TECHNICAL REPORT STANDARD PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/LA-89/234		

4. Title and Subtitle	5. Report Date
Determining Pile Bearing Capacity by Some Means Other Than The Engineering News Formula	December 1989
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Kenneth L. McManis, Michael D. Folse and Janet S. Elias	234
9. Performing Organization Name and Address	10. Work Unit No.
Department of Civil Engineering University of New Orleans New Orleans, LA 70122	11. Contract or Grant No. LA HPR Study No. 88-3GT (B)
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Louisiana Transportation Research Center P.O. Box 94245 Baton Rouge, LA 70804	Final Comprehensive Report
	14. Sponsoring Agency Code

15. Supplementary Notes

Conducted in cooperation with the U.S. Department of Transportation Federal Highway Administration

16. Abstract

A review of the practice used in monitoring pile driving activities within the Louisiana Department of Transportation and Development (LADOTD) and elsewhere is reported. The Engineering News Record formula is currently the most commonly reported method used by departments of transportation in the evaluation of pile driving. The performance of several alternate dynamic formulas, the wave equation, and dynamic testing with the pile driving analyzer are evaluated in a comparative study of LADOTD test piles. Development of a comprehensive program that includes dynamic formulas but has the goal of greater reliance on the wave equation, from design through construction, is recommended. Microcomputer software was developed to facilitate field implementation of WEAP87, the Hiley and Engineering News Record formulas. In a test pile study, the pile driving analyzer was found to be reliable in predicting pile capacity, monitoring the structural integrity of the pile during driving, and in evaluating setup.

17. Key Words	18. Distribution Statement
Dynamic Pile Formulas, Wave Equation, Pile Driving Analyzer, Test Piles, Pile Driving	Unrestricted. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	124	

Form DOT F 1700.7 (8-69)

FIGURE B-1

DETERMINING PILE BEARING CAPACITY BY SOME MEANS OTHER THAN THE ENGINEERING NEWS FORMULA

FINAL REPORT

By

KENNETH L. McMANIS
PROFESSOR OF CIVIL ENGINEERING

MICHAEL D. FOLSE
ASSOCIATE PROFESSOR OF CIVIL ENGINEERING

JANET S. ELIAS
RESEARCH ASSISTANT

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF NEW ORLEANS
NEW ORLEANS, LA 70148

Research Report No. 234
Research Project No. 88-3GT(B)

Conducted for

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
LOUISIANA TRANSPORTATION RESEARCH CENTER
in cooperation with
U. S. Department of Transportation
FEDERAL HIGHWAY ADMINISTRATION

December 1989

ACKNOWLEDGEMENTS

The authors would like to acknowledge the assistance and information provided by J.B. Esnard and Mark Morvant, Louisiana DOTD Foundation Engineers, in the creation of the historical data files. Also, Bob Boagni of the Louisiana DOTD Bridge Design Section should be credited for the I-310 Advance Test Pile Program which is reviewed in this study.

ABSTRACT

A review of the practice used in monitoring pile driving activities within the Louisiana Department of Transportation and Development (LADOTD) and elsewhere is reported. The Engineering News Record formula is currently the most commonly reported method used by departments of transportation in the evaluation of pile driving. The performance of several alternate dynamic formulas, the wave equation, and dynamic testing with the pile driving analyzer are evaluated in a comparative study of LADOTD test piles. Development of a comprehensive program that includes dynamic formulas but has the goal of greater reliance on the wave equation, from design through construction, is recommended. Microcomputer software was developed to facilitate field implementation of WEAP87, the Hiley and Engineering News Record formulas. In a test pile study, the pile driving analyzer was found to be reliable in predicting pile capacity, monitoring the structural integrity of the pile during driving, and in evaluating setup.

IMPLEMENTATION STATEMENT

study recommends that the current **Louisiana Standard ications For Roads And Bridges** (1) be revised and expanded to e other methods for evaluating the dynamic performance of piles in addition to the presently specified Engineering New formula. Familiarity and use of the wave equation analysis sign and construction should be encouraged. Field personnel d be instructed on the importance of duplicating the tions on which the dynamic analysis is based and should be ded the means for systematically conducting such an analysis he field. Dynamic measurements should also be considered as a s for supplementing or eliminating static load tests. It is cted that increased use of these advanced techniques will lead more accurate predictions of pile capacity and cost savings for e foundations.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

INTRODUCTION

BACKGROUND

Driven piles often provide the best foundation for facilities constructed on a site where the surface soils are weak and the water table is high. The high cost of piling makes extreme overdesign undesirable; however, failure of a pile under a bridge or other structure can have disastrous monetary and human consequences. Since most bridge foundation piles are loaded primarily in the axial direction, accurate estimates of pile axial capacities can lead to foundations that are both economical and safe.

Because of the critical nature and complexity of the problem, pile axial capacities are often estimated with a three-part program:

1) Capacity estimates based on analyses using information from soil borings and/or other geotechnical investigations,

2) Capacity estimates based on loaded, field test piles, and

3) Capacity estimates based on the driving performance, i.e., "dynamic methods."

Significant differences among the results of the above methods often occur. In many cases this can be attributed to variable soil conditions within the construction site. When load tests have been performed and a given production pile drives similarly to the test pile, actual capacity predictions of the dynamic method are generally ignored in favor of test pile results. However, when load tests are not performed or a pile drives much differently than the test pile, dynamic predictions may be very influential in construction decisions.

Presently, the Louisiana Department of Transportation and Development (DOTD) relies upon the Engineering News Record formula (ENR) in estimating pile capacity during construction. DOTD specifications (1) call for correlation with test pile driving and loading data if the safe bearing capacity of permanent piles is to be determined by formula results alone. It is generally recognized that the ENR is at best an indicator of the actual pile capacity and is not a reliable design tool. In practice, however, the formula has achieved prominence and is regarded as a means of providing the value to be used for bearing capacity. The specific goal of this study is to replace this dependence on the ENR with a more comprehensive and reliable approach.

LITERATURE REVIEW

The so-called "dynamic" methods range from the pile-driving formulas, including the wave equation, to the pile-driving analyzer (PDA). The ENR and most of the pile-driving formulas are based on the principle of energy conservation; i.e., the energy imparted by the hammer ram, minus any losses, should equal the ultimate pile capacity multiplied by the incremental penetration due to the last hammer blow. The method is simple to apply and involves no field expense other than recording blowcounts. Chellis (2) presented the history and use of dynamic formulas, including detailed guidance on hammer efficiencies and coefficients of restitution, and information on driving hammers, piles, and other items pertinent to contemporary pile driving. Derivations for many of the formulas and comparisons between formula predictions and load tests were also presented. As many as 450 dynamic pile formulas have been noted (3). Those formulas most often cited include the ENR, the Modified ENR, the Hiley, the Gates, the Janbu, and the Pacific Coast Uniform Building Code (PCUBC) .

A number of investigations have been made in an attempt to determine the reliability of the various formulas. These were accomplished by comparing the predicted load capacity, computed using individual formulas, to that capacity measured in a load test. The results of some of these investigations are summarized and presented in several texts. Poulos and Davis (4) present a summary of investigations by Sorensen and Hansen (5), Agerschou (6), Flaate (7), Housel (8), and Olsen and Flaate (9), as shown in Tables 1 and 2. Table 1 was produced by Housel for the Michigan Department of State Highways and compares the safety factor range in a pile-test program. Table 2 presents the statistical analysis for the different dynamic formulas and different investigators. Performances of the dynamic formulas were found to vary according to pile material and type, soil conditions, etc. Predictions by the various formulas in these studies have been shown to be unreliable, i.e., sometimes unacceptably high or low. However, the overall conclusion from the above comparisons was that the Janbu, the Danish, and the Hiley formulas involved the least uncertainty, while the most uncertain was the Engineering News Record (ENR) formula (4).

Investigations of the wave equation predictions for ultimate resistance indicate that the reliability of the results is reasonably consistent, and the wave equation is at least as good as the best of the pile-driving formulas (4). Lowery et al. (10) report the accuracy of the wave equation as:

Piles in sand:	+/- 25%
Piles in clay:	+/- 40%
Piles in sand and clay:	+/- 15%

According to Bowles (11), any comparison between the computer output of a wave equation analysis and pile capacity "within 30 percent deviation is likely to be a happy coincidence of input data." However, even with incomplete or unknown input, the wave

TABLE 1

SUMMARY OF SAFETY-FACTOR RANGE FOR EQUATIONS USED IN THE MICHIGAN PILE-TEST PROGRAM (Ref. 8) [a]

Formula	Upper and Lower Limits of Safety Factor Safety Factor = P_u / P_d [b] Pile Capacity Range, kips 0 - 200	200 - 400	400 - 700	Nominal Safety Factor
Engineering News	1.1 - 2.4	0.9 - 2.1	1.2 - 2.7	6
Hiley	1.1 - 4.2	3.0 - 6.5	4.0 - 9.6	3
PCUBC	2.7 - 5.3	4.3 - 9.7	8.8 -16.5	4
Redtenbacher	1.7 - 3.6	2.8 - 6.6	6.0 -10.9	3
Eytelwein	1.0 - 2.4	1.0 - 3.8	2.2 - 4.1	6
Navy-McKay	0.8 - 3.0	0.2 - 2.5	0.2 - 3.0	6
Rankine	0.9 - 1.7	1.3 - 2.7	2.3 - 5.1	3
Canadian NBC	3.2 - 6.0	5.1 -11.1	10.1 -19.9	3
Modified ENR	1.7 - 4.4	1.6 - 5.2	2.7 - 5.3	6
Gates	1.8 - 3.0	2.5 - 4.6	3.8 - 7.3	3
Rabe	1.0 - 4.8	2.4 - 7.0	3.2 - 8.0	2

[a] After Housel (1966)

[b] P_u = ultimate test load

P_d = design capacity, using the nominal safety factor recommended for the equation.

TABLE 2

SUMMARY OF STATISTICAL ANALYSES

mula		Standard Deviation on R	Upper Limit of 96% Safety if Lower Limit is 1.0	Nominal Safety Factor	Number of Load Tests
ineering News	A	0.78	26.0	0.86	171
	F	0.70	17.5	5.8	116
ey	S & H	0.27	3.8	1.4	50
	F	0.37	10.1	2.4	116
bu	S & H	0.25	3.6	2.3	78
	F	0.22	3.2	2.0	116
ish	S & H	0.26	3.8	2.0	78
	O & F	0.28	4.1	3.0	55
	A	0.30	4.2	2.3	123
lwein	S & H	0.57	17.0	7.1	78
sbach	A	0.36	6.0	2.6	123
as	S & H	0.35	5.1	2.3	55

legend: S & H = Sorensen and Hansen (1957)
A = Agerschou (1962)
F = Flaate (1964)
O & F = Olsen and Flaate (1967) (steel piles in sand)
R = ratio of measured to computed load capacity

equation analysis does provide the means for investigating the individual effects of variations in the hammer, hammer accessories, pile type and length, or soil conditions, without load tests.

Predictions based on the dynamic performance of driven piles using a pile-driving formula or the wave equation represent the pile capacity just after driving. However, in some clays the capacity is greatly altered with time due to "setup." This occurs with the dissipation of the induced pore pressure, produced as a result of soil displacement with the penetration of the pile. As it consolidates, the clay can experience a gain in strength and produce an increase in load resistance with time. The loading of test piles is generally conducted at least two weeks after they are driven. Thus, the pile capacity at the time of the test is often substantially higher than the pile capacity at the end of driving. It is also possible that in some soils a "relaxation" occurs, and the pile capacity is somewhat less at the time of loading. These effects should be kept in mind when considering the above comparative studies on method reliability.

The Pile Driving Analyzer (PDA) or Case Method, the wave equation, and the Engineering New Record formula were compared with static load tests in a statistical analysis of 20 sets of pile driving data by Rausch et al. (12), Figure 1. The PDA, wave equation with PDA input (CAPWAP), and ENR methods had correlation coefficients of 0.83, 0.94, and 0.29, respectively. In another presentation of this study (13), Rausch et al. emphasized that most of the dynamic data were obtained within a few hours before or after a static load test was performed so that the effects of setup were included. In another statistical comparison, Denver and Skov (14) concluded that "the procedure where the bearing capacity is estimated on the basis of stress-wave method (Case or CAPWAP) is superior to the traditional procedure where the bearing capacity is estimated by a pile-driving formula." The standard deviation for the ratio of the measured pile capacity to the predicted pile capacity for the

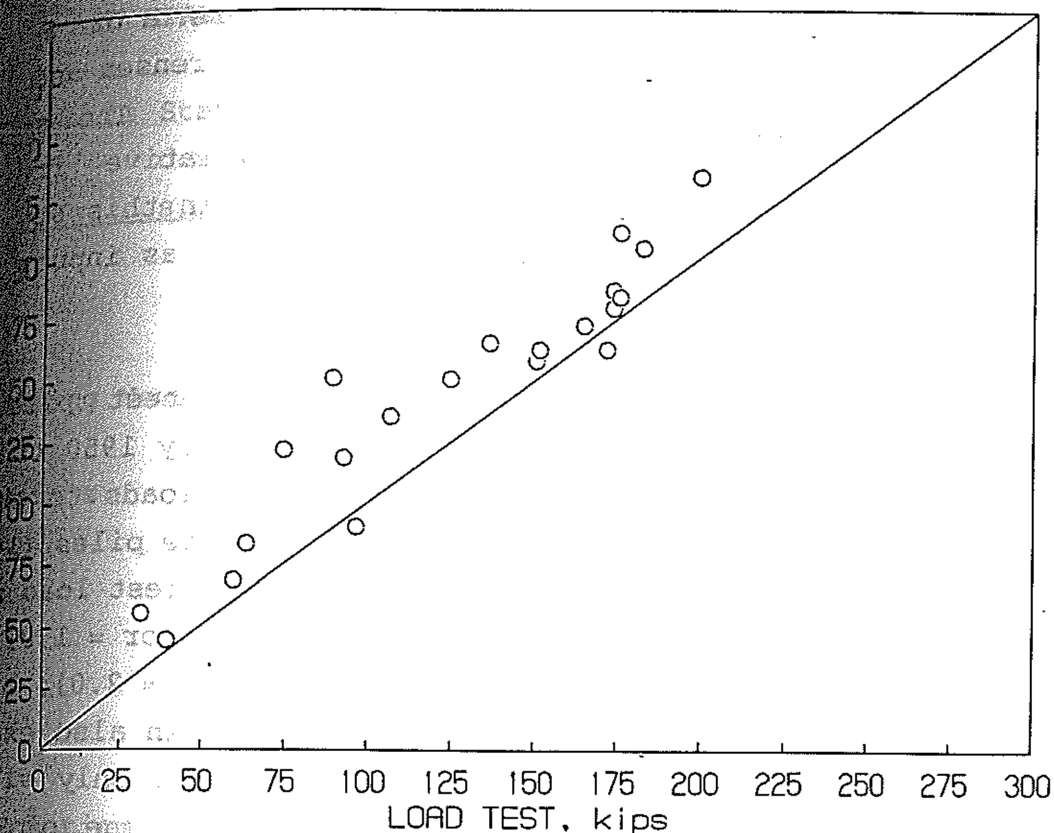


FIGURE 1. Pile Analyzer versus Load Tests (Ref. 12,13)

stress-wave methods, Table 3, was found to be approximately half of the standard deviation for the pile driving formulas. The statistics on the natural logarithm of R where R is $P_{measured}$ / $P_{estimated}$, were performed on the data reported by Sorensen and Hansen (5), Agerschou (6), and Olsen and Flaate (9). The standard deviation s_1 was calculated from the cumulative frequency distribution of ln R. The stress-wave methods in this study did include restrike measurements (not normally used as input in the pile driving formulas).

