VULCAN	VUL 400C	ECH	113.60	40.00	2.84
230	VULCAN	VUL 600C	ECH	179.16	60.00	2.99
231	VULCAN	VUL 320	ECH	60.00	20.00	3.00
232	VULCAN	VUL 330	ECH	90.00	30.00	3.00
233	VULCAN	VUL 340	ECH	120.00	40.00	3.00
234	VULCAN	VUL 360	ECH	180.00	60.00	3.00
235	VULCAN	VUL 505	ECH	25.00	5.00	5.00
236	VULCAN	VUL 506	ECH	32.50	6.50	5.00
237	VULCAN	VUL 508	ECH	40.00	8.00	5.00
238	VULCAN	VUL 510	ECH	50.00	10.00	5.00
239	VULCAN	VUL 512	ECH	60.00	12.00	5.00
240	VULCAN	VUL 520	ECH	100.00	20.00	5.00
241	VULCAN	VUL 530	ECH	150.00	30.00	5.00
242	VULCAN	VUL 540	ECH	200.00	40.90	4.89
243	VULCAN	VUL 560	ECH	300.00	62.50	4.80
245	VULCAN	VUL 3100	ECH	300.00	100.00	3.00
246	VULCAN	VUL 5100	ECH	500.00	100.00	5.00
247	VULCAN	VUL 5150	ECH	750.00	150.00	5.00
248	VULCAN	VUL 6300	ECH	1800.00	300.00	6.00
251	RAYMOND	R 1	ECH	15.00	5.00	3.00
252	RAYMOND	R 1S	ECH	19.50	6.50	3.00
253	RAYMOND	R 65C	ECH	19.50	6.50	3.00
254	RAYMOND	R 65CH	ECH	19.50	6.50	3.00
255	RAYMOND	R 0	ECH	24.38	7.50	3.25
256	RAYMOND	R 80C	ECH	24.48	8.00	3.06
257	RAYMOND	R 80CH	ECH	24.48	8.00	3.06
258	RAYMOND	R 2/0	ECH	32.50	10.00	3.25
259	RAYMOND	R 3/0	ECH	40.63	12.50	3.25
260	RAYMOND	R 150C	ECH	48.75	15.00	3.25
261	RAYMOND	R 4/0	ECH	48.75	15.00	3.25
262	RAYMOND	R 5/0	ECH	56.88	17.50	3.25
263	RAYMOND	R 30X	ECH	75.00	30.00	2.50
264	RAYMOND	R 8/0	ECH	81.25	25.00	3.25

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
265	RAYMOND	R 40X	ECH	100.00	40.00	2.50
266	RAYMOND	R 60X	ECH	150.00	60.00	2.50
270	MENCK	MHU 100C	ECH	73.71	11.10	6.64
271	MENCK	MH 68	ECH	49.18	7.72	6.37
272	MENCK	MH 96	ECH	69.43	11.02	6.30
273	MENCK	MH 145	ECH	104.80	16.53	6.34
274	MENCK	MHU 195	ECH	143.74	21.36	6.73
275	MENCK	MHU 220	ECH	162.17	24.84	6.53
276	MENCK	MHU 400	ECH	294.82	51.09	5.77
277	MENCK	MHU 600	ECH	442.28	75.52	5.86
278	MENCK	MHU 1000	ECH	737.38	126.98	5.81
279	MENCK	MHU 1700	ECH	1253.24	207.15	6.05
280	MENCK	MHU 2100	ECH	1548.29	257.18	6.02
281	MENCK	MHU 3000	ECH	2211.90	370.23	5.97
282	MENCK	MRBS 500	ECH	45.07	11.02	4.09
283	MENCK	MRBS 750	ECH	67.77	16.53	4.10
285	MENCK	MRBS 850	ECH	93.28	18.96	4.92
286	MENCK	MRBS1100	ECH	123.43	24.25	5.09
287	MENCK	MRBS1502	ECH	135.59	33.07	4.10
288	MENCK	MRBS1800	ECH	189.81	38.58	4.92
289	MENCK	MRBS2500	ECH	262.11	63.93	4.10
290	MENCK	MRBS2502	ECH	225.95	55.11	4.10
291	MENCK	MRBS2504	ECH	225.95	55.11	4.10
292	MENCK	MRBS3000	ECH	325.36	66.13	4.92
293	MENCK	MRBS3900	ECH	513.34	86.86	5.91
294	MENCK	MRBS4600	ECH	498.94	101.41	4.92
295	MENCK	MRBS5000	ECH	542.33	110.23	4.92
296	MENCK	MRBS6000	ECH	759.23	132.27	5.74
297	MENCK	MRBS7000	ECH	631.40	154.00	4.10
298	MENCK	MRBS8000	ECH	867.74	176.37	4.92
299	MENCK	MRBS8800	ECH	954.53	194.01	4.92
300	MENCK	MBS12500	ECH	1581.83	275.58	5.74
301	MKT	No. 5	ECH	1.00	0.20	5.00
302	MKT	No. 6	ECH	2.50	0.40	6.25
303	MKT	No. 7	ECH	4.15	0.80	5.19
304	MKT	9B3	ECH	8.75	1.60	5.47
305	MKT	10B3	ECH	13.11	3.00	4.37
306	MKT	C5-Air	ECH	14.20	5.00	2.84
307	MKT	C5-Steam	ECH	16.20	5.00	3.24
308	MKT	S-5	ECH	16.25	5.00	3.25
309	MKT	11B3	ECH	19.15	5.00	3.83
310	MKT	C826 Stm	ECH	24.40	8.00	3.05
311	MKT	C826 Air	ECH	21.20	8.00	2.65

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
312	MKT	S-8	ECH	26.00	8.00	3.25
313	MKT	MS-350	ECH	30.80	7.72	3.99
314	MKT	S 10	ECH	32.50	10.00	3.25
315	MKT	S 14	ECH	37.52	14.00	2.68
316	MKT	MS 500	ECH	44.00	11.00	4.00
317	MKT	S 20	ECH	60.00	20.00	3.00
318	IHC	S-30	ECH	21.70	3.53	6.15
319	IHC	S-40	ECH	28.93	4.85	5.97
320	IHC	S-35	ECH	25.53	6.63	3.85
321	IHC	S-70	ECH	51.25	7.73	6.63
322	IHC	S-90	ECH	65.90	9.94	6.63
323	IHC	S-120	ECH	89.37	13.48	6.63
324	IHC	S-150	ECH	110.06	16.60	6.63
325	IHC	S-200	ECH	145.64	22.00	6.62
326	IHC	S-280	ECH	205.31	30.06	6.83
327	IHC	S-400	ECH	292.60	44.20	6.62
328	IHC	S-500	ECH	366.09	55.30	6.62
329	IHC	S-600	ECH	443.54	67.00	6.62
330	IHC	S-900	ECH	658.36	99.45	6.62
331	IHC	S-1200	ECH	891.05	134.60	6.62
332	IHC	S-1800-L	ECH	1170.39	166.00	7.05
333	IHC	S-2300	ECH	1681.48	254.00	6.62
334	IHC	S-2000	ECH	1473.97	222.65	6.62
335	IHC	SC-30	ECH	21.81	3.76	5.80
336	IHC	SC-40	ECH	29.86	5.51	5.42
337	IHC	SC-50	ECH	36.82	7.29	5.05
338	IHC	SC-60	ECH	44.95	13.30	3.38
339	IHC	SC-75	ECH	54.80	12.15	4.51
340	IHC	SC-110	ECH	81.89	17.46	4.69
341	IHC	SC-150	ECH	109.35	24.30	4.50
342	IHC	SC-200	ECH	152.51	30.20	5.05
343	IHC	SC-250	ECH	179.80	37.26	4.83
344	IHC	S-750	ECH	550.79	83.11	6.63
345	IHC	S-800	ECH	589.97	88.15	6.69
346	IHC	S-1400	ECH	1033.84	147.94	6.99
347	IHC	S-1800	ECH	1340.21	195.64	6.85
348	IHC	S-2500	ECH	1843.16	275.80	6.68
349	HERA	1900	OED	44.41	4.19	10.60
350	HERA	1250	OED	24.85	2.76	9.02
351	HERA	1500	OED	29.81	3.31	9.02
352	HERA	2500	OED	49.70	5.51	9.02
353	HERA	2800	OED	55.70	6.18	9.02
354	HERA	3500	OED	69.59	7.72	9.02

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
355	HERA	5000	OED	99.45	11.03	9.02
356	HERA	5700	OED	113.38	12.57	9.02
357	HERA	6200	OED	123.30	13.67	9.02
358	HERA	7500	OED	149.19	16.54	9.02
359	HERA	8800	OED	174.99	19.40	9.02
360	ICE	I-12obs	OED	30.21	2.82	10.71
361	ICE	I-19obs	OED	43.24	4.02	10.77
362	ICE	I-30obs	OED	71.45	6.62	10.80
363	ICE	I-36obs	OED	90.68	7.94	11.42
364	ICE	I-46obs	OED	107.74	10.15	10.62
365	ICE	I-62obs	OED	164.98	14.60	11.30
366	ICE	I-80obs	OED	212.40	17.70	12.00
367	ICE	I-8v2obs	OED	17.60	1.76	10.00
368	ICE	I-100obs	OED	264.45	23.61	11.20
369	BSP	SL20	ECH	14.11	3.31	4.27
370	BSP	SL30	ECH	21.69	5.51	3.94
371	FAIRCHLD	F-45	ECH	45.00	15.00	3.00
372	FAIRCHLD	F-32	ECH	32.55	10.85	3.00
374	BSP	CX40	ECH	28.21	6.61	4.27
375	BSP	CX50	ECH	37.61	8.82	4.27
376	BSP	CX60	ECH	47.01	11.02	4.27
377	BSP	CX75	ECH	52.08	13.23	3.94
378	BSP	CX85	ECH	60.75	15.43	3.94
379	BSP	CX110	ECH	78.11	19.84	3.94
381	BSP	HH3	ECH	26.02	6.61	3.94
382	BSP	HH5	ECH	43.38	11.02	3.94
383	BSP	HH7	ECH	60.78	15.43	3.94
384	BSP	HH8	ECH	69.50	17.64	3.94
385	BSP	HH9	ECH	78.17	19.84	3.94
386	BSP	HH11-1.2	ECH	95.55	24.25	3.94
387	BSP	HH14-1.2	ECH	121.59	30.86	3.94
388	BSP	HH16-1.2	ECH	138.87	35.27	3.94
391	BSP	HA30	ECH	260.37	66.14	3.94
392	BSP	HA40	ECH	347.16	88.18	3.94
393	BSP	HH11-1.5	ECH	119.31	24.25	4.92
394	BSP	HH14-1.5	ECH	151.83	30.86	4.92
395	BSP	HH16-1.5	ECH	173.54	35.27	4.92
396	BSP	CG180	ECH	131.92	26.45	4.99
397	BSP	CG210	ECH	153.91	30.86	4.99
398	BSP	CG240	ECH	175.90	35.27	4.99
399	BSP	CG270	ECH	197.88	39.68	4.99
400	BSP	CG300	ECH	219.87	44.09	4.99
401	BERMINGH	B23	CED	22.99	2.80	8.21

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
402	BERMINGH	B200	OED	18.00	2.00	9.00
403	BERMINGH	B225	OED	29.25	3.00	9.75
404	BERMINGH	B300	OED	40.31	3.75	10.75
405	BERMINGH	B400	OED	53.75	5.00	10.75
406	BERMINGH	B-21	OED	53.25	4.63	11.50
410	BERMINGH	B300 M	OED	40.31	3.75	10.75
411	BERMINGH	B400 M	OED	53.75	5.00	10.75
412	BERMINGH	B400 4.8	OED	43.20	4.80	9.00
413	BERMINGH	B400 5.0	OED	45.00	5.00	9.00
414	BERMINGH	B23 5	CED	22.99	2.80	8.21
415	BERMINGH	B250 5	OED	26.25	2.50	10.50
416	BERMINGH	B3505	OED	47.20	4.00	11.80
417	BERMINGH	B4005	OED	59.00	5.00	11.80
418	BERMINGH	B4505	OED	77.88	6.60	11.80
419	BERMINGH	B5005	OED	92.04	7.80	11.80
420	BERMINGH	B5505	OED	108.56	9.20	11.80
421	BERMINGH	B550 C	OED	88.00	11.00	8.00
422	BERMINGH	B2005	OED	18.00	2.00	9.00
424	BERMINGH	B2505	OED	35.40	3.00	11.80
425	BERMINGH	B3005	OED	35.40	3.00	11.80
431	BERMINGH	B6005	OED	160.95	13.64	11.80
432	BERMINGH	B6505 C	OED	253.00	22.00	11.50
433	BERMINGH	B6505	OED	202.86	17.64	11.50
434	BERMINGH	B-9	OED	21.00	2.00	10.50
435	BERMINGH	B-32	OED	81.08	7.05	11.50
436	BERMINGH	B-64	OED	166.50	14.11	11.80
437	BERMINGH	B-6505HD	OED	220.50	22.05	10.00
441	MENCK	MHF5-5	ECH	38.69	11.02	3.51
442	MENCK	MHF5-6	ECH	46.43	13.23	3.51
443	MENCK	MHF5-7	ECH	54.17	15.43	3.51
444	MENCK	MHF5-8	ECH	61.91	17.64	3.51
445	MENCK	MHF5-9	ECH	69.65	19.84	3.51
446	MENCK	MHF5-10	ECH	77.39	22.05	3.51
447	MENCK	MHF5-11	ECH	85.13	24.25	3.51