In 1971, Poplin (15) examined and evaluated test pile data collected by the Louisiana DOTD from approximately 1950 to 1970. Included in the study was a comparison of test loads to the ENR formula predictions for 104 square precast concrete piles (14 inch and 16 inch). The ratio of ENR allowable load to test load ranged from 0.11 (safety factor = 9.0) to 1.0 (safety factor = 1.0). The average ratio (P_{ENR} / P_{TEST}) was 0.506 (safety factor = 2.0), but the standard deviation of 0.183 was quite high. Poplin also examined a soil mechanics prediction of capacity and found only slightly better accuracy on the average. However, the range of safety factors was much less.

Blessey and Lee (16) investigated the use of the wave equation for prediction of pile capacities in the New Orleans area. The scope of their study was "the investigation of the input soil parameters and the development of the relationship of soil resistance from the Wave Equation to actual pile load capacities obtained from the pile load tests performed in the field for both friction and end-bearing piles." Fifty test piles from the New Orleans area were studied. The ratio of the test pile failure load to the wave equation predicted failure (P_{TEST} / P_{WAVE}) was referred to as "R." The method of determining test pile failure load was not stated, but the maximum load applied before pile plunging was probably intended. Input parameters used in the wave equation analysis were given and

TABLE 3

STANDARD DEVIATION FOR PILE DRIVING FORMULAS AND STRESS-WAVE METHOD - DENVER AND SKOV (1988)

[illegible]	Standard deviation s_1 (ln μ)	Standard deviation s (ln μ)	Number of piles	Source
[illegible]	0.30		78	S & H
[illegible]	0.35		123	A
[illegible]	0.27	(0.36)	114	O & F
...ering News	0.90		171	A
[illegible]	0.84	(0.80)	114	O & F
...ein's	0.66		78	S & H
[illegible]	0.41	(0.41)	114	O & F
[illegible]	0.31		50	S & H
[illegible]	0.46	(0.49)	114	O & F
[illegible]	0.29		78	S & H
[illegible]	0.31	(0.38)	114	O & F (C_d=1)
[illegible]	0.35	(0.41)	114	O & F
...Equation	0.26		78	S & H
...ach	0.41		123	A
[illegible]	0.12	(0.14)	97	Goble et al.(1981)
[illegible]		0.11	19	Skov (1988)
[illegible]		0.14	14	Present Investigation
...p [illegible]	0.13	(0.16)	17	Goble et al. (1981)
[illegible]		0.22	26	Different sources
[illegible]		0.10	10	Skov (1988)
[illegible]		0.13	14	Present Investigation

Legend: S & H = Sorensen and Hansen (1957)
A = Agerschou (1962)
F = Flaate (1964)
O & F = Olsen and Flaate (1967) (steel piles in sand)

are reproduced in Table 4. Many of the input items, such as capblock and cushion stiffnesses, were not stated in the report.

For end-bearing prestressed concrete piles, the average R was 1.15 when "minimum" parameters were used in the wave equation. Average R values for average and maximum soil parameters were 0.9 and 0.5, respectively. The least variation between high and low R values was obtained for minimum values. For end-bearing pipe piles, average R values were 1.4, 1.1, and 0.9 for minimum, average, and maximum soil parameters, respectively. Again, the minimum parameters produced the most consistent results.

For friction piles, using the average blowcount for the final five feet of penetration, average R values for prestressed concrete piles were 6.0, 3.53, and 3.3 for minimum, average, and maximum soil parameters, respectively. For friction pipe piles, average R values were 6.0, 4.5, and 3.3. For friction H-piles, average R values were 5.0, 4.0, and 2.9. Minimum and/or average soil parameters produced the most consistent results in all cases.

In a Mississippi DOT study of prediction methods for pile axial capacity, the performance of a modified ENR formula and other techniques were compared with pile load tests (17). The modified ENR had a loss constant C of 0.2 instead of the more common 0.1, and the predicted ultimate load was taken as twice the computed allowable load. The pile capacity in a given load test was defined as the load corresponding to a settlement of one-tenth the pile diameter plus the elastic compression of the pile. Sixty-four test piles, which included mostly 14-to-18-inch-square concrete piles, were compared with the modified ENR. The mean value of the "predicted divided by the load test" was 0.82; the coefficient of variation (cov) was 0.46.

TABLE 4

WEAP INPUT PARAMETERS IN NEW ORLEANS STUDY (Ref. 16)

DIFFERENT TYPES OF COMPUTER INPUT SOIL PARAMETERS

Type of Input Parameter	Type of Soil	Quake (Q) (Inches)	Damping (J) (Sec/ft) Side	Damping (J) (Sec/ft) Point	Hammer Efficiency
Minimum	Clay	0.30	0.20	0.01	0.65
Average	Clay	0.10	0.20	0.01	0.75
Maximum	Clay	0.05	0.10	0.01	0.85
Minimum	Sand	0.20	0.07	0.20	0.65
Average	Sand	0.10	0.05	0.15	0.75
Maximum	Sand	0.05	0.03	0.05	0.85

In 1982 Whited and Laughter (18) described the pile design process for the Arrowhead Bridge located between Superior, Wisconsin, and Duluth, Minnesota. Piles on this job were driven from 130 ft. to 260 ft. through loose sands and soft clays to a dense sand. The two pile types considered were a 16-in. diameter, closed-end pipe (cast-in-place, CIP, concrete filled) and an HP 14 x 73. A minimum bearing of 172 tons, as determined by the Wisconsin driving formula (same as modified ENR for Mississippi described above), was required. Construction control for the job consisted of using both the Wisconsin DOT driving formula and the dynamic pile analyzer. Wave equation analyses using the WEAP computer program were conducted independently by Federal Highway Administration (FHWA) personnel. Results of the wave analysis "indicated that the piles could not have been driven to the capacities measured with the hammer used." The performance of the pile analyzer in predicting load test results was found to be reliable for the H-piles but not for the CIP piles. Errors for the CIP were attributed to larger setup together with the unavailability of restrike data. Based on the test pile program, H-piles were selected and driven to the dense sand. Attempts to use the wave equation to establish driving criteria for production piles were not successful; the Wisconsin DOT standard driving formula was used instead. A comparison of predicted pile capacities with the measured test loads given in this paper is shown in Table 5. The pile analyzer was included in monitoring some production piles and was found to be useful in identifying piles damaged by driving.

TABLE 5

…ISON OF PILE CAPACITY PREDICTIONS WITH CRP LOAD TEST RESULTS[a]

	ULTIMATE LOAD (TON-FORCE)			
…ATION	CRD LOAD TEST	PDA (CASE METHOD)	WEAP	WISDOT FORMULA[b]
…1				
…Pile	375(F)	380	245	405
…P	360+	180		
…3				
…Pile	300(F)	310		
…4				
…Pile	380(F)	330	240	420
…IP	425+	230		

…) = actual failure load

…RP = constant-rate-of-penetration

…ISDOT = Wisconsin Modified ENR

… = Whited and Laughter (18)

… Safety Factor = 2.0 assumed

In a recent Washington state comparative study of formula predictions with pile load tests (19,20), the performances of the ENR, Hiley, Gates, Janbu, and PCUBC formulas were examined. Using an R ratio of the test pile failure loads to the formula predictions for those test piles given in the paper (pile 63 was not included), the following ratio means and coefficients of variation (cov) were computed.

FORMULA	RATIO MEAN	RATIO COV
ENR	0.49	0.54
Hiley	1.11	0.54
Gates	1.71	0.40
Janbu	1.24	0.46

Summary of Literature Review

Unfortunately, most of the classical, simple-to-apply dynamic formulas have been judged by previous research as inaccurate predictors of pile capacity as evidenced by comparisons of measured capacities with dynamic formula predictions for load-tested piles. In fact, several issues affecting these comparisons have not been adequately addressed by most previous research. These include the treatment of time-dependent changes in pile capacity occurring between end of driving and the time of the load test, and the consistent computation of test pile failure loads from load deflection data.

Most researchers found the wave equation approach to be more accurate than any of the formulas. However, it is computationally intensive (requiring appropriate computer hardware and software) and requires much more input. One of the acknowledged shortcomings the wave equation approach is the difficulty in determining propriate values for many of the input items, such as hammer ficiency, coefficients of restitution, distribution of side iction forces, etc. The PDA allows direct measurement of some of ese inputs. However, while the pile analyzer has generally been und to be very successful, it does err considerably in some ses. This continuing uncertainty about the results, together ith its significant additional expense, currently prevents niversal use of the PDA.

OBJECTIVE

The specific objectives of this study were to conduct a review of the current practice for driven pile construction; to create and analyze a local, historical database; to produce a computational tool that can be used at the job site; and to consider other methods not currently used on a routine basis. The general objective was to identify an improved method(s) or philosophy for construction control of driven piles for the Louisiana Department of Transportation and Development. If successful, a greater degree of confidence in the method(s) employed will be developed than exists in the current specifications and practice.

SCOPE

...e of this study was to examine available information ...g the use of dynamic methods employed by the Louisiana ... others and to recommend an approach that will be an ...ent over the current dependence on the Engineering News ...ormula. Several tasks were identified in an attempt to ... the goal. A literature survey and other inquiries were ... identify dynamic methods used in monitoring pile driving. ...ncluded consideration of the philosophies and methods ...ly employed and those being considered by the Louisiana DOTD ...her state transportation departments. Current usage of ...c formulas, the wave equation, and the pile analyzer, as well ...vious research efforts by the Louisiana DOTD on this subject, ... reviewed and are reported herein.

...mparative study of dynamic methods based on local information ... experiences is included. A test pile database was assembled ... DOTD files. Computer software was developed for the creation ... computer data file for each pile selected. One objective of ...s part of the study was to assemble pile load test records that ...tain at least a bare minimum of the information needed for ...luation of dynamic predictions. Records included contained ...ormation documenting the hammer, pile, and soil details and ...ply to piles loaded to failure or to a point sufficient for ...asonably accurate determination of pile capacity using accepted ...thods. The various techniques for interpreting pile capacity ...rom a static load test were reviewed and a consistent method was ...elected. Computer software was written for reading the data files and checking the accuracy of the various dynamic methods in predicting the test pile results.

The relatively low cost and portability of microcomputer hardware and software permit extensive use of computationally intensive methods such as the wave equation. Thus, the use of existing software and the development of additional microcomputer software suitable for field implementation of the selected dynamic method(s) was included.

METHODOLOGY

CURRENT PRACTICE

lications of similar studies were reviewed in order to methods currently used for monitoring pile driving. esearch by the LADOTD was included in this study. It was proposed to conduct a mail and/or phone survey of other sportation departments. However, through the initial review it was discovered that two such surveys had just leted (20,21). Results of these surveys are summarized in rt. Phone inquiries were made to area pile contractors governmental agencies to ascertain their usages and es with dynamic prediction methods.

VE STUDY OF DYNAMIC FORMULAS

capacities predicted by six dynamic formulas and the wave were compared with the measured capacity of piles in a composed of LADOTD test pile records. LADOTD has used the selectively in the recent past; therefore, only a few test ords with the analyzer were available for this study. An on of the pile analyzer with the historical load tests is ible. However, the replacement or supplementation of the ula with a comparable method in terms of effort, expertise, requires consideration of the dynamic formulas, including equation. These techniques (formulas and/or wave equation) ain a vital component of construction planning and control.

FORMULAS SELECTED

the initial project tasks was to select the various dynamic to be evaluated. The ENR was included due to its current use and simplicity and as a basis for comparison. The Hiley, Gates, Janbu, and Pacific Coast Uniform Building Code (PCUBC) formulas were selected because of favorable reviews in the literature which had found them to be more accurate than the ENR. The wave equation method was also selected because of its successful performance in many studies. Descriptions of all of the selected methods can be found in the text Foundation Analysis and Design, third edition, (11) by Joseph E. Bowles. A summary of these methods as given in the Bowles text and used in this research is included below.

Engineering News Formula

The Engineering News formula (ENR) may be expressed as:

$$P_u = \frac{E}{S + C}$$

where:

P_u = Predicted pile axial capacity, kips

E = Rated energy, in-kips, of driving hammer
= Weight of ram W_r, kips, * height of fall H, inches, for free falling rams

S = Pile penetration or "set" due to the latest hammer blow, inches
= 12./(final blowcount in blows per foot)

C = Loss constant, inches
= 1.0 for drop hammers
= 0.1 for all other hammers

The loss constant c is generally considered to account for all losses, including the hammer imperfect efficiency.

Although there are many modifications to the ENR, the above form is used on the test pile reports of LADOTD. Recorded values in the "Pile Capacity in tons, P" column of these reports can be generated he above formula, a safety factor of 6.0, and conversion s to tons. Variations to this formula include application ammer efficiency ratio to the energy and adjustments to the istant.

804.08, Determining Pile Bearing Capacity, of the **Louisiana d Specifications for Roads and Bridges** (1), requires that be used "if the safe bearing capacity is to be determined ulas." The following form of the ENR formula is specified as a guide and for correlation with test pile driving and data.

$$P = \frac{2W_rH}{S + C}$$

here:

P = Safe bearing capacity, pounds
W_r = Weight of hammer ram, pounds
H = Height of fall, feet
S, C = As defined above

d Engineering News Record

ied ENR (ENRMOD) is presented in Formulated Pile Loads For Acting and Approved Differential Hammers, (22) a manual of nes for DOTD inspectors. It is an attempt to account for ght of the pile with respect to the weight of the ram. In owable load format it is written as follows:

$$P = \frac{2W_rH}{S + .1(W_p/W_r)}$$

where:

W_p = weight of the pile, pounds, and other terms are as defined in the allowable load form of the ENR above.

This formula is also known as the Eytelwein formula. The inspectors' manual indicates that the applicable formula for the use of diesel hammers will be based on a performance comparison and correlation with a single-acting hammer of the same energy range or will be acceptable on a basis of 85% of the rated energy of the diesel hammer.

Hiley Formula

The Hiley formula may be expressed:

$$P_u = \frac{(e_h\ E)\quad (W_r + n^2 W_p)}{(s + .5(k_1 + k_2 + k_3))\ (W_r + W_p)}$$

where:

P_u = Predicted pile capacity, kips

E = Rated energy, in.-kips, of hammer

s = Pile set, in., due to latest hammer blow

e_h = Hammer efficiency, as a fraction of 1.0

W_r = Ram weight, kips

W_p = Pile weight, kips (including pile cap)

n = Coefficient of restitution

k_1 = Elastic compression of capblock and pile cushion, in., (a pile cushion is normally used only on concrete piles)

k_2 = Elastic compression of pile, in.,

k_3 = Elastic compression of the soil ("quake"), in.

A safety factor of 3.0 is commonly applied to Hiley formula predictions.