448	MENCK	MHF5-12	ECH	92.87	26.45	3.51
449	MENCK	MHF3-3	ECH	24.76	7.05	3.51
450	MENCK	MHF3-4	ECH	30.96	8.82	3.51
451	MENCK	MHF3-5	ECH	38.69	11.02	3.51
452	MENCK	MHF3-6	ECH	46.43	13.23	3.51
453	MENCK	MHF3-7	ECH	54.17	15.43	3.51
454	MENCK	MHF10-15	ECH	124.73	33.06	3.77
455	MENCK	MHF10-20	ECH	166.28	44.07	3.77
456	MENCK	MHF 5-14	ECH	108.34	30.86	3.51

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
457	MENCK	MHU135T*	ECH	110.59	17.99	6.15
458	MENCK	MHU500T*	ECH	368.74	65.96	5.59
459	MENCK	MHU 300S	ECH	221.20	35.73	6.19
460	MENCK	MHU 270T	ECH	221.20	35.73	6.19
461	MENCK	MHU 200T	ECH	162.24	26.75	6.07
462	MENCK	MHU 400T	ECH	324.37	52.45	6.18
463	MENCK	MHU 500T	ECH	405.53	65.96	6.15
464	MENCK	MHU 700T	ECH	567.72	92.88	6.11
465	MENCK	MHU 840S	ECH	619.22	92.88	6.67
466	MENCK	MHU 600B	ECH	457.03	65.96	6.93
467	MENCK	MHU 600T	ECH	486.63	80.39	6.05
468	MENCK	MHU 800S	ECH	604.57	99.93	6.05
469	MENCK	MHU1200S	ECH	884.84	145.71	6.07
470	MENCK	MHU1500S	ECH	1106.07	178.94	6.18
471	MENCK	MHU1700T	ECH	1400.86	227.36	6.16
472	MENCK	MHU1900S	ECH	1400.86	227.36	6.16
473	MENCK	MHU 150S	ECH	110.59	17.99	6.15
474	MENCK	MHU2700S	ECH	1990.19	318.77	6.24
475	MENCK	MHU 135T	ECH	110.59	17.99	6.15
476	MENCK	MHU 750T	ECH	604.57	99.93	6.05
477	MENCK	MHU1100T	ECH	899.66	145.71	6.18
478	MENCK	MHU150S*	ECH	110.59	17.99	6.15
479	MENCK	MHU600B*	ECH	457.03	65.96	6.93
481	JUNTTAN	HHK3A	ECH	26.05	6.62	3.94
482	JUNTTAN	HHK4A	ECH	34.73	8.82	3.94
483	JUNTTAN	HHK5A	ECH	43.41	11.03	3.94
484	JUNTTAN	HHK6A	ECH	52.10	13.23	3.94
485	JUNTTAN	HHK7A	ECH	60.75	15.43	3.94
486	JUNTTAN	HHK10A	ECH	86.83	22.05	3.94
487	JUNTTAN	HHK12A	ECH	104.19	26.47	3.94
488	JUNTTAN	HHK14A	ECH	121.56	30.88	3.94
491	JUNTTAN	HHK9A	ECH	78.14	19.85	3.94
494	JUNTTAN	HHK16A	ECH	138.92	35.29	3.94
495	JUNTTAN	HHK18A	ECH	156.29	39.70	3.94
496	JUNTTAN	HHK20A	ECH	173.65	44.11	3.94
497	JUNTTAN	HHK4SL	ECH	43.40	8.82	4.92
498	JUNTTAN	HHK3AL	ECH	17.37	6.62	2.63
499	JUNTTAN	HHK4AL	ECH	23.15	8.82	2.63
500	JUNTTAN	HHK5AL	ECH	28.94	11.03	2.63
501	HPSI	110	ECH	44.00	11.00	4.00
502	HPSI	150	ECH	60.00	15.00	4.00
503	HPSI	154	ECH	61.60	15.40	4.00
504	HPSI	200	ECH	80.00	20.00	4.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
505	HPSI	225	ECH	90.00	22.50	4.00
506	HPSI	650	ECH	32.50	6.50	5.00
507	HPSI	1000	ECH	50.00	10.00	5.00
508	HPSI	1605	ECH	83.00	16.60	5.00
509	HPSI	2005	ECH	95.10	19.02	5.00
510	HPSI	3005	ECH	154.33	30.87	5.00
511	HPSI	3505	ECH	176.33	35.27	5.00
512	HPSI	2000	ECH	80.00	20.00	4.00
514	UDDCOMB	H2H	ECH	16.62	4.40	3.77
515	UDDCOMB	H3H	ECH	24.88	6.60	3.77
516	UDDCOMB	H4H	ECH	33.18	8.80	3.77
517	UDDCOMB	H5H	ECH	41.47	11.00	3.77
518	UDDCOMB	H6H	ECH	49.76	13.20	3.77
519	UDDCOMB	H8H	ECH	82.19	17.60	4.67
520	UDDCOMB	H10H	ECH	86.88	22.05	3.94
521	DAWSON	HPH1200	ECH	8.72	2.30	3.79
522	DAWSON	HPH1800	ECH	13.72	3.30	4.16
523	DAWSON	HPH2400	ECH	17.32	4.19	4.13
524	DAWSON	HPH6500	ECH	46.98	10.25	4.58
525	DAWSON	HPH4500	ECH	32.56	7.72	4.22
526	DAWSON	HPH9000	ECH	66.30	10.47	6.33
530	BRUCE	SGH-0312	ECH	26.00	6.60	3.94
531	BRUCE	SGH-0512	ECH	43.34	11.00	3.94
532	BRUCE	SGH-0712	ECH	60.68	15.40	3.94
533	BRUCE	SGH-1012	ECH	86.77	22.05	3.94
534	BRUCE	SGH-0412	ECH	34.67	8.80	3.94
535	BANUT	S3000	ECH	26.04	6.62	3.94
536	BANUT	S4000	ECH	34.72	8.82	3.94
537	BANUT	S5000	ECH	43.41	11.03	3.94
538	BANUT	S6000	ECH	52.09	13.23	3.94
539	BANUT	S8000	ECH	69.45	17.64	3.94
540	BANUT	S10000	ECH	86.81	22.05	3.94
541	BANUT	3 Tonnes	ECH	17.35	6.61	2.62
542	BANUT	4 Tonnes	ECH	23.14	8.82	2.62
543	BANUT	5 Tonnes	ECH	28.92	11.02	2.62
544	BANUT	6 Tonnes	ECH	34.72	13.23	2.62
545	BANUT	7 Tonnes	ECH	40.49	15.43	2.62
550	ICE	70	ECH	21.00	7.00	3.00
551	ICE	75	ECH	30.00	7.50	4.00
552	ICE	110-SH	ECH	37.72	11.50	3.28
553	ICE	115-SH	ECH	37.95	11.50	3.30
554	ICE	115	ECH	46.00	11.50	4.00
555	ICE	160-SH	ECH	64.00	16.00	4.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