Formula

tes formula can be expressed as:

$$P_u = a * SQRT(e_h E) * (b - \log s)$$

where:

P_u = Predicted pile capacity, kips
a = 27.0 feet per second(fps)
b = 1.0 fps (a and b are empirical constants)
SQRT = Abbreviation for "square root"
* = Abbreviation for "multiply"
e_h = Hammer efficiency
= 0.75 for drop hammers
= 0.85 for all other hammers
log = Abbreviation for base ten logarithm
s = Pile set, in., due to final hammer blow

Gates formula was derived through a statistical correlation tween final blowcounts (or set equivalents) and selected test le results. This is unlike the other formulas, which are based energy conservation. A safety factor of 3.0 is commonly used th the Gates formula.

nbu Formula

ne Janbu formula can be expressed as:

$$P_u = \frac{e_h E}{k_u s}$$

where:

P_u = Predicted pile capacity, kips
e_h = Hammer efficiency
E = Rated energy of hammer, in.-kips
k_u = $C_d(1. + SQRT(1. + u/C_d))$
C_d = $0.75 + 0.15(W_p/W_r)$

Soil -

length of pile penetration, percentage and distribution of skin friction, soil damping and quake values along the side and at the pile tip, ultimate soil resistance, etc.

A number of somewhat different computer programs for solution of the wave equation have been produced (23,24,25). The Texas Transportation Institute produced the TTI Wave Equation (24) for the Federal Highway Administration (FHWA) in 1976 to assist highway engineers in analyzing practical pile problems. The Wave Equation Analysis of Pile Driving (WEAP) program was developed at Case-Western University in 1976 for the FHWA. It provides several pile-driver simulation routines with improved computer models for diesel hammers and air/steam hammers. The latest version, WEAP87 (26), is available and can be run on a microcomputer.

METHOD OF EVALUATION

The accuracy of a dynamic method is generally judged by comparing its predicted ultimate capacities to measured capacities for load-tested piles. A method which does a good job predicting load test results is assumed to be accurate in its predictions of capacities for the much more numerous non-load-tested piles. There are several shortcomings to this evaluation process that will be discussed below. However, the comparison to load test results is presently the most common and widely accepted evaluation technique.

Load Test Records

Records for test piles, dating back twenty years, were obtained from the LADOTD Headquarters Office in Baton Rouge. These files included test piles from almost all parishes in Louisiana. All of the files were studied and almost all of those meeting the following criteria were selected for the database.

1. Pile loaded beyond linear portion of the load versus displacement curve.
2. Pile driven with a hammer contained in the WEAP87 hammer file or a similar hammer.
3. Sufficient soil information to compute capacity.

urces of pile-driving records were sought. However, nearly incomplete with respect to the information required. Even e requirements for inclusion of the database might be d as "barely adequate" for the study undertaken.

ard form was developed to facilitate extraction of ole data from test pile files and entry of this data into a r file. This form is shown in Figure 2. The intent is that cord contain sufficient information for executing all methods being studied and computing pile capacity by a soil cs approach.

tion contained on the data extraction forms was transferred uter files so it could be readily and rapidly accessed and d. Use of a proprietary database program was considered, ce the resulting data files would have to be accessed by a e analysis program, it was decided to custom write the data reation software.

cal computer file for one of the test piles is shown in 3. Every second line in the file contains one piece of ation. The preceding line in each case describes the piece ormation to follow. This was done to facilitate changes to le after it has been created and to minimize the chances of g a piece of information in the wrong place. The files are andard ASCII files.

Engineer____________________ Date____________________

Rev.11/89

Louisiana Department of Transportation and Development
Pile Test Data Bank Entry Form
File Name:
Test Pile Number:
State:
Parish:
Additional Location info:

State project no.:
Geographic code:
Date of driving:
Date of testing:
Pile description:
Pile type code(1=tim,2=con,3=st,4=other,5=compos,6=mandrel driven):
Pile length,ft:
Pile embedment,ft:
Ground elev,ft:
Pile tip area,sq in(b*d for H piles,total enclosed area pipe pile):
Pile butt area,sq in(pile cross section area):
Depth to water table,ft:
Predrilled hole diameter,in:
Predrilled hole depth,ft:
Jetting depth,ft:
Final blow count,blows per ft:
Avg blowcount last five feet:
Approx. avg blow count entire embed:
Description of hammer:
Hammer type code(1=sgl act air/stm,2=dbl act air/stm,
3=op end diesel,4=cl end diesel,5=drop,6=other):
Hammer number from table:
Wt. of hammer ram,kips:
Total weight of hammer,kips(often approx twice ram weight):
Hammer rated energy,ft-kips:
Speed of ram,blows per minute:
Hammer energy efficiency ratio:
Design load per pile,tons:
Maximum test load,tons:
Duration of maximum load,hours:
Total pile deflection at maximum pile load,in:
Pile deflection at 50% of maximum load,in:
Pile deflection at 75% of maximum load,in:
Permanent deflection due to test load,in:
Estimated ratio test load to failure load at testing:
Method used to determine above ratio:

Figure 2. Data Extraction Form

side soil(1=sand,2=stiff to med clay,3=med to soft
soft to very soft clay):
ercent skin friction:
etup factor from end of driving to start of testing:
ile cross sections to be input:(number 1 at top)
n of pile with section numbers,x-sect area,sq in,
in,(2*(b+d)for H-piles),modulus of elasticity,ksi,
ht,pcf, and extent of each section

soil layers to be in soil model:(number 1 at top)
h of soil profile with layer numbers,soil descriptions,
ses,ft,total unit weights,pcf,angles of friction,degrees,
ed compressive strengths,psf,% moisture content,
limit, % plasticity index, each layer

weight,kips(Pile cap includes capblock and helmet
mes called hood);alternating layers of 1" micarta
" aluminum generally used)
stiffness,kips/inch(capblock is source of spring):
assumed pile cushion type,if any(cushions generally
ly with concrete piles):
hion stiffness,kips/inch:
ght,kips(anvil only on diesel hammers):
efficient of restitution(cor)):
cor:
hion cor(if cushion used):
cor:
cor for formulas:
ping factor:
mping factor:
r skin friction:
r point resistance:
iction distribution type(number from table):
ed Pile Failure Load, kips,by Weap86:
ed WEAP Failure Load,kips using default values
ll input:

Figure 2. (cont)

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
TEST PILE DATA BANK

```
FILE NAME
  LATP.091
TEST PILE NUMBER
  091
STATE IN WHICH TEST PILE IS LOCATED
  LOUISIANA
PARISH OR COUNTY OF TEST PILE
  EAST FELICIANA
ADDITIONAL LOCATION INFORMATION
  CLINTON-OLIVE BRANCH HWY, LA-67
  TEST PILE # 1
STATE PROJECT NUMBER,IF ANY
  60-03-12
DATE OF DRIVING
  9-9-81
DATE OF TESTING
  10-6-81
PILE DESCRIPTION
  16" PRECAST CONCRETE, L=50'
PILE TYPE CODE(1=TIM,2=CONC,3=STL,4=OTH,5=COMP,6=MAND DRIV)
  2
PILE LENGTH,FT
  50.00
PILE EMBEDMENT,FT
  34.00
GROUND ELEVATION,FT,AT TEST PILE
  199.9
PILE TIP BEARING AREA,SQ IN
  256.00
PILE BUTT AREA,SQ IN
  256.00
DEPTH TO WATER TABLE,FT
  0.00
PREDRILLED HOLE DIAMETER,IN
  0.00
PREDRILLED HOLE DEPTH,FT
  0.00
JETTING DEPTH,FT
  0.00
FINAL BLOW COUNT IN BLOWS PER FOOT OF PENETRATION
  33.00
AVERAGE BLOW COUNT LAST FIVE FEET
  26.40
APPROX AVG BLOW COUNT ENTIRE EMBED
      47.
```

Figure 3. Computer Data File (Example)

```
SCRIPTION OF HAMMER
VULCAN NO. 1
MER TYPE CODE(1=SAAS,2=DAAS,3=OED,4=CED,5=DROP,6=OTHER)
1
MER NUMBER FROM TABLE
204
IGHT OF HAMMER RAM,KIPS
5.00
TAL WEIGHT OF HAMMER,KIPS
10.00
MMER RATED ENERGY,FT-KIPS
15.0
EED OF HAMMER RAM,BLOWS/MIN
55.0
MMER ENERGY EFFICIENCY RATIO
0.670
SIGN LOAD PER PILE,TONS
57.40
XIMUM TEST LOAD,TONS
143.50
URATION OF MAXIMUM LOAD,HOURS
2.00
OTAL PILE DEFLECTION AT MAXIMUM LOAD,IN
0.19
ILE DEFLECTION AT 50 % OF MAXIMUM LOAD,IN
0.03
ILE DEFLECTION AT 75 % OF MAXIMUM LOAD,IN
0.08
ERMANENT DEFLECTION DUE TO TEST LOAD,IN
0.0938
STIMATED RATIO TEST LOAD TO FAILURE LOAD AT TIME OF TESTING
0.951
NAME OF METHOD USED TO CALC THIS RATIO
Van der Veen
NUM PREDOM SIDE SOIL,1=SAND,2=ST TO MED CL,3=M TO S,4=S TO VS
1
ESTIMATED PERCENT SKIN FRICTION AT END OF DRIVING
70.
ESTIMATED SETUP FACTOR FROM END OF DRIVING TO START OF TESTING
1.00
NUMBER OF PILE X-SECTIONS TO BE INPUT
1
X-SECTION AREA,SQ IN, SECTION    1
256.000
SIDE FRICT PERIM,IN,SECTION    1
64.000
MODULUS OF ELASTICITY,KSI,SECTION    1
3640.000
UNIT WEIGHT,PCF,SECTION    1
150.00
```

Figure 3. (cont)

A FORTRAN computer program, PILET, was written to allow interactiv transfer of information from the data extraction form to th computer file. Upon running PILET from a terminal, the operator i prompted for each piece of information in the same sequence as i on the form. A listing of PILET is available upon request from th authors.

In addition to creating an ASCII file named "LATP.xxx," wher "xxx" is the file number, PILET adds each pile to a cumulati catalogue file named "LATP.CAT," which stores the pile number an certain key information. A listing of the catalogue file is give in Figure 4.

Test Pile Failure Loads

Dynamic formulas and the wave equation are premised upon relationship between the blowcount and pile axial capacity. Sinc this relationship is evaluated on the basis of test pile result the question arises as to with what test pile load the final pi blowcount should correlate. For piles that are load-tested aft a time delay (this includes most test piles), the issue of setup part of the question and will be discussed below. For the prese discussion, it is assumed that the pile is load-tested immediate after the final hammer blow.

There are many alternate methods of determining "failure" load fr a given load versus deflection curve for a test pile. It difficult to say which of these often very different failure loa should correlate with the predicted ultimate load of a giv dynamic method. There is some logic to assuming that the test lo corresponding to a deflection equal to the penetration of the fin hammer blow should correlate to this prediction. This deflecti may be considerably more or less than what actually constitu failure of the pile in its design use.

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
CATALOGUE FILE OF PILE LOAD TEST FILES(LATP.CAT)

CATALOGUE ENTRIES LATP.XXX ARE AUTOMATICALLY ADDED BY PROGRAM PILET WHEN THE FILE LATP.XXX IS CREATED.
HOWEVER, IF LATP.XXX IS LATER EDITED IN THE CATALOGUE FIELDS, THIS INFORMATION MUST ALSO BE EDIT MODIFIED IN THIS CATALOGUE

LIST OF ABBREVIATIONS:

```
ENT    = ENTRY NUMBER
STA    = STATE
PAR    = PARISH OR COUNTY
YR     = YEAR
MON    = MONTH
PT     = PILE TYPE
         WD = WOOD POLE
         CS = SQUARE CONCRETE
         CP = COMPOSITE(WOOD POLE WITH CIP CONC TOP)
         PO = OPEN END PIPE PILE
         PC = CLOSED END PIPE PILE
         HP = STEEL H-PILE
         CH = HOLLOW CIRCULAR
KDM    = KEY DIMENSION,INCHES,DIAMETER FOR CIRCULAR
        SECTION,DEPTH FOR SQUARE OR H
EMB    = PILE EMBEDDMENT,FT
MTL    = MAXIMUM TEST LOAD,TONS
PDF    = PERMANENT DEFLECTION,IN,DUE TO TEST LOAD
PSL    = PREDOMINANT SOIL TYPE (SAND,CLAY,OR BOTH)
HNO    = DRIVING HAMMER NUMBER FROM WEAP TABLE
FBC    = FINAL BLOW COUNT, BLOWS PER FOOT
```

NT	STA	PAR	YR	MON	PT	KDM	EMB	MTL	PDF	PSL	HNO	FBC
1	LA	ACAD	1980	2	CS	14	32	95	0.070	CLAY	55	15
2	LA	JEFF	1981	6	OR	12	114	145	0.078	CLAY	206	7
3	LA	JEFF	1981	6	OR	16	109	140	1.250	CLAY	206	62
4	LA	JEFF	1981	6	OR	14	96	145	0.141	MIXD	206	150
5	LA	JEFF	1981	6	OR	14	105	145	0.688	MIXD	206	60
6	LA	JEFF	1981	6	OR	16	107	125	0.141	CLAY	206	21
7	LA	JEFF	1982	12	OR	17	113	145	0.083	MIXD	253	33
8	LA	JEFF	1982	12	OR	18	104	187	0.063	CLAY	253	60
9	LA	JEFF	1982	12	OR	17	116	145	0.030	CLAY	253	68
10	LA	JEFF	1983	1	OR	18	107	187	0.021	MIXD	253	23
11	LA	IBER	1986	5	CS	18	71	128	0.031	SAND	9	80
12	LA	IBER	1985	5	CS	16	50	142	0.250	CLAY	172	40
13	LA	IBER	1985	5	CS	16	50	107	0.063	CLAY	172	25
14	LA	ORLS	1985	10	CS	16	69	150	0.109	MIXD	224	74