556	ICE	160	ECH	64.00	16.00	4.00
557	ICE	220	ECH	88.00	22.00	4.00
558	ICE	275	ECH	110.00	27.50	4.00
559	ICE	DKH-3U	ECH	26.00	6.60	3.94
560	HMC	28A	ECH	28.00	7.00	4.00
561	HMC	28B	ECH	21.00	7.00	3.00
562	HMC	62	ECH	46.00	11.50	4.00
563	HMC	86	ECH	64.00	16.00	4.00
564	HMC	119	ECH	88.00	22.00	4.00
565	HMC	149	ECH	110.00	27.50	4.00
566	HMC	187	ECH	138.00	34.50	4.00
567	HMC	19D	ECH	14.00	3.50	4.00
568	HMC	38D	ECH	28.00	7.00	4.00
569	APE	D 8-42	OED	19.80	1.76	11.25
570	APE	D 1-42	OED	1.32	0.21	6.33
571	APE	D 19-42	OED	47.13	4.19	11.25
572	APE	D 30-42	OED	74.42	6.62	11.25
573	APE	D 36-42	OED	89.30	7.94	11.25
574	APE	D 46-42	OED	114.11	10.14	11.25
575	APE	D 62-42	OED	153.80	13.67	11.25
576	APE	D 80-42	OED	198.45	17.64	11.25
577	APE	D 100-42	OED	248.06	22.05	11.25
579	APE	D 16-42	OED	39.69	3.53	11.25
580	APE	D 16-52	OED	39.69	3.53	11.25
581	APE	D 25-42	OED	62.01	5.51	11.25
582	APE	D 125-42	OED	310.08	27.56	11.25
583	APE	D 50-42	OED	124.03	11.03	11.25
584	APE	D 12-42	OED	29.77	2.65	11.25
585	APE	D 36-26	OED	89.30	7.94	11.25
586	APE	D 128-42	OED	317.25	28.20	11.25
587	APE	D 138-42	OED	342.00	30.40	11.25
588	APE	D 160-42	OED	396.90	35.28	11.25
589	APE	D 180-42	OED	446.51	39.69	11.25
590	APE	D 225-42	OED	558.00	49.60	11.25
591	APE	5.4mT	ECH	26.00	12.00	2.17
592	APE	7.2mT	ECH	51.30	16.20	3.17
593	APE	D 220-42	OED	540.81	48.46	11.16
594	APE	15-60	ECH	150.00	30.00	5.00
595	APE	10-60	ECH	100.00	20.00	5.00
596	APE	400U	ECH	400.00	80.00	5.00
598	APE	750U	ECH	750.00	120.00	6.25
599	APE	D 100-13	OED	300.04	23.70	12.66
600	BSP	DX20	ECH	14.11	3.31	4.27

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
601	BSP	DX25	ECH	18.09	4.41	4.10
602	BSP	DX30	ECH	21.71	5.51	3.94
603	BSP	LX2.5-SA	ECH	14.47	5.51	2.63
604	BSP	LX4-SA	ECH	23.15	8.82	2.63
605	BSP	LX5-SA	ECH	28.94	11.03	2.63
606	BSP	CGL370	ECH	271.22	55.11	4.92
607	BSP	CGL440	ECH	325.47	66.14	4.92
608	BSP	CGL520	ECH	379.71	77.16	4.92
609	BSP	CGL590	ECH	433.96	88.18	4.92
610	BSP	LX7-SA	ECH	40.52	15.44	2.63
625	BRUCE	SGH-1212	ECH	104.13	26.46	3.94
626	BRUCE	SGH-1312	ECH	112.81	28.66	3.94
627	BRUCE	SGH-1315	ECH	141.01	28.66	4.92
628	BRUCE	SGH-1412	ECH	121.48	30.87	3.94
629	BRUCE	SGH-1415	ECH	151.85	30.87	4.92
630	BRUCE	SGH-1612	ECH	138.84	35.27	3.94
631	BRUCE	SGH-1615	ECH	173.55	35.27	4.92
632	BRUCE	SGH-1618	ECH	208.26	35.27	5.90
633	BRUCE	SGH-1619	ECH	219.83	35.27	6.23
634	BRUCE	SGH-1812	ECH	156.19	39.68	3.94
635	BRUCE	SGH-1815	ECH	195.24	39.68	4.92
636	BRUCE	SGH-2012	ECH	173.55	44.09	3.94
637	BRUCE	SGH-2015	ECH	216.94	44.09	4.92
638	BRUCE	SGH-2312	ECH	199.58	50.71	3.94
639	BRUCE	SGH-2315	ECH	249.48	50.71	4.92
640	BRUCE	SGH-3012	ECH	260.32	66.14	3.94
641	BRUCE	SGH-3013	ECH	282.02	66.14	4.26
642	BRUCE	SGH-3015	ECH	325.40	66.14	4.92
643	BRUCE	SGH-4012	ECH	347.10	88.19	3.94
644	BRUCE	SGH-4212	ECH	364.45	92.59	3.94
645	BRUCE	SGH-5012	ECH	433.87	110.23	3.94
650	Twinwood	V20B	ECH	35.58	9.04	3.94
651	Twinwood	V100D	ECH	87.66	22.27	3.94
652	Twinwood	V160B	ECH	140.58	35.71	3.94
653	Twinwood	V400A	ECH	263.84	67.02	3.94
656	Pilemast	24-750	ECH	1.50	0.75	2.00
657	Pilemast	24-900	ECH	1.80	0.90	2.00
658	Pilemast	24-2000	ECH	4.00	2.00	2.00
659	Pilemast	24-2500	ECH	5.00	2.50	2.00
660	Pilemast	36-3000	ECH	9.00	3.00	3.00
661	Pilemast	36-5000	ECH	15.00	5.00	3.00
669	MVE	M-12	OED	30.21	2.82	10.71
670	MVE	M-19	OED	49.38	4.02	12.30

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
671	MVE	M-30	OED	83.35	6.62	12.60
801	DKH	PH-5	ECH	43.40	11.02	3.94
802	DKH	PH-7	ECH	60.75	15.43	3.94
803	DKH	PH-7S	ECH	60.75	15.43	3.94
804	DKH	PH-10	ECH	86.79	22.05	3.94
805	DKH	PH-13	ECH	112.83	28.66	3.94
806	DKH	PH-20	ECH	216.98	44.09	4.92
807	DKH	PH-30	ECH	325.47	66.14	4.92
808	DKH	PH-40	ECH	433.96	88.18	4.92
809	DKH	DKH-713	ECH	112.92	28.66	3.94
850	PILECO	D8-22	OED	18.66	1.76	10.60
851	PILECO	D12-42	OED	29.89	2.82	10.60
852	PILECO	D19-42	OED	42.51	4.01	10.60