Figure 4. Catalogue File

15	LA	ORLS	1986	1	CS	16	73	162	0.109	MIXD	224	267
16	LA	ORLS	1985	11	OR	20	62	245	0.797	MIXD	255	30
17	LA	ORLS	1986	5	CS	16	90	135	0.016	CLAY	208	14
18	LA	ORLS	1986	4	CS	16	91	137	0.000	CLAY	208	9
19	LA	ORLS	1986	4	CS	16	90	137	0.000	CLAY	208	41
20	LA	ORLS	1986	4	OR	12	55	24	0.328	CLAY	206	3
21	LA	ORLS	1986	4	CP	12	51	24	0.344	CLAY	206	3
22	LA	ORLS	1986	4	CP	12	51	21	0.281	CLAY	206	3
23	LA	ORLS	1986	3	CP	12	61	30	0.313	CLAY	206	2
24	LA	ORLS	1986	3	CP	12	61	30	0.484	CLAY	206	3
25	LA	ORLS	1986	3	CP	12	61	30	0.375	CLAY	206	3
26	LA	ORLS	1982	6	CS	30	122	428	0.328	MIXD	68	200
27	LA	ORLS	1980	4	CS	16	70	113	0.016	CLAY	207	186
28	LA	ORLS	1980	4	CS	16	52	113	0.094	CLAY	207	100
29	LA	ORLS	1982	4	PO	8	37	25	0.016	MIXD	304	113
30	LA	SMRY	1981	3	CS	16	48	11	0.000	CLAY	207	13
31	LA	SMRY	1981	3	CS	18	44	82	0.000	CLAY	207	22
32	LA	SMRY	1981	3	CS	16	56	111	0.188	CLAY	207	11
33	LA	ORLS	1986	3	CP	12	70	35	0.094	CLAY	206	2
34	LA	ORLS	1986	3	CP	12	71	40	0.000	CLAY	206	4
35	LA	ORLS	1986	4	CP	12	71	43	0.031	CLAY	206	5
36	LA	VERM	1974	2	CS	24	51	221	0.094	CLAY	212	13
37	LA	VERM	1974	2	CS	24	45	221	0.063	MIXD	212	15
38	LA	VERM	1974	3	CS	24	54	255	0.063	MIXD	212	135
39	LA	VERM	1974	3	CS	24	48	221	0.094	MIXD	212	16
40	LA	VERM	1981	3	CS	16	37	125	0.063	MIXD	235	57
41	LA	VERM	1977	3	CS	54	102	665	0.094	CLAY	214	71
42	LA	VERM	1980	10	CS	16	63	107	0.313	CLAY	206	25
43	LA	SMRT	1982	12	CS	18	76	106	0.125	MIXD	24	8
44	LA	TERR	1978	8	CS	24	59	128	0.344	CLAY	23	6
45	LA	TERR	1979	4	CS	24	68	146	0.375	CLAY	23	22
46	LA	TERR	1980	1	CS	24	91	147	0.031	CLAY	212	6
47	LA	TERR	1986	3	CS	24	98	146	0.500	CLAY	177	14
48	LA	TERR	1984	12	CS	24	59	178	0.219	CLAY	181	14
49	LA	TERR	1985	1	CS	24	86	223	0.531	CLAY	181	10
50	LA	TERR	1985	1	CS	24	107	251	0.313	CLAY	181	22
51	LA	TERR	1985	1	CS	24	97	186	0.375	CLAY	181	10
52	LA	RAPI	1981	3	HP	14	89	213	0.109	MIXD	253	134
53	LA	EBAT	1982	4	WD	15	46	100	0.094	CLAY	204	100
54	LA	EBAT	1982	4	WD	15	35	90	0.000	CLAY	204	38
55	LA	EBAT	1984	3	CS	16	62	150	0.000	CLAY	172	57
56	LA	EBAT	1983	6	CS	18	66	172	0.000	MIXD	208	32
57	LA	EBAT	1983	6	CS	24	80	312	0.078	CLAY	212	43
58	LA	EBAT	1984	3	CS	16	56	107	1.000	CLAY	207	51
59	LA	EBAT	1979	11	CS	14	58	108	0.031	CLAY	204	125
60	LA	EBAT	1978	7	CS	14	44	150	0.047	CLAY	175	38
61	LA	EBAT	1978	7	CS	14	46	150	0.016	CLAY	175	30
62	LA	EBAT	1978	7	CS	14	43	150	0.000	CLAY	175	45
63	LA	EBAT	1987	8	CS	14	57	112	0.031	MIXD	204	72

Figure 4 (cont)

	EBAT	1980	3	CS	14	51	120	0.000	CLAY	206	56
	EBAT	1980	4	CS	14	46	120	0.000	CLAY	206	122
	EBAT	1979	12	WD	15	40	100	0.313	CLAY	204	70
	EBAT	1979	12	WD	15	40	100	0.063	CLAY	204	98
	EBAT	1979	12	WD	15	40	100	0.203	CLAY	204	63
	EBAT	1979	12	WD	16	40	100	0.156	CLAY	204	60
	EBAT	1979	12	WD	15	40	100	0.469	CLAY	204	53
	EBAT	1979	12	WD	15	40	92	1.031	CLAY	204	53
	EBAT	1979	12	WD	16	40	100	0.188	CLAY	204	59
LA	VNON	1981	5	CS	16	35	115	0.063	SAND	172	98
LA	VNON	1978	6	CS	16	63	125	1.000	SAND	208	24
LA	MORE	1979	3	CS	24	60	255	0.328	SAND	210	96
LA	JACK	1981	6	CS	14	40	115	0.156	MIXD	150	135
LA	RAPI	1983	5	CS	16	59	115	0.500	CLAY	13	10
LA	RAPI	1983	5	CS	16	51	96	0.438	MIXD	13	10
LA	RAPI	1983	8	CS	16	57	100	0.625	CLAY	13	5
LA	RAPI	1981	9	CS	24	79	235	0.453	MIXD	66	50
LA	RAPI	1982	9	PO	30	45	540	0.000	MIXD	66	151
LA	EBAT	1979	12	WD	16	40	100	0.344	CLAY	204	67
LA	EBAT	1980	1	WD	15	40	100	0.125	CLAY	204	72
LA	EBAT	1980	1	WD	15	38	100	0.094	CLAY	204	56
LA	EBAT	1980	1	WD	15	43	145	0.625	CLAY	204	125
LA	EBAT	1980	2	CS	14	61	85	0.563	CLAY	204	24
LA	ASCN	1982	4	CS	16	87	114	0.750	CLAY	206	19
LA	AVOY	1985	4	CS	16	65	96	0.125	CLAY	4	14
LA	AVOY	1985	1	CS	24	71	150	1.625	CLAY	13	52
LA	AVOY	1985	1	CS	24	68	218	1.125	CLAY	13	55
LA	EFEL	106	10	CS	16	34	143	0.094	SAND	204	33
LA	JDAV	1978	5	CS	30	77	288	0.375	SILT	212	24
LA	JDAV	1978	5	CS	30	53	224	0.250	CLAY	212	19
LA	JEFF	1984	3	CS	16	83	112	0.031	CLAY	183	6
LA	JEFF	1984	3	CS	16	87	112	0.016	CLAY	183	6
LA	JEFF	1977	8	CS	16	100	145	0.906	CLAY	210	19
LA	JEFF	1977	9	CS	18	135	110	0.844	CLAY	210	32
LA	JEFF	1977	8	OR	20	100	185	0.797	CLAY	134	26
LA	JEFF	1977	7	PO	16	120	278	0.641	MIXD	206	71
LA	JEFF	1977	9	PO	14	100	190	0.938	CLAY	206	16
LA	JEFF	1977	8	PO	16	135	207	0.953	CLAY	206	28
LA	STCH	1987	7	CH	54	80	398	0.470	CLAY	24	38
LA	STCH	1987	7	CS	24	80	210	0.262	CLAY	24	10
LA	STCH	1987	7	CS	30	80	267	0.308	CLAY	24	8
LA	STCH	1987	7	CS	30	80	270	0.452	CLAY	24	23
LA	STCH	1987	7	CH	36	80	270	0.481	CLAY	24	15
LA	STCH	1987	7	CH	36	80	270	0.585	CLAY	24	46

Figure 4. (cont)

Several methods for determining pile capacity were consi These included the Van der Veen (27), Mazurkiewicz Davisson(28), Chin (29), AASHTO, and Swedish Ninety P Criterion technique. After reviewing these methods an existing load tests, it was determined that several could n used. Many of the test piles had not been loaded far enou produce a load-settlement curve required by some of the metho the requirements of the method did not fit test procedures, conditions, etc. The Van der Veen and Mazurkiewicz were consi possible candidates.

In comparing the Van der Veen and Mazurkiewicz methods, they to predict similar failure loads. The Van der Veen method u mathematical representation of the load curve near failure, the Mazurkiewicz uses a more cumbersome graphical method to d the point of ultimate load. In addition, the Van der Veen m has been successfully used in a previous study on pile desig Louisiana (30). Thus, the Van der Veen method was selected for based on its simplicity, consistency, and previous use in LA studies. The failure loads derived by the Van der Veen and o test load analysis methods differ by an unknown amounts from capacity to which the blowcount "should" correlate. This sourc error deserves future study.

Van der Veen proposed the following relation between a pi ultimate capacity and its load versus deflection behavior:

$$Q = Q_u \left(1. - e^{-rz} \right)$$

where:

Q = Applied load causing butt deflection z

Q_u = Pile ultimate capacity

r = Coefficient determined from the load-deflection curve

Using two (Q,z) points near the upper end of load-deflection curve, Q_u and r can be determined.

ifications (1) require a load test when the bearing
mputed by the ENR formula is less than twice the design
ding of the test piles generally does not begin for at
ys after installation. The test loading consists of the
of incremental static loads on the pile and measuring
ting settlement. Test piles are loaded to failure or
times the design load is reached. The test pile is
to have failed when the permanent settlement at the top
le is 1/4 inch (regardless of pile size). The Van der
d worked very well for most of these test pile records.

tors

losses in pile capacity often begin immediately after the
riving. Depending on the soil environment, water table,
driving, type of pile, length of pile, and possibly other
he pile capacity may change significantly between the end
ng and a load test conducted two to four weeks later.
ubmerged soft clays, deriving most of their capacity from
tion in these clays, tend to significantly increase in
during the first month after driving. Conversely, piles
through clays but deriving practically all of their
from a hard stratum below may lose capacity because of
skin friction if the clays are underconsolidated. This
nsolidation may occur naturally or may be due to a recent
of the water table or placement of fill.

heast Louisiana, most piles increase significantly in
h (setup) during the first few weeks after driving,
s 400 to 500 percent (16). Logically, the end-of-driving
nt can only be expected to predict the pile capacity at the
driving; it cannot predict a significantly different
y at the time of the load test (or the time of design
for the typical production pile). In recognition of this
he practice of "restriking" is growing. Restriking refers
riving a pile for a short distance after some time delay.

It can be performed after a majority of the time-dependent changes are assumed to have occurred. The restrike blowcount, along with the characteristics of the restriking hammer, are used to predict capacity. For load-tested piles, the restrike can be performed immediately before or after the load test. There are, however, several problems associated with restriking:

1) The restriking must be performed after an appropriate delay. Pile accessibility is often impaired by installation of surrounding piling. Furthermore, there is considerable cost involved in resetting the pile driver over each pile.

2) In soils of considerable setup, the pile hammer used for production driving may not be of adequate size to restart the pile. A suitable starter hammer or other device for obtaining an "after setup" blowcount or pile analyzer data may not be suitable for driving additional pile length, should it be required.

3) Very little restrike data has been gathered for test piles. Thus it is impossible to check any method's ability to predict historic load test results by using restrike blowcounts.

4) Significant increases or decreases in capacity may occur after the restriking.

These costs and problems involved with restriking preclude its present use for all or most production piles. Thus, any dynamic method intended for use with every pile must retain dependence on the end-of-driving blowcount. This requires that pile setup be accounted for in some other manner.

hould be noted that practically all evaluations of dynamic ods to date have used end-of-driving blowcounts to predict pile city. This capacity has been compared to the load test result out regard to setup. It might be argued that any time-ndent changes are considered indirectly through the customary ty factors that have been settled upon through the observation satisfactory" results. However, it is likely that these safety tors are higher than would be required if the time-dependent nges in each pile could be better quantified and included in the iction process.

should also be noted that the Gates formula included in this dy is different from the other methods in that it was derived m a statistical correlation between end of driving blowcounts load test results. Thus, it includes the effect of setup that urred before the load test <u>on the average for that group of es on which the formula development was based</u>. Because of the reme variation in amounts of setup that occurs, it is unlikely t this method of dealing with setup can be widely accurate.

this study, the dynamic methods were evaluated with and without nsideration of setup. For evaluation of the dynamic methods with nsideration of setup, the test pile failure loads obtained using e Van Der Veen method described above were divided by setup ctors to obtain estimated failure loads at end of driving. Pile pacities predicted by the dynamic methods (based on end-of-iving blowcounts) were compared to these estimated end of driving ilure loads. For evaluation of the dynamic methods without nsideration of setup, unmodified test pile failure loads were mpared to dynamic predictions.

e setup factor, SUF, was computed as follows for this study:

$$SUF = S(P_s) + 1.0(P_t)$$

where:

P_s = Fraction of total pile resistance coming from side friction

P_t = Fraction of total pile resistance coming from tip bearing

S = 1.0 if predominant side soil has high permeability (sand or gravel)

= 2.0 if predominant side soil is medium to stiff clay

= 3.0 if predominant side soil is soft to medium clay

= 4.0 if predominant side soil is very soft to soft clay

The fraction, P_s, of total resistance coming from side friction refers to "end-of-driving" conditions and was computed as follows:

P_s = 0.95 if the final blowcount is less than 3.5 times the average blowcount

= 0.75 if the final blowcount is between 3.5 and 4.0 times the average blowcount

= 0.50 if the final blowcount is greater than 4.0 times the average blowcount

The above setup factor calculation method was based on the following logic and study.

While the mechanism of setup is not well understood, it is generally believed that the increase in capacity for a pile driven in soft submerged clays is due to the dissipation of excess pore pressures that build up during driving. The thin film of pressurized water holding back the clay gradually retreats and allows the cohesion-accompanied clay to pack in. It was assumed in this study that only the side friction portion of pile capacity is subject to time dependent change. That is, it was assumed that the city at end of driving is constant throughout time. If the is hard clay, sand, gravel, or rock, this is probably a sumption. It is probably a good assumption for soft d clays also since excess pore pressure can effectively the axial compressive stresses that occur at the pile tip. ase, the capacity of a pile tip in soft submerged clay is ly less than five percent of the total capacity, and ions relating to the setup of that tip capacity are not very tial.

up factors for pile side friction were decided upon through of the literature. Lowery recommended setup factors of 3 es in soft clays, 2 for firm and stiff clays, and 1 for oils (4). Under the assumption that their wave equation h was accurately predicting end-of-driving capacities, and Lee give pile setup factors in the form of the "R" cited in the literature review above (16).

necessary to estimate the portion of the total end-of- capacity coming from side friction so the setup factor be applied to it. Two methods of doing this were developed. rst method, a soil mechanics approach, relied upon cohesion, on angle, and unit weight data for the surrounding soils. ations of side friction and tip bearing were performed using pha" method. Setup factors were applied to side friction to reduce them from long-term to end-of-driving values. ined compression values Q_u (twice the cohesion) in pounds per foot (psf) were divided by a setup factor equal to $2000/Q_u$, t less than 1.0. Thus, for a medium clay with Q_u equal 1000 he end of driving "effective" unconfined compression strength sumed to be 1000 psf divided by 2.0.

od of estimating percent skin friction that was not dependent nalysis of soils data was desired since this data may not be available. It was reasoned that this percentage might be related to the degree of change in the pile blowcount near the end of driving. A pile completely in soft clay generally experiences little change in blowcount and has approximately 100 percent skin friction. In this case the ratio of the final blowcount to the average blowcount is 1.0 . A high ratio of final blowcount to average blowcount generally indicates that the tip is seated in a stronger stratum than those along the piles side. In this case, a substantial percentage of the total pile capacity probably comes from tip bearing. The values of P_s given above were derived through study of the blowcount histories of several of the selected LADOTD test piles, together with the side friction percentages predicted using the soil mechanics approach. They work fairly well for the test piles studied but probably require considerable adjustment for use in other locations.

Safety Factors

The safety factors used with dynamic prediction methods range from 2.0 to 6.0. That is, the recommended allowable design load is the predicted ultimate load divided by a safety factor between 2.0 and 6.0. The wave equation and all of the formulas, except the Gates formula, theoretically predict the pile capacity at the end of driving. If it can be assumed that the pile either retains this capacity or increases in capacity, why would some of the formulas require a safety factor much greater than 2.0? It is either because a given method contains a systematic error that causes it to overpredict pile capacity on the average, or because there is such fluctuation in the accuracy of the method that to be conservative in almost all cases, a high safety factor is required. The fact is that the present status of safety factors for dynamic prediction methods is very confusing and without firm reasoning.

In this study an "adjusted" predicted ultimate capacity was computed for each dynamic method. This adjusted value equals the customary allowable load multiplied by 2.0. (The customary allowable load is the predicted ultimate load divided by the safety factor.) Both theoretical and adjusted ...ns of each dynamic method were compared to test pile

... Studies

...cuting any of the dynamic methods, there is often some ...nty about the proper values of the parameters. For ... the loss constant c in the Engineering News formula has ...ied by some foundation engineers in order to obtain a ...correlation between the formula predictions and selected ... results (e.g., the Wisconsin formula uses c = 0.2). In ... and other formulas, values of coefficient of restitution ...ion stiffness are often varied towards the same end. ...nts in any of the formula parameters affect the predicted ... versus capacity relation; however, some parameters have ... influence than others.

...dynamic methods studied, the wave equation requires by far ...st input variables. While the literature contains ...ndations on the values of many of these variables, there is ...considerable uncertainty about many of the inputs. Parameter ... were conducted for several of the WEAP87 input values to ...ne the effect of "reasonable" variations in these inputs on ...dicted blowcount versus capacity relation (31). Results of ...study indicated that reasonable variations in hammer ...ncy, coefficients of restitution, damping factors, and quake ... can have a very significant effect on this relation.

... Input Selected

... on how some of the WEAP87 input values were selected for ...roject are given below. Complete copies of the input data ...or the selected test piles are available by request from the ... Input for the other formulas is discussed in a later ...n.

All test piles selected were driven by hammers included in or closely matching hammers in the WEAP87 hammer data file. Thus it was not necessary to research such inputs as hammer efficiency, ram weight, hammer casing weight and stiffness, or other hammer related variables. It was assumed that the driving hammer conformed with WEAP87 table values.

The percent skin friction was selected on the basis of the soils information and ranged from 50 to 95 percent. Piles tipped in a hard stratum, as evidenced by the soil boring and a large increase in blowcount, were near the 50 percent level. Piles penetrating and tipped in soft-to-medium clay were near the 95 percent level. Regarding distribution of skin friction, only the built-in rectangular or triangular distributions were used. Piles in clay were generally assigned rectangular distributions, while piles in sand or in clay with strength increasing with depth were assigned triangular distributions. The percent of pile length receiving skin friction was based on the final ratio of pile embedment to pile length. Embedment and length were both contained in the test pile records.

Capblock stiffness (the capblock is a cushion between the ram and the pilecap) was based on recommendations in the WEAP87 user's manual. No information on capblock stiffness was found in the test pile records.

A pile cushion is used between the pilecap and concrete piles. It is normally wood four to eight inches thick and covers the entire area of the pile top. The input stiffness of this cushion often has a large influence on WEAP87 predictions. Unfortunately, none of the test pile records contained information on the pile cushion used. Values used were generally within the 1900 to 4500 kip-per-inch range.

d side quake values used were 0.1 inch; point and side soil factors (Smith damping) used were 0.15 seconds per foot.

on Ratios

ratios were calculated to compare the performances of the formulas. The R numbers are ratios of the measured pile ies to predicted pile capacities. There are three different ed" capacities for each pile; namely, 1) the actual maximum ad, 2) the Van der Veen Failure load (time of test), and 3) the Van der Veen load divided by the setup factor driving capacity). The two "predicted" capacities for each e 1) the theoretical predicted ultimate capacity of a formula and 2) twice the customary allowable load given by la. Thus there are six combinations of measured and ed capacities.

st two ratios, R1 and R2, use the maximum test load as the d pile capacity.

1 = Maximum Test Load / Formula Predicted Capacity

2 = Maximum Test Load / Formula Allowable times 2.0

the maximum test load is not consistently related to the pile e load, R1 and R2 were not used in the evaluation.

xt R ratios, R3 and R4, compare the failure load as ined by the Van der Veen method with the predictions of the c formula. These ratios constitute the "without eration of setup" comparison.

3 = Test Failure Load / Formula Predicted Capacity

= Test Failure Load / Formula Allowable times 2.0

st two R ratios attempt to account for setup between the time le was driven and the time of the load test.

R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity
R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

Formula allowables are the Formula predicted capacities divided by the following customary safety factors: ENR = 6.0, Hiley = 3.0, Gates = 3.0, Janbu = 4.5, PCUBC = 4.0, WEAP87 = 2.0. For the rare case in which the test load equals the Van der Veen failure load and has no setup, and the prediction method's theoretical safety factor is 2.0, all six ratios will be equal. If the dynamic method is also a "perfect" predictor, all ratios would be 1.0.

It was hoped that by examining and analyzing these ratios for many load tests, the best prediction method for the state of Louisiana would become evident. The mean and coefficient of variation (cov) of the ratios were calculated for several groupings of the selected test piles. Low values of cov (cov = standard deviation divided by the mean) indicate that the dynamic method is consistent in predicting pile capacities equal to some constant multiple times the load test value. Systematic errors in the predictions, indicated by a mean different from unity, can easily be "factored out." Thus a low cov is the primary focus when selecting a good prediction method.

For each of the database test piles, it was necessary to compute the previously described prediction ratios and then compute the ratio means and covs for various groups of the test piles. A FORTRAN computer program, PILCAP, was written to interactively prompt for file numbers of a desired group of test piles (or the name of another file that contains these file numbers), open and read the appropriate LATP.xxx files, calculate the prediction ratios for each pile and each dynamic method, and compute the ratio means and covs for that group. A description of the program is given below. A listing of the program is available upon request from the authors.

```
    NUMBER OF PILING TO BE ANALYZED =   3


     ANALYSIS FOR PILE    1
PILE CATALOGUE NUMBER =091
FILE NAME = LATP.091
STATE =   LOUISIANA
PARISH =    EAST FELICIANA
      CLINTON-OLIVE BRANCH HWY, LA-67
      TEST PILE # 1
PROJECT NO.=    60-03-12
DATE OF DRIVING =    9-9-81
DATE OF TESTING =    10-6-81
PILE DESCRIPTION:    16" PRECAST CONCRETE, L=50'
PILE LENGTH,FT =     50.0     PILE EMBEDDMENT,FT =     34.0
PILE TIP BEARING AREA,SQ IN =   256.00
PILE BUTT AREA,SQ IN =   256.00
DEPTH TO WATER TABLE,FT =      0.0     GRD ELEV =   199.9
FINAL BLOW COUNT,BLOWS PER FT =   33.0
AVG BLOW COUNT LAST 5 FT,BLOWS/FT =    26.4
AVG BLOW COUNT ENTIRE EMBED =    47.0
HAMMER DESCRIPTION =    VULCAN NO. 1
SINGLE ACTING AIR/STEAM HAMMER
HAMMER NUMBER FROM WEAP86 TABLE =         204
RAM WEIGHT,KIPS = 5.00    HAMMER RATED ENERGY,FT-KIPS = 15.00
HAMMER ENERGY EFFIC RATIO = 0.67   HAMMER TOT WT,KIPS =  10.00
PILE DESIGN LOAD,TONS =     57.4   MAX TEST LOAD,TONS =   143.5
DURATION OF MAXIMUM LOAD,HOURS =    2.00
TOTAL PILE DEFLECTION AT MAX LOAD,IN =   0.190
PERMANENT DEFLECTION,IN =   0.094
PILE DEFL,IN, AT 50% MAX LOAD =   0.030
PILE DEFL,IN, AT 75% MAX LOAD =   0.080
EST RATIO TEST LOAD TO FAILURE LOAD    0.95
EST PERCENT SKIN FRICTION AT END OF DRIVING   70.00
EST SETUP FACTOR FROM END OF DRIVING TO TIME OF TESTING    1.00
PREDOMINANT SIDE SOIL IS SAND
NUMBER OF PILE X-SECTIONS INPUT =   1
SECTION   1  AREA,SQ IN =  256.00
SIDE FRICT PERIM,IN =    64.0  MOD OF ELAS,KSI = 3640.00
UNIT WT,PCF =   150.0  DIST,FT,BELOW TOP =    0.00
NUMBER OF SOIL LAYERS =   2
LAYER   1       STIFF SANDY CLAY
THICKNESS,FT =      6.7  TOTAL UNIT WT,PCF =   132.0
ANGLE OF FRICTION,DEGREES =      0.0
UNCONFINED COMPRESSIVE STRENGTH,PSF =  2560.0
WATER CONTENT % =    16.0
```

Figure 5. PILCAP Output

MIT = 24.0 PLASTICITY INDEX = 11.0
CLAYEY SILTY SAND
,FT = 49.0 TOTAL UNIT WT,PCF = 120.0
FRICTION,DEGREES = 30.0
ED COMPRESSIVE STRENGTH,PSF = 0.0
NTENT % = 20.0
IMIT = 0.0 PLASTICITY INDEX = 0.0
WEIGHT,KIPS = 0.96
STIFFNESS,K/IN = 4591. CUSHION STIFFNESS = 1920.
,KIPS = 0.00 ANVIL COEF OF RESTITUTION= 0.000
COR= 0.800 PILE TOP COR= 1.000
COR = 0.500
COR FOR FORMULAS = 0.800
DE DAMPING FACTOR,SEC/FT = 0.05
MPING FACTOR = 0.15
CTION QUAKE,IN = 0.10 POINT QUAKE = 0.13
CTION DISTRIBUTION TYPE NUMBER = 3
LOAD,KIPS,PREDICTED BY WEAP86 = 99.000
LOAD,KIPS, WEAP WITH DEFAULT INPUT= 106.000
TEST LOAD,TONS= 143.50
URE LOAD,TONS,AT TESTING TIME = 150.89
LURE LOAD,TONS,AT END OF DRIVING = 150.89
D PORTION SKIN FRICTION AT EOD BASED ON
N BETWEEN AVG AND FINAL BLOW COUNTS = 0.950
CTOR BASED ON PREDOM SIDE SOIL AND ESTIM
SKIN FRICTION = 1.000
US PILE CAP WT,KIPS = 14.29
,KIPS = 0.932E+06
IDE FRICT TO TOTAL BASED ON SOIL PROFILE
TERM = 0.696 END OF DRIVING = 0.696
TED END OF DRIVING ULTIMATE CAP,TONS = 99.30
TED LONG TERM ULT CAP,TONS = 99.30

PRED LT,T	NOM SF	CUST ALLOW,T	ADJ P ULT	PPSF	ADJ PPSF
4.1	6.000	32.4	64.7	4.435	4.664
1.8	3.000	27.3	54.5	5.266	5.537
5.2	3.000	21.7	43.4	6.605	6.946
3.3	4.500	14.1	28.1	10.209	10.735
7.4	4.000	11.8	23.7	12.120	12.744
9.5	2.000	24.8	49.5	5.798	6.097
99.3	2.000	49.7	99.3	2.890	3.039
27.9	6.000	4.7	9.3	30.827	32.415
53.0	2.000	26.5	53.0	5.415	5.694

Figure 5. (cont)

MAXIMUM TEST LOAD,TONS= 143.50
EST FAILURE LOAD,TONS,AT TESTING TIME = 150.89
EST FAILURE LOAD,TONS,AT END OF DRIVING = 150.89
ESTIMATED PORTION SKIN FRICTION AT EOD BASED ON
RELATION BETWEEN AVG AND FINAL BLOW COUNTS = .70

METHOD	R1	*	R2	*	R3	*	R4	*	R5	*	R6
	*		*		*		*		*		*
ENR*	0.739	*	2.218	*	0.777	*	2.332	*	0.777	*	2.332
HIL*	1.755	*	2.633	*	1.846	*	2.768	*	1.846	*	2.768
GAT*	2.202	*	3.303	*	2.315	*	3.473	*	2.315	*	3.473
JAN*	2.269	*	5.105	*	2.386	*	5.368	*	2.386	*	5.368
PCU*	3.030	*	6.060	*	3.186	*	6.372	*	3.186	*	6.372
WP1*	2.899	*	2.899	*	3.048	*	3.048	*	3.048	*	3.048
SLG*	1.445	*	1.445	*	1.520	*	1.520	*	1.520	*	1.520
EMD*	5.138	*	15.413	*	5.403	*	16.208	*	5.403	*	16.208
WDF*	2.708	*	2.708	*	2.847	*	2.847	*	2.847	*	2.847

ENR=Engineering News Record, HIL = Hiley, GAT = Gates, JAN = Janbu, PCU = Pacific Coast Uniform Building Code, WP1 = WEAP87, SLG = Soil Boring Method, EMD = Modified ENR, WDF = WEAP87 with default input.

PRED ULT,T = ULTIMATE LOAD,TONS, PREDICTED BY METHOD
NOM SF = SAFETY FACTOR GENERALLY USED WITH METHOD
CUST ALLOW,T = CUSTOMARY ALLOWABLE LOAD,TONS = PRED ULT/NOM SF
ADJ P ULT = ADJUSTED PREDICTED ULTIMATE LOAD,TONS
= CUSTOMARY ALLOWABLE * 2.0
PPSF = PRODUCTION PILE SAFETY FACTOR
= MAX TEST LOAD/CUSTOMARY ALLOWABLE
ADJ PPSF = PPSF/EST RATIO MAX TEST LD TO FAILURE LD AT TIME OF TEST
R1 = MAX TEST LOAD/PREDICTED ULTIMATE LOAD
R2 = MAX TEST LOAD/ADJUSTED PREDICTED ULT LD
R3 = EST FAILURE LOAD AT TIME OF TEST/PRED ULT LD
R4 = EST FAIL LD AT TIME OF TEST/ADJ PRED ULT LD
R5 = EST FAILURE LOAD AT END OF DRIVING/PRED ULT LD
R6 = EST FAIL LD AT EOD/ADJ PRED ULT LD

Figure 5. (cont)

ut for files 092 and 093 similar, omitted for brevity)

MARY STATISTICS

R1			R2			R3		
EAN	COV	SD	MEAN	COV	SD	MEAN	COV	SD
0.558	0.28	0.16	1.674	0.28	0.47	0.583	0.29	0.17
1.494	0.25	0.37	2.241	0.25	0.56	1.552	0.23	0.36
2.236	0.09	0.19	3.354	0.09	0.29	2.327	0.05	0.11
1.887	0.22	0.41	4.247	0.22	0.93	1.964	0.21	0.41
2.429	0.24	0.57	4.858	0.24	1.15	2.530	0.23	0.59
2.555	0.14	0.35	2.555	0.14	0.35	2.662	0.13	0.35
1.007	0.42	0.42	1.007	0.42	0.42	1.047	0.42	0.44
2.741	0.79	2.17	8.222	0.79	6.50	2.885	0.79	2.29
2.315	0.16	0.37	2.315	0.16	0.37	2.412	0.16	0.38

R4			R5			R6		
EAN	COV	SD	MEAN	COV	SD	MEAN	COV	SD
1.748	0.29	0.51	0.398	0.83	0.33	1.194	0.83	0.99
2.328	0.23	0.54	1.003	0.73	0.73	1.504	0.73	1.10
3.491	0.05	0.17	1.433	0.54	0.78	2.149	0.54	1.17
4.419	0.21	0.93	1.286	0.74	0.95	2.892	0.74	2.15
5.059	0.23	1.19	1.682	0.78	1.31	3.364	0.78	2.62
2.662	0.13	0.35	1.717	0.68	1.17	1.717	0.68	1.17
1.047	0.42	0.44	0.729	0.94	0.68	0.729	0.94	0.68
8.654	0.79	6.87	2.305	1.18	2.72	6.914	1.18	8.15
2.412	0.16	0.38	1.572	0.71	1.11	1.572	0.71	1.11

RAW DATA OUTPUT, SHORT TONS

V = VAN DER VEEN LOAD TEST ULT LOAD
D = VDV LOAD REDUCED TO END OF DRIVING BY EST SETUP
MBERS REFER TO METHODS AS NUMBERED ABOVE

VDV	EOD	1	.333*1	2	3	4	5	6	7	8	9
151.	151.	194.	65.	82.	65.	63.	47.	50.	99.	28.	53.
288.	99.	600.	200.	174.	118.	148.	122.	113.	297.	311.	127.
341.	124.	492.	164.	210.	109.	155.	119.	102.	370.	104.	114.