853	PILECO	D25-32	OED	58.41	5.51	10.60
854	PILECO	D30-32	OED	70.07	6.61	10.60
855	PILECO	D36-32	OED	84.16	7.94	10.60
856	PILECO	D46-32	OED	107.48	10.14	10.60
857	PILECO	D62-22	OED	161.31	13.67	11.80
858	PILECO	D80-23	OED	197.57	17.64	11.20
859	PILECO	D100-13	OED	246.85	22.04	11.20
860	PILECO	D125-32	OED	308.67	27.56	11.20
861	PILECO	D225-22	OED	555.34	49.58	11.20
862	PILECO	D250-22	OED	617.06	55.09	11.20
863	PILECO	D138-32	OED	340.61	30.41	11.20
864	PILECO	D180-32	OED	444.27	39.67	11.20
865	PILECO	D280-22	OED	688.55	61.73	11.16
866	PILECO	D160-32	OED	395.08	35.28	11.20
867	PILECO	D400-12	OED	810.10	88.15	9.19
868	PILECO	D600-12	OED	1215.10	132.22	9.19
869	PILECO	D800-22	OED	1620.20	176.30	9.19
921	BRUCE	SGH-0212	ECH	17.34	4.40	3.94
922	BRUCE	SGH-0715	ECH	75.77	15.40	4.92
923	BRUCE	SGH-1015	ECH	108.47	22.05	4.92
924	BRUCE	SGH-1215	ECH	130.16	26.46	4.92
925	BRUCE	SGH-2512	ECH	216.94	55.12	3.94
926	BRUCE	SGH-2515	ECH	271.17	55.12	4.92
927	BRUCE	SGH-3512	ECH	303.71	77.16	3.94
928	BRUCE	SGH-3515	ECH	379.64	77.16	4.92
929	BRUCE	SGH-4015	ECH	433.87	88.19	4.92
930	BRUCE	SGH-4215	ECH	455.56	92.59	4.92
931	BRUCE	SGH-4512	ECH	390.48	99.21	3.94
932	BRUCE	SGH-4515	ECH	488.10	99.21	4.92
933	BRUCE	SGH-4712	ECH	407.84	103.62	3.94

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
934	BRUCE	SGH-4715	ECH	509.80	103.62	4.92
935	BRUCE	SGH-4719	ECH	645.74	103.62	6.23
936	BRUCE	SGH-5015	ECH	542.34	110.23	4.92
937	BRUCE	SGH-5715	ECH	618.26	125.66	4.92
938	BRUCE	SGH-6015	ECH	650.80	132.28	4.92
939	BRUCE	SGH-7015	ECH	759.27	154.32	4.92
940	BRUCE	SGH-8015	ECH	867.74	176.37	4.92
949	JUNTTAN	HHK10S	ECH	108.49	22.05	4.92
950	JUNTTAN	HHK28S	ECH	303.78	61.73	4.92
951	JUNTTAN	HHK5S	ECH	54.27	11.03	4.92
952	JUNTTAN	HHK7S	ECH	75.97	15.44	4.92
953	JUNTTAN	HHK9S	ECH	97.68	19.85	4.92
954	JUNTTAN	HHK12S	ECH	130.24	26.47	4.92
955	JUNTTAN	HHK14S	ECH	151.95	30.88	4.92
956	JUNTTAN	HHK16S	ECH	173.65	35.29	4.92
957	JUNTTAN	HHK18S	ECH	195.36	39.70	4.92
958	JUNTTAN	HHK20S	ECH	217.07	44.11	4.92
959	JUNTTAN	HHK25S	ECH	271.22	55.11	4.92
960	JUNTTAN	HHK36S	ECH	390.56	79.36	4.92
961	JUNTTAN	HHU5A	ECH	54.27	11.03	4.92
962	JUNTTAN	HHU7A	ECH	75.94	15.43	4.92
963	JUNTTAN	HHU9A	ECH	97.64	19.84	4.92
964	JUNTTAN	HHU12A	ECH	130.19	26.45	4.92
965	JUNTTAN	HHU14A	ECH	151.88	30.86	4.92
966	JUNTTAN	HHU16A	ECH	173.58	35.27	4.92
968	JUNTTAN	SHK100-3	ECH	26.91	6.61	4.07
969	JUNTTAN	SHK100-3	ECH	35.89	8.82	4.07
970	JUNTTAN	SHK100-3	ECH	44.84	11.02	4.07
971	JUNTTAN	SHK100-3	ECH	53.82	13.23	4.07
972	JUNTTAN	SHK110-5	ECH	44.98	11.02	4.08
973	JUNTTAN	SHK110-5	ECH	53.82	13.23	4.07
974	JUNTTAN	SHK110-5	ECH	65.62	15.43	4.25
975	JUNTTAN	SHK110-5	ECH	77.42	17.64	4.39
976	JUNTTAN	SHK110-5	ECH	87.74	19.84	4.42
977	JUNTTAN	SHK100-5	ECH	44.84	11.02	4.07
978	JUNTTAN	SHK100-5	ECH	53.82	13.23	4.07
979	JUNTTAN	SHK110-7	ECH	65.63	15.43	4.25
980	JUNTTAN	SHK110-7	ECH	77.43	17.64	4.39
981	JUNTTAN	SHK110-7	ECH	87.74	19.84	4.42
998	HYPOTHET	EX 4	OED	23.38	2.75	8.50
999	SELF	Drop/10t	ECH	300.00	20.00	15.00
1001	DFI-Corp	HHA250-4	ECH	25.18	5.51	4.57
1002	DFI-Corp	HHA300-4	ECH	28.75	6.61	4.35

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
1003	DFI-Corp	HHA325-4	ECH	30.36	7.16	4.24
1004	DFI-Corp	HHA350-4	ECH	31.80	7.70	4.13
1005	DFI-Corp	HHA400-6	ECH	51.92	8.80	5.90
1006	DFI-Corp	HHA450-6	ECH	57.04	9.92	5.75
1007	DFI-Corp	HHB500-6	ECH	66.66	11.00	6.06
1008	DFI-Corp	HHB600-6	ECH	77.88	13.20	5.90
1020	J&M	70B HIH	ECH	21.00	7.00	3.00
1021	J&M	82 HIH	ECH	32.80	8.20	4.00
1022	J&M	115 HIH	ECH	46.00	11.50	4.00
1023	J&M	160 HIH	ECH	64.00	16.00	4.00
1024	J&M	220 HIH	ECH	88.00	22.00	4.00
1025	J&M	275 HIH	ECH	110.00	27.50	4.00
1026	J&M	345 HIH	ECH	138.00	34.50	4.00
1134	Pilemer	DKH-3U	ECH	26.00	6.60	3.94
1135	Pilemer	DKH 10L	ECH	86.79	22.05	3.94
1201	Liebherr	H 50/3	ECH	28.97	6.60	4.39
1202	Liebherr	H 50/4	ECH	35.02	8.80	3.98
1203	Liebherr	H 85/5	ECH	43.34	11.00	3.94