Figure 5. (cont)

The constant k_2 represents the elastic compression of the pile and is a function of the pile load and its axial stiffness distribution. For non-prismatic piles the average stiffness is used. Since the pile load is not initially known, the Hiley formula requires several cycles in which pile load is estimated, elastic compression computed, pile load computed, elastic compression recomputed, etc., until the computed pile load equals the value of the pile load on which the elastic compression was based.

The Gates and Janbu formulas do not require further explanation. PILCAP executes them exactly as described previously. The Janbu formula uses an average stiffness for non-prismatic piles. The PCUBC formula requires iteration similar to that required by the Hiley formula. Again, an average axial stiffness is used for non-prismatic piles.

WEAP87 - PILCAP does not perform wave equation predictions; it simply outputs the input results of separate WEAP87 runs. For each test pile, the WEAP87 program was run with estimated pile capacities until the accompanying blowcount of one of the capacities matched the actual final blowcount of the test pile. The matching capacity was input to the test pile data file.

STUDY OF DYNAMIC PILE TESTING

Methods based on the dynamic performance of a driven pile also include in-place measurements of the induced wave during driving, i.e., the pile driving analyzer (PDA). LADOTD has recently acquired limited experience with the PDA. It was used in the I-220 Cross Lake project and is currently being used in the I-310 Luling Bridge Approach. The capacities measured in static load tests of piles driven in the I-220 Cross Lake project were much less than those predicted using the ENR formula. Since the piling were driven using a follower, it was suspected that significant losses of energy occurred between the pile and the hammer as a result. Thus the PDA was used to check pile capacity and to estimate the hammer energy delivered to the pile. The incentive to use the PDA for the I-310 Luling Bridge Approach was the difficulty or inability to conduct static load tests with the required "end-on" construction method (i.e., pile driving from the structure as it is being built, generally and in this case for environmental protection reasons). In the beginning of the project, the LADOTD conducted an in-house comparison of in-place test methods with pile load tests. Based on the results, the LADOTD is using the PDA in lieu of static tests for construction of the I-310 approach to the Luling Bridge.

When using either a pile driving formula or the wave equation, a great amount of uncertainty accompanies the estimation of the energy delivered to the pile by the hammer. The PDA is a dynamic test that directly measures the force and acceleration at the top of an instrumented pile during driving. This eliminates the need to assume certain input values required to model the hammer and other accessories. With a field computer and appropriate available software, the measured information can be used as input to a pile capacity calculation based on single force balance theory (32). This approach is known as the CASE method. Assuming a uniform elastic pile and using wave propagation theory, the total soil resistance R acting during driving is:

$$R = 1/2[F(t_1) + F(t_2)] + mc/2L[v(t_1) - v(t_2)]$$

where:

$F(t)$ = measured force as function of time

$v(t)$ = velocity of the pile top

t_1 = a selected time during the blow

t_2 = $t_1 + 2L/c$

m = pile mass

c = wave transmission speed of pile

The above total soil resistance is the sum of a static S (displacement dependent) and a dynamic D (velocity dependent) component:

$$R = S + D$$

The static resistance S is determined by subtracting the damping force D from the total resistance. The damping force is approximated as:

$$D = J \times v_{toe}$$

where:

J = damping constant

v_{toe} = pile toe velocity

= $2v_{top} - (L/mc)R$

v_{top} = pile top velocity at time t_1

The measured force and acceleration can also be used in a wave equation analysis for predicting the pile's static capacity (12,13). Using the wave equation, a predictor-corrector numerical integration is performed with the known values of acceleration as boundary conditions. Soil resistance properties are adjusted until the computed output force equals the measured force (33,34). The computer program CAPWAP (12) iteratively evaluates soil resistance and damping values along the pile to determine the conditions required to produce the actual dynamic measurements.

Using the results of the CAPWAP analysis, the pile-soil model can be analyzed further in a "simulated static test." The pile is loaded incrementally, and displacements at the pile head and along the shaft are computed. A load versus displacement graph is produced. Applications of the PDA also include an analysis of the integrity of the driven pile (35).

<u>I-310 Advance Test Pile Program</u> - The comparative study of dynamic methods for the I-310 Luling Bridge North Approach involved seven 84-foot-long prestressed concrete test piles of various sizes and

d in-place dynamic tests of those piles. The "Special ns" of the construction contract for this job required the or to submit a wave equation analysis of all test piles th the approved hammer to the Bridge Design Engineer prior ding work. The piles were driven with a Delmag D46-23 cting diesel hammer to an 80-foot tip penetration. aluations of pile capacity, driving stresses, and hammer nce were conducted using the PDA and the CAPWAP method. urements were made during initial driving and for a series ikes conducted after specified time intervals for all of piles. All test piles except one were statically loaded ure at a time interval of approximately 14 days from the trike test. A quick-load test was used for testing the ile capacity (36). Results of the test sequences used for static and restrike measurements made it possible to the effects of time-dependent changes on the soil and pile capacities. A study of the results of this test gram is given in the next section.

ENT OF IMPLEMENTATION SOFTWARE

he tasks of the project was the creation of a field use allow convenient application of a superior dynamic on method during production pile driving. Following the on of the dynamic formulas and wave equation, two formulas ave equation were selected for inclusion in the field use . The intent is that the software developed can be on a mobile microcomputer similar to an IBM AT. A description of the software and hardware requirements is the next section.

ANALYSIS OF DATA

REVIEW OF CURRENT PRACTICE

An evaluation of existing specifications and the current practice used in selecting pile types and length as well as those for monitoring pile installation have been under consideration by LADOTD. Other state transportation departments are conducting similar evaluations. In recent years several states have completed this task and implemented new methods.

In Louisiana production piles are furnished by the contractor in accordance with an itemized list established by the LADOTD engineer (1). The list includes the number, size and type, and location of all permanent piles. The type and lengths (and tip elevations) of the permanent piling are generally based on results of a previously conducted load test of a similar pile at the jobsite. LADOTD specifications state that "the order length may be revised by the engineer when driving conditions deviate from test pile results." The Louisiana Standard Specifications (1) also state that "if the safe bearing capacity of permanent piles is to be determined by formulas," the ENR Formula "shall be used as a guide and shall be correlated with the test pile driving and loading data."

Other state transportation departments have recently reviewed or are reviewing their pile driving programs. Included in a 1985 Washington State Department of Transportation study by Fragazy et al. (19) is a survey of the current practice of state transportation departments with respect to use of dynamic formulas, the wave equation, and the pile analyzer. A letter was sent to departments in each state and the District of Columbia requesting information on the method(s) used for construction control of pile driving. Of the 34 states responding, 21 states indicated that they use the Engineering News formula in its original or modified with no other dynamic formula. Several states indicated a ...ent switch to wave equation analyses due to the resulting ...ease in accuracy. Comparative studies of pile driving formulas ...ducted by some states were found to be "either quite old...or ...ormal." A few states had previously conducted a comparative ...dy of pile load tests with formulas and/or the wave equation. ...ur states were conducting such a study at the time of the survey, ...d one was considering such an investigation in the near future. ...hough their study was not complete, Pennsylvania transportation ...gineers indicated that they were finding that the wave equation ...d pile analyzer underpredict pile capacity if the pile does not ...perience relaxation.

...n states responding to the Washington State survey indicated they ...e the wave equation. Only two states indicated regular use of ...e pile analyzer, but they were very satisfied with it. It was ...ated in the report that although "these methods clearly are more ...ifficult to implement, and require more highly trained personnel, ...he intermediate step, using a more sophisticated equation, does ...ot seem to have been considered."

...he Washington State Department of Transportation procedure for ...onstruction control of pile foundations, as presented in the ...ragazy et al. report (20), is similar to that used by many other ...tates. The Engineering News Record is used for estimation of pile ...apacity and construction control of small pile driving jobs. This ...ncludes the majority of pile driving projects. For interstate ...onstruction and larger projects, the wave equation and pile ...nalyzer are used. Outside contractors are used when the pile ...nalyzer is employed since the Washington DOT does not currently ...ossess the in-house capability for this dynamic test.

...n the survey of the Washington State study, the Wisconsin DOT ...eported that the Wisconsin (modified ENR) formula and dynamic pile analyzer are used in construction control. The Wisconsin formula is a modified ENR as follows:

$$P_{allow} = 2WH/(S + 0.2)$$

where:

P_{allow} = allowable bearing value, lb.
W = ram weight, lb.
H = height of ram fall, ft.
S = penetration per blow, in.

The Mississippi DOT also uses the above expression (17).

The New York DOT (37) uses the wave equation analysis (WEAP), the dynamic pile analyzer (with the CASE method and CAPWAP), and occasionally a static test to estimate and verify pile resistance. WEAP is used on all pile projects during the design process and construction. In the design phase, WEAP is used to analyze potential for overstressing the pile by driving, specifying limits on hammer size or type, or specifying thicknesses and/or types of pile cushions. The most common and routine use of the wave equation is in construction. New York DOT requires contractors to submit the proposed hammer and pile system for approval. Using WEAP, the contractor's hammer is checked for its ability to drive the pile without overstress. Also, a blowcount versus capacity chart is prepared for inspector use.

The New York DOT utilizes the dynamic pile analyzer to determine in-place capacity, monitor stresses, measure hammer performance and pile integrity, and determine the length of existing embedded piles and sheeting. The pile analyzer is used on special projects that have unique soil conditions or where soil parameters are difficult to determine. This test is also conducted where soil conditions are different from those assumed in the design, and to troubleshoot pile driving problems. The CAPWAP analysis permits refinement of damping and quake parameters for the soil and increases the nce in the predicted design capacities. New York DOT s piles to be driven to a predicted ultimate capacity of he allowable load. As a result of increased confidence in amic procedures, a lower safety factor is occasionally used truction. Static tests are normally used only on large s with high capacity piles.

io DOT initially estimates the pile type and lengths with mpling and testing information (21). Pay items for static sts and dynamic load tests are established. The contractor n the responsibility for determining the lengths of piles to red. All piles not driven to refusal or bedrock must be ed to a penetration that satisfies the ENR formula. After driving experiences at the site, a decision is made between or dynamic tests (or both). Static load tests are generally ed only for relatively large projects. The Ohio DOT has ilizing dynamic tests since the mid-1970s. A Pile Driving r became the property of the Ohio DOT at the conclusion of arch project on pile capacity at Case Western Reserve ity. Since that time, the usage of and reliance on dynamic g has greatly increased relative to static testing. The wave on has been used on selected projects.

rth Carolina DOT (38) found that the wave equation analysis des a means of not only increasing the Engineer's confidence he required capacity is achieved but that the pile is not verstressed and that the pile driving hammer is capable of g the pile to the desired depth." Since 1977, the WEAP r program has been used unofficially on pile projects to perience. Methods of correlating the wave equation computer with pile load tests were practiced on at least four s per year. The wave equation has been used since 1980 to r pile driving. However, the ENR formula is still generally sed on bridges with steel H-piles driven to rock. As part design, a wave equation analysis is performed using damping parameters and a side friction distribution corresponding to static bearing capacity computations. This gives an estimate of the driving stresses and tests the ability of the hammer to drive the pile to the required depth. Specifications on the hammer, cap block, and cushion material are submitted by the contractor for approval two weeks prior to driving. The contractor is required to conduct load tests and to restrike the test pile. Production pile lengths are specified based on the load test results. After determining the test pile capacity and restriking, a wave analysis is conducted. The soil damping parameters are varied until the wave analysis produces a capacity equal to that measured in the pile test. The soil quake of 0.1 remains unchanged. The ratio of side resistance to total capacity from the static analysis is used as input. Ultimately, a table or bearing graph of capacity versus blowcount foot is generated. Field control of production pile driving is secured by providing bearing graphs to the resident engineer and order lengths to the contractor. It is expected that the acquisition of a Pile Driving Analyzer will further refine the North Carolina DOT construction control and reduce the number of pile load tests required.

In the FHWA Demonstration Project No. 66 manual (39), determination of pile load capacity during installation using dynamic formulas is cited as being unreliable and having large built-in factors of safety. Thus it is recommended that dynamic formulas for construction control be eliminated as experience is gained with the wave equation analysis. The wave equation analysis coupled with dynamic monitoring is recommended for construction control. Pile load tests are recommended for big jobs to verify the predictions made by the wave equation and in-place dynamic measurements. The safety factors recommended for the various methods used in quality control of construction are given as the following:

Construction Control Method	Recommended Safety Factor
Static Load Test	2
Dynamic Measurement coupled with Wave Equation Analysis	2.5
Wave Equation Analysis	3

PERFORMANCE OF DYNAMIC FORMULAS WITH LOUISIANA TEST PILES

EVALUATION OF CANDIDATE METHODS

The database test piles were grouped in certain categoties for the purpose of computing the ratio means and covs. In addition to the group of "all" test piles, there is a practically infinite number of subsets possible. The hope in studying any of these subsets was that the means and covs of the prediction ratios would indicate one or more of the dynamic methods to be significantly more accurate for that subset than for the entire group. The following groups were selected:

Square concrete piles
Timber piles
Piles driven with single acting air/steam hammers
Piles bearing in clay
Piles bearing in sand.

The specific pile numbers in each group are given in Appendix A .

As defined in the previous chapter:

R3 = Test Failure Load / Formula Predicted Capacity

R4 = Test Failure Load / Formula Allowable times 2.0

R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity

R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

Logically, R5 and R6 are the "best" ratios to examine for all methods except the Gates formula. If setup effects had been correctly calculated, if the Van der Veen failure load was the "appropriate" one for correlation with the final blowcount, and if the particular dynamic method used an accurate structural model, then the mean of R5 would be 1.0 and the cov would be zero. If the predicted ultimate capacity is taken as twice the allowable load, R6 should be examined.

For the Gates formula, R3 and R4 should be the more logical measures, since setup effects are already included (on the average) by correlating blowcounts with load test results.

Table 6 represents 56 square concrete piles. The ENR mean predicted pile capacity is close to the mean load test value if its predicted capacities are taken as twice the customary allowables (R6) instead of the theoretical six times (R5). In contrast, the Hiley formula performed better if its theoretical ultimate is used (3 times customary allowable). The Gates, Janbu, and PCUBC also performed better when theoretical capacities were used instead of capacities adjusted to twice the customary allowables. For dynamic methods with customary safety factors of 2.0, such as the WEAP87, there is no difference between R5 and R6.

In comparing R3 with R5 or R4 with R6, it is evident that the proposed setup factors brought the ratio means closer to unity for all cases except R3 to R5 for the ENR. This indicates a beneficial average performance of the setup factors.

The cov values for all methods are very high, indicating poor performance of the methods for individual cases. Comparing the cov values for R3 and R4 with those of R5 and R6, it can be seen that the setup factors being used did not greatly improve individual performances. For the Hiley formula, the setup factors actually

TABLE 6

SQUARE CONCRETE PILES: PILE FORMULA CAPACITIES

R - Ratios*

	Mean		COV	Mean		COV
Method	R3	R4	R3 and R4	R5	R6	R5 and R6
ENR	0.599	1.797	0.61	0.348	1.044	0.60
Hiley	1.598	2.398	0.50	0.953	1.429	0.53
Gates	2.239	3.359	0.46	1.361	2.041	0.54
Janbu	2.186	4.918	0.55	1.264	2.844	0.51
PCUBC	2.972	5.944	0.52	1.730	3.460	0.50
WEAP87	2.605	2.605	0.64	1.461	1.461	0.58

* R3 = Test Failure Load / Formula Predicted Capacity
R4 = Test Failure Load / Formula Allowable times 2.0
R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity
R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

worsened the agreement between measured and predicted capacities. R5 and R6 information for the Gates formula should be ignored.

Table 7 represents twelve timber piles. As indicated by the lower cov values, all methods performed much better for this group of piles. Most other comments for Table 6 also apply to Table 7.

Table 8 represents 61 piles driven with single-acting air/steam hammers. Most of the piles in this grouping are also covered by Table 6. Comparing the cov values of R3 and R4 with those of R5 and R6, it can be seen that the proposed setup factors performed better for this pile group.

Table 9 represents 43 piles bearing in clay. Again, most of these piles are also in Tables 6 and 8. Performance of the setup factors was mixed. Table 10 represents 12 piles bearing in sand. Performance of the setup factors was very poor.

In summary, this study did not indicate any of the candidate dynamic formulas to be greatly superior to the others at predicting the results of Louisiana load tests. It is the authors' opinion that further study of additional load test results would lead to the same conclusion, if those load test results are similar in information to the ones studied (as most are). The real need is for a higher quality database within which more of the test pile characteristics are measured and recorded.

The results of this and other studies indicate that for the ENR, the pile capacity is closer to being twice the customary allowable load; using a predicted capacity six times the customary allowable (the theoretical value) results in large overpredictions in virtually every case. In contrast, for the other formulas with customary safety factors greater than 2.0, a better estimate of pile capacity is obtained using "unadjusted" theoretical values.

TABLE 7

TIMBER PILES: PILE FORMULA CAPACITIES

R - Ratios*

Mean		COV	Mean		COV
R3	R4	R3 and R4	R5	R6	R5 and R6
0.436	1.308	0.30	0.389	1.168	0.30
1.429	2.143	0.24	1.280	1.920	0.24
1.643	2.464	0.24	1.471	2.206	0.24
1.670	3.758	0.24	1.497	3.369	0.25
2.056	4.112	0.24	1.841	3.683	0.25
1.728	1.728	0.24	1.553	1.553	0.26

Test Failure Load / Formula Predicted Capacity
st Failure Load / Formula Allowable times 2.0
st Failure Ld divided by Setup / Formula Predicted Capacity
st Failure Ld divided by Setup / Formula Allowable times 2.0

TABLE 8
PILES DRIVEN WITH SINGLE ACTING AIR/STEAM HAMMERS: PILE FORMULA CAPACITIES

R - Ratios*

Method	Mean R3	Mean R4	COV R3 and R4	Mean R5	Mean R6	COV R5 and R6
ENR	0.650	1.949	0.60	0.391	1.172	0.50
Hiley	1.727	2.591	0.42	1.084	1.626	0.42
Gates	2.251	3.377	0.43	1.438	2.157	0.49
Janbu	2.278	5.126	0.52	1.381	3.106	0.44
PCUBC	2.954	5.908	0.50	1.805	3.609	0.44
WEAP87	2.685	2.685	0.63	1.573	1.573	0.49

* R3 = Test Failure Load / Formula Predicted Capacity
R4 = Test Failure Load / Formula Allowable times 2.0
R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity
R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

TABLE 9

PILES BEARING IN CLAY: PILE FORMULA CAPACITIES

R - Ratios*

Method	Mean R3	Mean R4	COV R3 and R4	Mean R5	Mean R6	COV R5 and R6
ENR	0.619	1.858	0.60	0.360	1.079	0.53
Hiley	1.609	2.414	0.42	0.993	1.489	0.50
Gates	2.032	3.048	0.46	1.259	1.888	0.54
Janbu	2.204	4.959	0.49	1.298	2.921	0.48
PCUBC	2.828	5.657	0.47	1.678	3.355	0.49
WEAP87	2.728	2.728	0.60	1.529	1.529	0.48

* R3 = Test Failure Load / Formula Predicted Capacity

R4 = Test Failure Load / Formula Allowable times 2.0

R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity

R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

TABLE 10
PILES BEARING IN SAND: PILE FORMULA CAPACITIES

R - Ratios

Method	Mean R3	Mean R4	COV R3 and R4	Mean R5	Mean R6	COV R5 and R6
ENR	0.469	1.408	0.56	0.367	1.102	0.79
Hiley	1.498	2.247	0.58	1.091	1.636	0.59
Gates	2.450	3.676	0.63	1.925	2.887	0.85
Janbu	1.873	4.213	0.47	1.421	3.198	0.60
PCUBC	2.473	4.945	0.47	1.822	3.644	0.53
WEAP87	1.977	1.977	0.61	1.563	1.563	0.83

* R3 = Test Failure Load / Formula Predicted Capacity
R4 = Test Failure Load / Formula Allowable times 2.0
R5 = Test Failure Ld divided by Setup / Formula Predicted Capacit
R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.

setup factors used to reduce test loads to end of driving
cities greatly improved the accuracy of formula predictions on average (i.e., formulas were much closer at predicting end of ving capacities, as they should logically be). As evidenced by parisons of R6 to R4 for the ENR and R5 to R3 for the other hods, ratio means were much closer to unity for all pile groups. thermore, none of the methods became "unconservative mean dictors" even for the end-of-driving capacities (i.e., end-of-ving capacities were on average higher than the dynamic dictions). These results and the logic behind the approach mpt the authors to conclude that the setup factor approach uld be retained. However, the only slight improvement in ratio s indicates that more work is required to improve the individual formance of the setup factors.

comparison of this study's ratio means and covs to the means and s of similarly calculated ratios from two other recently sented studies (<u>17</u>,<u>19</u>) shows that the results are very similar. mean value for "Test Failure Load \ ENR allowable * 2.0" was ven as 1.22 by Briaud (<u>17</u>); a value of 1.47 can be calculated milarly for the pile data given by Fragazy (<u>19</u>). The covs for ove defined ratio were 0.46 and 0.54, respectively. Other mulas investigated also had high covs (<u>18</u>).

VIEW OF I-310 TEST PILE PROGRAM

I-310 North Approach to the Luling Bridge is an elevate adway that crosses environmentally sensitive wetlands. In orde confine the construction activity and to cause the leas sturbance to this area, an end-on construction technique wa lected. In this method, the elevated roadway is advanced b ilding off the previously completed section. The customar ocedure for performing static load tests of roadway bent pile s not permitted for environmental reasons. Thus, a higher tha usual reliance on dynamic methods was required. The me considered for monitoring the pile performance and load included the pile driving analyzer (PDA) and the Shock Me (Transient Dynamic Response Testing Technique). A correlation load test results with electronic cone penetrometer tests (EC was also considered. The ECPT soundings were to be used as a me to establish pile tip elevations at the bent locations.

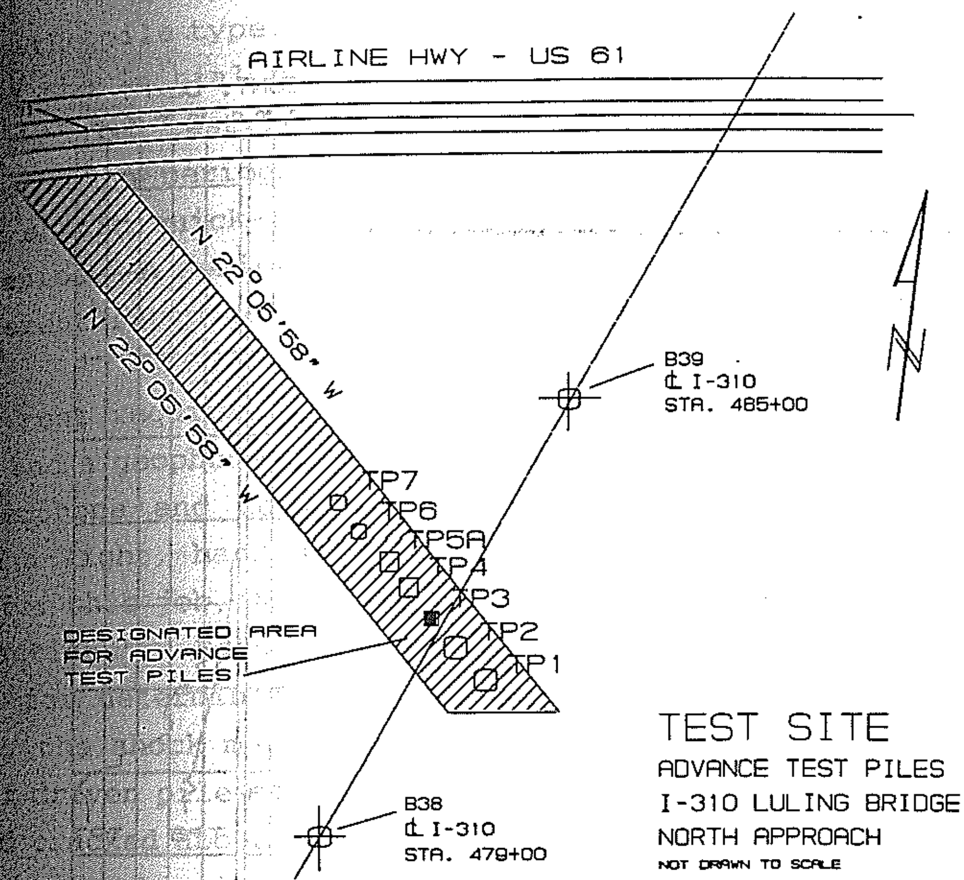
In order to verify the proposed pile evaluation techniques, advance test pile program was conducted at a nearby accessib site. The test site was located in St. Charles Parish at th intersection of US 61 and I-310, North Approach to the Luli Bridge. The location and arrangement of the piles are shown i Figure 6. The approximate locations of the two soil borings taken along the I-310 centerline and near the test site are also shown and are designated as B38 and B39. Figure 7 shows the boring logs and driving records for the placement of all test piles. The soil profile consists of soft to stiff gray clays from the surface to an approximate elevation of -80 ft., where a fine silty-sand occurs. All piles were installed at modest blowcounts.

Seven prestressed concrete piles were driven as part of a preliminary testing program. Each pile had a total length of 84 ft.; two piles were spliced (54 ft. bottom and 30 ft. top). Test Piles 1 and 2 (TP1, TP2) are 54 in. x 5 in. cylinders, Test Pile 3 (TP3) is 24 in. square, Test Piles 4 and 5 (TP4, TP5) are 30 in. square, and Test Piles 6 and 7 (TP6, TP7) are 36 in. x 5 in. cylinders. TP5 and TP7 were spliced. The piling were driven with a Delmag D46-23 open end diesel hammer. This hammer has a ram weight of 10.14 kips and a rated energy of 107.18 kip-ft. A reduced fuel pump setting for the hammer was recommended to limit tension during easy driving. The hammer cushion consisted of laminated aluminum and Conbest. The pile cushion was made of layers of plywood and/or red oak; the area and thickness was

ed. Th
e and I
the Sho
A corre
eter tes
be used a
ations.

on techni
nearby a
es Paris
ch to th
les are
oil borin
e are al
s the bor
piles.
the surf
ilty-sand

as par
otal leng
) ft. top
ders, Tes
TP5) ar
36 in.
were dri
hammer ha
07.18 kip
ommended
ion consi
nion was
hickness



ure 6. Test Site: I-310 Advance Test Pile Study

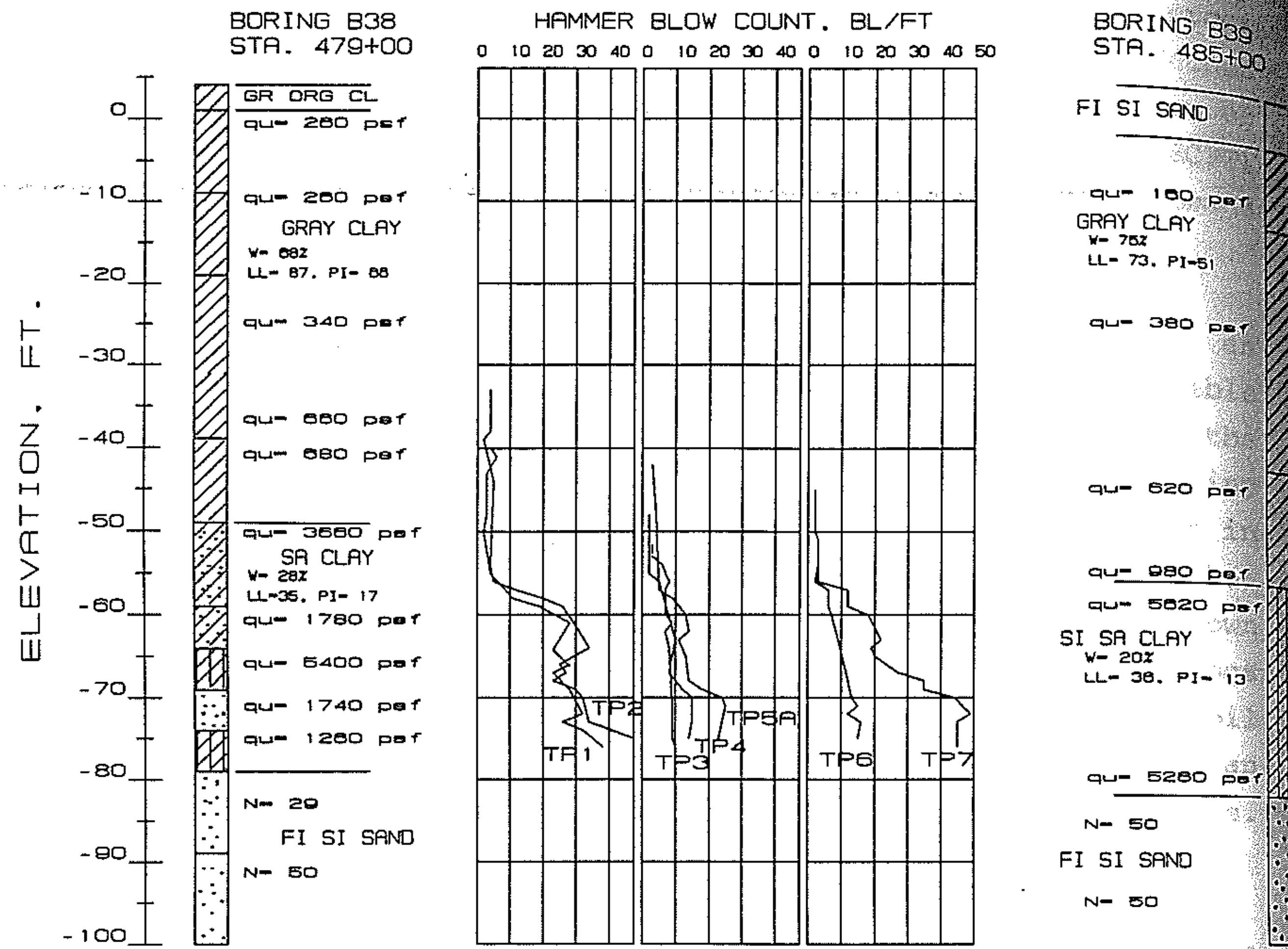


Figure 7. Bore Hole/Pile Driving Record

on the pile type. Dynamic measurements with the PDA were ing the initial pile installation and subsequently in a scheduled restrikes. The restrike tests were conducted ious setup periods that varied from 1 to 22 days. For on, static quick-load tests were conducted at the end of p period. A series of static load tests were conducted on ifferent setup periods.