1204	Liebherr	H 85/7	ECH	60.16	15.43	3.90
1205	Liebherr	H 110/7	ECH	60.16	15.43	3.90
1206	Liebherr	H 110/9	ECH	78.01	19.85	3.93
1251	ICE	I-30 V2	OED	71.71	6.62	10.84
1261	APE	D 19-52	OED	47.13	4.19	11.25
1262	APE	D 16-32	OED	39.69	3.53	11.25
1263	APE	D 19-32	OED	47.13	4.19	11.25
1264	APE	D 25-32	OED	62.01	5.51	11.25
1265	APE	D 30-32	OED	74.42	6.62	11.25
1266	APE	D 36-32	OED	89.30	7.94	11.25
1267	APE	D 46-32	OED	114.11	10.14	11.25
1268	APE	D 62-22	OED	153.80	13.67	11.25
1269	APE	D 80-23	OED	198.45	17.64	11.25
1270	APE	D 100-32	OED	248.06	22.05	11.25
1271	APE	D 120-32	OED	349.69	27.60	12.67
1272	APE	D 70-42	OED	173.64	15.44	11.25
1273	APE	D 25-52	OED	62.01	5.51	11.25
1274	APE	D 30-52	OED	74.42	6.62	11.25
1275	APE	D 36-52	OED	89.30	7.94	11.25
1276	APE	D 46-52	OED	114.11	10.14	11.25
1277	APE	D 50-52	OED	124.03	11.03	11.25
1278	APE	D 62-52	OED	153.80	13.67	11.25
1279	APE	D 70-52	OED	173.64	15.44	11.25
1280	APE	7.5a	ECH	24.00	12.00	2.00
1281	APE	7.5b	ECH	20.40	10.20	2.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
1282	APE	7.5c	ECH	15.20	7.60	2.00
1283	APE	9.5a	ECH	50.66	16.00	3.17
1284	APE	9.5b	ECH	44.32	14.00	3.17
1285	APE	7-3	ECH	42.00	14.00	3.00
1286	APE	8-3	ECH	48.00	16.00	3.00
1287	APE	8	ECH	16.00	8.00	2.00
1288	APE	8a	ECH	24.00	12.00	2.00
1289	APE	10-4	ECH	80.00	20.00	4.00
1321	MENCK	MHU 240U	ECH	221.20	35.73	6.19
1322	MENCK	MHU 440S	ECH	324.37	52.45	6.18
1323	MENCK	MHU 360U	ECH	324.37	52.45	6.18
1324	MENCK	MHU 550S	ECH	404.06	65.96	6.13
1325	MENCK	MHU 450U	ECH	404.06	65.96	6.13
1326	MENCK	MHU 660S	ECH	485.17	80.39	6.04
1327	MENCK	MHU 540U	ECH	485.17	80.39	6.04
1328	MENCK	MHU 720T	ECH	588.19	99.93	5.89
1329	MENCK	MHU 650U	ECH	588.19	99.93	5.89
1330	MENCK	MHU1000S	ECH	736.48	126.98	5.80
1331	MENCK	MHU 900T	ECH	736.48	126.98	5.80
1332	MENCK	MHU 810U	ECH	736.48	126.98	5.80
1333	MENCK	MHU1700S	ECH	1272.95	207.15	6.15
1334	MENCK	MHU2100S	ECH	1573.92	257.18	6.12
1335	MENCK	MHU3000S	ECH	2216.56	370.23	5.99
1336	MENCK	MHU1400B	ECH	1032.08	145.71	7.08
1337	MENCK	MHU3500S	ECH	2582.43	385.85	6.69
1371	IHC	S-3000	ECH	2211.93	332.44	6.65
1372	IHC	S-4000	ECH	2948.91	444.30	6.64
1401	FAMBO	HR250	ECH	1.81	0.55	3.28
1402	FAMBO	HR500akk	ECH	3.62	1.10	3.28
1403	FAMBO	HR500	ECH	4.34	1.10	3.94
1404	FAMBO	HR1000	ECH	8.68	2.20	3.94
1405	FAMBO	HR1500	ECH	13.02	3.31	3.94
1406	FAMBO	HR2000	ECH	17.36	4.41	3.94
1407	FAMBO	HR2750	ECH	23.87	6.06	3.94
1408	FAMBO	HR3000	ECH	26.04	6.61	3.94
1409	FAMBO	HR4000	ECH	34.72	8.82	3.94
1410	FAMBO	HR5000	ECH	43.40	11.02	3.94
1411	FAMBO	HR7000	ECH	60.75	15.43	3.94
1412	FAMBO	HR8000	ECH	69.45	17.64	3.94
1413	FAMBO	HR10000	ECH	86.79	22.05	3.94
1501	ICE	I-12v2	OED	29.63	2.82	10.50
1502	ICE	I-8v2	OED	18.69	1.76	10.60
1503	ICE	I-19v2	OED	46.14	4.01	11.50

Type	Hammer Make	Hammer Model	Hammer Type	Rated Energy kip-feet	Ram Weight kips	Stroke feet
1504	ICE	I-30v2	OED	76.05	6.61	11.50
1505	ICE	I-36v2	OED	93.73	7.94	11.81
1506	ICE	I-46v2	OED	119.77	10.14	11.81
1507	ICE	I-62v2	OED	172.37	14.59	11.81
1508	ICE	I-80v2	OED	208.30	17.64	11.81
1509	ICE	I-100v2	OED	260.37	22.05	11.81
1510	ICE	I-125v2	OED	310.10	27.56	11.25
1511	ICE	I-160v2	OED	393.45	35.27	11.16
1512	ICE	IP-2	ECH	17.36	4.41	3.94
1513	ICE	IP-3	ECH	26.04	6.61	3.94
1514	ICE	IP-5	ECH	43.40	11.02	3.94
1515	ICE	IP-7	ECH	60.75	15.43	3.94
1516	ICE	IP-10	ECH	86.78	22.04	3.94
1517	ICE	IP-13	ECH	112.83	28.66	3.94
1520	ICE	I-138v2	OED	328.62	30.40	10.81
1531	SPI	D 19-42	OED	42.61	4.02	10.60
1532	SPI	D 30-32	OED	72.08	6.80	10.60
1601	DELMAG	D 2	OED	1.78	0.49	3.61
1602	DELMAG	D 4	OED	3.60	0.84	4.30
1603	DELMAG	D 8-12	OED	20.10	1.76	11.42
1604	DELMAG	D 12-52	OED	33.98	2.82	12.05
1605	DELMAG	D 16-52	OED	40.20	3.52	11.42
1606	DELMAG	D 25-52	OED	66.34	5.51	12.04
1607	DELMAG	D 30-52	OED	75.44	6.60	11.43
1611	DELMAG	D138-32	OED	339.51	30.44	11.16
1612	DELMAG	D180-32	OED	442.64	39.68	11.16
1613	DELMAG	D300-32	OED	737.73	66.14	11.16
1614	DELMAG	D400-32	OED	983.64	88.18	11.16
1620	HMC	TD19	OED	46.09	4.01	11.49
1621	HMC	TD30	OED	69.87	6.61	10.57

Type	Hammer Make	Hammer Model	Hammer Type	Rated Power kW	Ecc. Mass kips	Frequency Hz
620	MAIT	34	VIB	227.00	1.23	33.30
621	MAIT	42	VIB	309.00	1.52	33.30
622	MAIT	54	VIB	450.00	0.98	33.30
623	MAIT	68	VIB	531.00	1.23	33.30
624	MAIT	120	VIB	674.00	1.74	30.00
698	ICE	50B	VIB	432.00	10.42	26.70
699	ICE	3117	VIB	235.00	1.12	28.30
700	ICE	23-28	VIB	21.00	0.10	26.70
701	ICE	216	VIB	130.00	0.46	26.70
702	ICE	216E	VIB	130.00	0.46	26.70
703	ICE	23-Nov	VIB	164.00	0.46	31.70
704	ICE	223	VIB	242.00	0.46	38.30
705	ICE	416L	VIB	242.00	0.92	26.70
706	ICE	812	VIB	375.00	1.82	26.70
707	ICE	815	VIB	375.00	1.84	26.70
708	ICE	44-30	VIB	242.00	1.30	20.00
709	ICE	44-50	VIB	377.00	1.30	26.70
710	ICE	44-65	VIB	485.00	1.30	27.50
711	ICE	66-65	VIB	485.00	1.95	21.70
712	ICE	66-80	VIB	597.00	1.95	26.70
713	ICE	1412B	VIB	597.00	2.04	21.00
714	ICE	1412C	VIB	470.00	2.02	23.00
715	ICE	V125	VIB	984.00	1.04	25.80
716	ICE	14RF	VIB	242.00	1.01	38.30
717	ICE	14-23	VIB	164.00	1.17	35.00
718	ICE	22-23V	VIB	164.00	0.92	26.90
719	ICE	22-30	VIB	250.00	0.92	26.90
720	HMC	3+28	VIB	21.00	0.11	26.80
721	HMC	3+75	VIB	56.00	0.11	36.10
722	HMC	13+200	VIB	149.00	0.35	26.70
723	HMC	13S+200	VIB	149.00	0.35	26.70
724	HMC	13H+200	VIB	164.00	0.35	29.80
725	HMC	25+220	VIB	164.00	0.61	20.90
726	HMC	26+335	VIB	242.00	0.71	25.60
727	HMC	26S+335	VIB	242.00	0.71	25.60
728	HMC	51+335	VIB	242.00	1.21	19.50
729	HMC	51+535	VIB	377.00	1.21	26.40
730	HMC	51S+535	VIB	377.00	1.21	26.40
731	HMC	51+740	VIB	485.00	1.21	27.50
732	HMC	76+740	VIB	485.00	1.82	21.70
733	HMC	76+800	VIB	597.00	1.82	26.10
734	HMC	115+800	VIB	597.00	1.35	20.40
735	HMC	230+1600	VIB	1193.00	2.69	20.40

Type	Hammer Make	Hammer Model	Hammer Type	Rated Power kW	Ecc. Mass kips	Frequency Hz
750	MKT	V-2B	VIB	52.00	0.15	30.00
751	MKT	V-5C	VIB	138.00	0.43	28.33
752	MKT	V-20B	VIB	242.00	0.20	28.33
753	MKT	V-30	VIB	448.00	1.47	28.33
754	MKT	V-35	VIB	485.00	1.60	28.33
755	MKT	V-140	VIB	1341.00	1.17	23.33
770	APE	3	VIB	10.58	0.00	38.30
771	APE	6	VIB	10.58	0.01	38.30
772	APE	15	VIB	59.67	0.11	30.00
773	APE	20	VIB	59.67	0.15	38.30
774	APE	20E	VIB	59.67	0.15	38.30
775	APE	50	VIB	194.00	0.23	30.00
776	APE	50E	VIB	194.00	0.23	30.00
777	APE	100	VIB	194.00	0.32	30.00
778	APE	100E	VIB	194.00	0.14	30.00
779	APE	100HF	VIB	260.00	0.14	43.00
780	APE	150	VIB	260.00	0.14	30.00
781	APE	150T	VIB	260.00	0.17	30.00
782	APE	150HF	VIB	466.00	0.32	43.00
783	APE	200	VIB	466.00	0.29	30.00
784	APE	200T	VIB	466.00	0.34	30.83
785	APE	200T HF	VIB	738.00	0.34	43.00
786	APE	300	VIB	738.00	0.34	25.00
787	APE	400B	VIB	738.00	0.78	23.33
788	APE	600	VIB	800.00	1.05	23.30
789	APE	Tan 400	VIB	1476.00	1.37	23.33
790	APE	Tan 600	VIB	1800.00	2.11	23.30
791	APE	200-6	VIB	470.00	0.43	30.00
811	MGF	RBH 80	VIB	50.00	0.60	30.00
812	MGF	RBH 140	VIB	85.00	1.04	26.67
813	MGF	RBH 200	VIB	125.00	0.74	26.67
814	MGF	RBH 320	VIB	200.00	0.79	26.67
815	MGF	RBH 460	VIB	255.00	1.13	26.67
816	MGF	RBH 1050	VIB	460.00	1.55	22.50
817	MGF	RBH 1575	VIB	700.00	1.16	22.50
818	MGF	RBH 2400	VIB	975.00	1.77	23.50
880	ICE	23RF	VIB	384.00	0.83	38.30
881	ICE	1412BT	VIB	1193.00	1.67	21.70
882	ICE	23-40	VIB	30.00	0.19	31.80
883	ICE	28-35	VIB	261.00	1.16	27.30
884	ICE	28RF-35	VIB	261.00	1.16	27.30
885	ICE	V360	VIB	783.00	0.94	25.00
886	ICE	V360 T	VIB	1566.00	1.88	25.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Power kW	Ecc. Mass kips	Frequency Hz
887	ICE	44-30V	VIB	250.00	0.92	26.00
888	ICE	44-70	VIB	585.00	0.92	28.10
889	ICE	66-70	VIB	585.00	0.92	23.00
890	ICE	7RF	VIB	154.00	0.51	38.30
891	ICE	66-70HS	VIB	585.00	0.92	26.70
892	ICE	66-80HS	VIB	597.00	0.92	29.20
893	ICE	100c-Tdm	VIB	1774.00	1.83	26.67
894	ICE	423	VIB	377.00	0.92	38.30
895	ICE	32RF	VIB	391.00	1.16	33.30
896	ICE	36RF	VIB	431.00	1.30	33.30
897	ICE	46RF	VIB	678.00	1.66	38.30
898	ICE	64RF	VIB	663.00	1.16	32.50
899	ICE	44B	VIB	595.00	1.30	30.00
900	Mueller	MS16HF	VIB	219.00	1.16	39.20
901	Mueller	MS25H2	VIB	218.00	0.90	28.00
902	Mueller	MS25H3	VIB	218.00	0.90	28.00
903	Mueller	MS50H2	VIB	419.00	1.20	27.00
904	Mueller	MS50H3	VIB	419.00	1.20	27.00
905	Mueller	MS25HHF	VIB	274.00	0.58	27.30
906	Mueller	MS50HHF	VIB	562.00	1.17	27.30
907	Mueller	MS100HHF	VIB	750.00	2.33	24.90
908	Mueller	MS120HHF	VIB	895.00	2.30	25.60
909	Mueller	MS200HHF	VIB	837.00	4.25	22.90
910	Mueller	MS-10HFV	VIB	203.00	0.39	39.30
911	Mueller	MS-16HFV	VIB	294.00	0.53	39.20
912	Mueller	MS-24HFV	VIB	720.00	0.85	39.20
913	Mueller	MS-32HFV	VIB	551.00	1.05	39.60
914	Mueller	MS-48HFV	VIB	823.00	1.69	39.20
915	Mueller	MS-62HFV	VIB	735.00	1.82	35.00
1039	J&M	11-23	VIB	164.00	0.92	31.70
1040	J&M	1412	VIB	559.00	1.67	21.70
1041	J&M	1412T	VIB	1119.00	1.67	21.70
1042	J&M	216	VIB	149.00	0.92	26.70
1044	J&M	22-23	VIB	164.00	0.92	20.80
1045	J&M	22-30	VIB	261.00	0.92	27.50
1050	J&M	28-35	VIB	261.00	1.17	27.50
1051	J&M	360	VIB	783.00	0.94	21.70
1052	J&M	416	VIB	250.00	0.92	26.70
1053	J&M	416B	VIB	261.00	0.92	26.70
1054	J&M	416S	VIB	250.00	0.92	26.70
1055	J&M	815	VIB	429.00	0.92	26.70
1056	J&M	44-30	VIB	250.00	0.92	20.00
1057	J&M	44-50	VIB	399.00	0.92	26.70

Type	Hammer Make	Hammer Model	Hammer Type	Rated Power kW	Ecc. Mass kips	Frequency Hz
1058	J&M	44-65	VIB	552.00	0.92	27.50
1060	J&M	66-65	VIB	552.00	0.92	21.70
1061	J&M	66-80	VIB	559.00	0.92	26.70
1100	PVE	14M	VIB	190.00	1.01	28.30
1101	PVE	23M	VIB	234.00	1.66	27.50
1102	PVE	25M	VIB	294.00	0.98	28.30
1103	PVE	27M	VIB	294.00	0.98	28.30
1104	PVE	38M	VIB	392.00	0.92	28.30
1105	PVE	50M	VIB	440.00	1.20	28.30
1106	PVE	52M	VIB	564.00	0.75	28.30
1107	PVE	105M	VIB	784.00	1.52	22.50
1108	PVE	110M	VIB	784.00	0.80	22.50
1109	PVE	200M	VIB	1130.00	1.45	23.30
1110	PVE	2307	VIB	190.00	0.47	38.30
1111	PVE	1420	VIB	190.00	1.01	33.30
1112	PVE	2315	VIB	234.00	1.09	38.30
1113	PVE	2520	VIB	294.00	1.81	33.30
1114	PVE	2310VM	VIB	190.00	0.72	38.30
1115	PVE	2315VM	VIB	234.00	1.09	38.30
1116	PVE	2316VM	VIB	294.00	1.16	38.30
1117	PVE	2319VM	VIB	392.00	1.37	38.30
1118	PVE	2323VM	VIB	392.00	0.83	38.30
1119	PVE	2332VM	VIB	564.00	1.16	38.30
1120	PVE	2335VM	VIB	784.00	1.27	38.30
1121	PVE	40VM	VIB	564.00	1.45	33.30
1122	PVE	50VM	VIB	564.00	1.20	30.00
1123	PVE	55M	VIB	403.00	1.17	28.33
1124	PVE	82M	VIB	565.00	1.76	28.33
1125	PVE	300M	VIB	1796.00	6.21	23.33
1126	PVE	16VM	VIB	335.00	0.35	38.33
1127	PVE	20VM	VIB	395.00	0.41	38.33
1128	PVE	24VM	VIB	395.00	0.52	38.33
1129	PVE	28VM	VIB	403.00	0.61	38.33
1130	PVE	2070VM	VIB	1130.00	1.52	33.33
1131	PVE	2312VM	VIB	252.00	0.26	38.33
1132	PVE	2350VM	VIB	790.00	1.09	38.33
1142	PTC	30HP	VIB	196.00	0.87	27.00
1143	PTC	40HD	VIB	269.00	0.87	28.00
1144	PTC	50HD1	VIB	255.00	0.87	25.00
1145	PTC	50HD2	VIB	290.00	0.87	25.00
1146	PTC	65HD	VIB	305.00	0.87	26.00
1147	PTC	60HD	VIB	305.00	0.87	28.00
1148	PTC	75HD	VIB	410.00	0.87	25.00

Type	Hammer Make	Hammer Model	Hammer Type	Rated Power kW	Ecc. Mass kips	Frequency Hz
1149	PTC	100HD	VIB	451.00	0.87	23.00
1150	PTC	100HDS	VIB	564.00	0.87	23.00
1151	PTC	175HD	VIB	611.00	0.87	23.00
1152	PTC	240HD	VIB	988.00	0.87	23.00
1153	PTC	240HDS	VIB	988.00	0.87	30.00
1154	PTC	120HD	VIB	410.00	0.87	23.00
1155	PTC	130HD	VIB	564.00	0.87	23.00
1156	PTC	200HD	VIB	710.00	0.87	23.00
1157	PTC	265HD	VIB	1080.00	0.87	24.00
1340	H&M	H-150	VIB	94.00	0.11	28.30
1341	H&M	H-1700	VIB	165.00	0.20	20.00
1431	BRUCE	SGV-80	VIB	112.20	0.13	33.33
1432	BRUCE	SGV-100	VIB	142.60	0.18	30.00
1433	BRUCE	SGV-200	VIB	184.80	0.31	28.83
1434	BRUCE	SGV-300	VIB	211.20	0.35	27.50
1435	BRUCE	SGV-400	VIB	286.00	0.44	26.67
1436	BRUCE	SGV-450	VIB	323.40	0.48	26.67
1437	BRUCE	SGV-600	VIB	451.50	0.72	26.67
1438	BRUCE	SGV-1000	VIB	569.10	1.03	25.00
1630	LBFoster	4150	VIB	335.00	0.53	25.00