ck" test requires placement of a small loadcell centrally ile and a geophone near the circumference of the pile head. geophone and loadcell are connected, the loadcell is ith a light, hand-held hammer. According to the Special ns for this job, "data is obtained for determining the pile teraction." Results of this test did not support its tion for determination of pile capacity. However, during ject, the shock method did prove helpful in evaluating the ty of driven piles. For TP5, the shock tests indicated that had cracked about 5 ft. below the splice. A review of the surements also indicated that TP5 cracked during driving. k tests were conducted by CEBTP Limited, 2201 Wisconsin , Suite 230, Washington D.C. 20007. A 30-inch square ment pile, TP5A, was driven and tested in place of TP5.

field measurements with the PDA were taken by Goble Rauche and Associates, Inc. (GRL). Field evaluations of pile y, driving stresses, and hammer performance were conducted he CASE Method with the PDA measurements. Results were d to the pile driving contractor, Louisiana Paving Company r, La. CAPWAPC (microcomputer version of CAPWAP) analyses lso performed to confirm and extend the field evaluations Using the measured force and velocity, the CAPWAPC procedure for the soil resistance parameters of a soil model similar model used in the wave equation.

Static load tests were performed by DOTD personnel on all piles after the series of tests involving dynamic measurements with hammer restrikes. TP2 was not tested with the PDA in a series of restrikes, but this pile was tested under a series of static load tests over different periods of time. For the "quick test" load method used, 5-to-10 ton load increments were applied; gross settlements and applied loads were recorded immediately before and after the application of each load increment. The pile was considered "failed" when the load on the pile could only be held by constant pumping of the hydraulic jack and with the pile being driven into the ground. In evaluating the test results, DOTD personnel defined failure as that load where the slope of the load-settlement curve became greater than 0.5 in. per ton (36).

The results of the field tests are summarized in Table 11. These include the dynamic tests and analyses with the PDA by GRL (40) and the static load tests by the DOTD.

Measured Pile Capacities - The increase in pile capacities with time are depicted graphically in Figure 8. Pile capacities increased by at least a factor of four over measured or estimated capacities at the end of initial driving (EOD). Unfortunately, this rapid increase in strength, together with the fact that static and dynamic testing were in most cases performed several days apart, limits the ability to compare PDA pile capacities directly with the static load test results. However, in viewing Figure 8, the increase in pile capacity, as measured by both the PDA and load tests, does produce a smooth, fairly continuous curve with time. The failure loads for the load tests performed at the end of the test series for the large displacement piles (i.e., TP3, TP4 and TP5A) do appear to be greater than failure loads projected off the PDA measurements. The static test failure loads for the cylindrical piles do, however, seem to fall on a curve projection of the PDA values. In general, the test results of the load tests and the PDA-computed capacities are in agreement within an

Pile	Test	Date	Blow Count	Average Energy Transfer	Average Max. Measured Compressive Force*	Average Max. Measured Compressive Stress	Maximum Computed Tensile Force*	Maximum Computed Tensile Stress	Axial Bearing Capacity PDA**		Axial Bearing Capacity Load Test		Remarks
			bl/ft	kip-ft	kips	ksi	kips	ksi	kips	tons	kips	tons	
TP1	EOD	6/10	38	16	1200	1.56	300	0.39	169	84.5			54"
	RSTK	6/12	(42/1")	12	1100	1.43	160	0.21	551	275.5			cylinder
	"	6/19	(43/1.25")	17	1520	1.97	120	0.61	658	329			
	"	7/2	(43/0.25")	16	1465	1.91	41	0.05	797	398.5			
TP2	EOD	6/10	48	12	930	1.21	330	0.49	160	80			54"
	Static	6/12									404	202	cylinder
	"	6/19									608	304	
	"	7/2									696	348	
TP3	EOD	6/8	10	23	820	1.77	300	0.65	60	30			24"
	RSTK	6/9	(6/1")	25	950	2.05	260	0.56	205	102.5			square
	"	6/18	(12/3")	18	950	2.04	183	0.40	344	172			
	"	6/26	(21/1.75")	15	996	2.15	226	0.43	376	188			
	Static	7/29									430	215	
TP4	EOD	6/8	14	24	1400	2.24	640	1.02	45	22.5			30"
	RSTK	6/9	(16/8.5")	34	1750	2.80	600	0.96	200	100			square
	"	6/12	(9/2")	36	1900	3.04	570	0.91	292	146			
	"	6/17	(12/0.5")	32	1980	3.17	560	0.90	341	170.5			
	"	6/26	(21/1.75")	25	1310	2.09	260	0.42	360	180			
	Static	7/9									518	259	

* Force measured 4 ft below pile top
** Calculated with Pile Driving Analyzer Measurements

TABLE 11: (Continued) SUMMARY OF DYNAMIC AND STATIC PILE TESTS, 6/8/87-7/15/87, I-310 APPROACH TO LULING BRIDGE

Pile	Test	Date	Blow Count	Average Energy Transfer	Average Max. Measured Compressive Force*	Average Max. Measured Compressive Stress	Maximum Computed Tensile Force*	Maximum Computed Tensile Stress	Axial Bearing Capacity PDA**		Axial Bearing Capacity Load Test		Remarks
			bl/ft	kip-ft	kips	ksi	kips	ksi	kips	tons	kips	tons	
TP5A	EOD	6/25	23	15	780	1.25	670	1.07	59	29.5			30"
	RSTK	6/26	(44/9")	25	1110	1.77	430	0.69	214	107			spliced
	"	6/29	(24/3")	24	1150	1.84	300	0.48	315	157.5			replacement
	"	7/6	(36/4.75")	24	1231	1.97	273	0.44	357	178.5			
	"	7/15	(27/1.5")	21	1130	1.81	284	0.46	393	196.5			
	Static	7/29									534	267	
TP6	EOD	6/15	15	15	910	1.87	615	1.26	90	45			36"
	RSTK	6/16	(34/10")	22	1000	2.05	580	1.19	199	99.5			cylinder
	"	6/19	(24/4.5")	20	1120	2.30	330	0.68	279	139.5			
	"	6/26	(18/2")	19	1120	2.30	260	0.53	397	198.5			
	"	7/6	(33/3.5")	20	1170	2.40	198	0.41	517	258.5			
	Static	7/20									530	265	
TP7	EOD	6/16	32	9	660	1.35	413	0.77	102	51			36"
	RSTK	6/17	(20/7.5")	20	1110	2.28	120	0.25	197	98.5			cylinder,
	"	6/20	(17/2")	23	1210	2.48	170	0.35	287	143.5			spliced
	"	6/26	(22/1.25")	20	1250	2.57	150	0.31	425	212.5			
	"	7/6	(31/2")	18	1250	2.59	213	0.99	508	254			
	Static										526	263	

* Force measured 4 ft. below pile top
** Calculated with Pile Driving Analyzer Measurements

range. An agreement of 10 to 15 percent between static and dynamic pile testing, when the available static is fully mobilized, has been cited (41). However, the capacities can be significantly in error when a poor best obtained." ("Match" refers to the program-computed and the sured pile head force waves.)

rements and predictions were conducted on TP1 concurrent tests on TP2; both TP1 and TP2 are 84-ft-long, 54 in. x indrical piles, in similar soil environments. At the end itial driving of these two piles, the PDA indicated a TP2 approximately 5 percent less than TP1. The differences the two piles' capacities measured at later times did not ate any regular pattern; however, the test loads for TP2 sistently lower than the PDA-predicted capacities for TP1 ame times, Figure 8.

tup - All methods used in the field control of pile ation determine the pile capacity at the time of the test. cludes static load tests, dynamic measurements of the stress d pile driving formulas. As shown in Figure 8, the test t this site experienced a significant gain in bearing y over the period of time from EOD to the final load tests. the pile capacities at EOD, as measured by the PDA for the les, the final measured pile capacities ranged from 4.4 to mes EOD capacities. Thus in some cases these setup values e than twice those used in this study for analyzing the pile formulas with the Louisiana historical test pile database.

iously discussed, setup is a gradual increase in capacity ts in clay or other soils with low permeability. The gain resistance can continue over long periods of time, with the pid increases generally occurring within the first few days. values for the test piles of this study also indicate the

Force measured 4 ft. below pile top

of the size and shape of the pile. In comparing setups ifferent piles, the gains in pile capacities occurring eriod of testing for the 24 and 30 inch square piles were ally larger than the capacity gains for the 54- and 36- ndrical piles.

in Figure 9, the gains in capacities for these test piles ximately linear when plotted against the log of time. It stated that the time-dependent increase in a pile's stabilizes after some time, t_o, beyond the initial driving timates of bearing capacity based on measurements from driving or on redriving performed at times $t < t_o$ have rved to be unreliable. Thus the EOD estimates of pile are not included in Figure 9. The resulting linear fits seven piles seem to indicate similar patterns of capacity for similar pile types. For example, consider the n in capacities for the cylindrical piling, TP1, TP2, TP6 Although there is a difference in the magnitude of the acities for the different size piles, the rate of increase ng capacities is similar. The differences between the TP1 linear fits may also be influenced by the different testing i.e., the PDA test of TP1 and the static load test of TP2. re "larger displacement piles" also had a common pattern of increase that was different than the pattern for the ical piles. The regression formulas for the variations in pacities with the $\log_{10}$ of time for the seven piles are as :

ical Piles -

1 (54"x5"): $P = 235.32 + 114.34 \log_{10} t$

2 (54"x5"): $P = 161.83 + 141.86 \log_{10} t$

6 (36"x5"): $P = 87.85 + 115.69 \log_{10} t$

7 (36"x5"): $P = 91.58 + 115.49 \log_{10} t$

are also presented in Figure 11. Although the pile predicted with the PDA measurements are similar (84.5 for TP1 and TP2, respectively), the displacements of simulated static test are greater than those of TP1 for ng loads. However, keeping in mind a possible load-nt difference between the two piles, Figures 12, 13, and the simulated static load curves of TP1 to the measured ves of TP2 at corresponding setup times.

-generated load-settlement curves have been proposed as supplementing or eliminating the conventional static However, it was suggested that "CAPWAPC ultimate pile and corresponding displacements should be checked he allowable pile head settlement, particularly for ntly end bearing piles and in large quake soils (41)."

ystem Performance and Driving Stresses - As measured by the energy actually transferred to the pile was much less D46-23 hammer's rated energy of 107.18 kip-ft. The energy at the end of driving varied between 9 and 24 In restrike tests, the maximum transferred energy of 36 curred during the 6/12/87 test of TP4, Table 11. However, itial driving, it was necessary to operate the hammer at ergies in order to limit the tensile driving stresses that in the concrete piles during easy driving. All seven of piles were installed with moderate blowcounts to a tip approximately 80 feet. By varying the hammer fuel setting iesel hammer being used, the combustion pressure and stroke increased or decreased. In a "Wave Equation Analysis" (43) prepared by Goble Rausche Likins (GRL) for Louisiana o., the pile driving contractor, it was recommended that l setting be reduced several levels until the blowcount a specified minimum value that varied with the pile type.

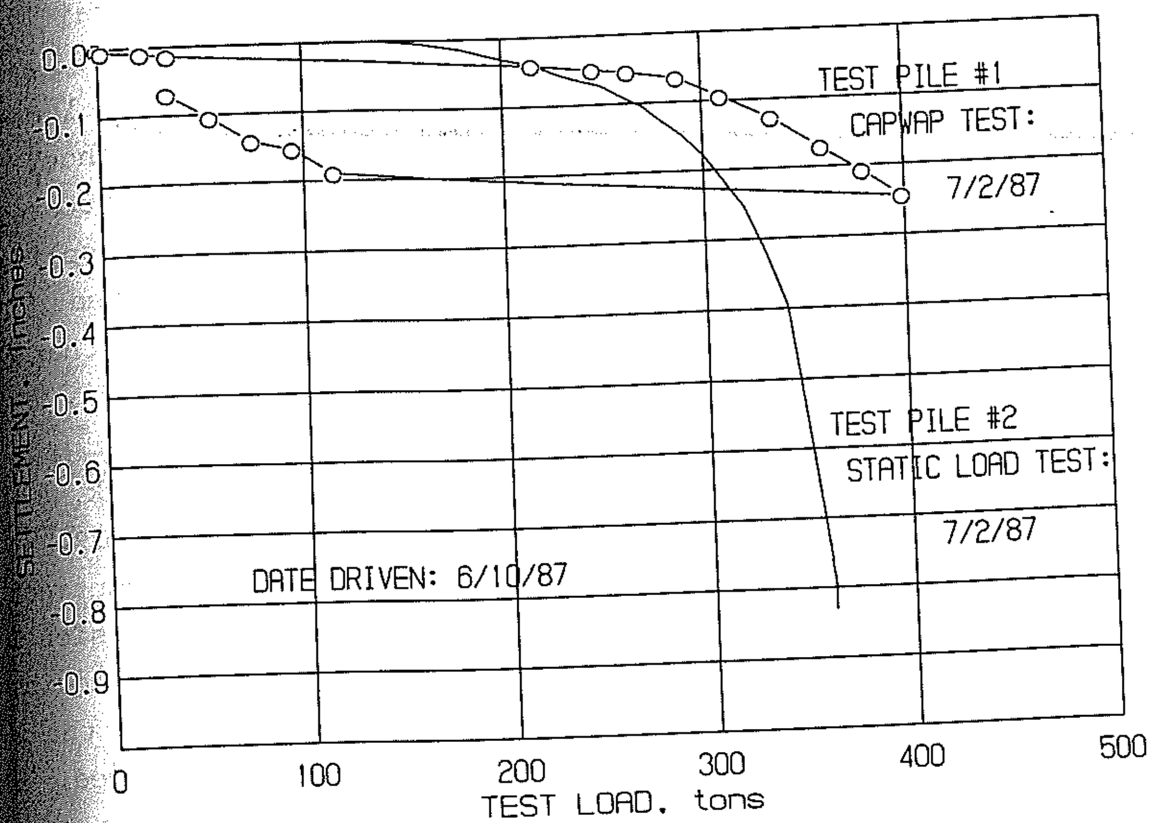


Figure 14. Load-Settlement Curve for TP1 and TP2 after Twenty-Two Days

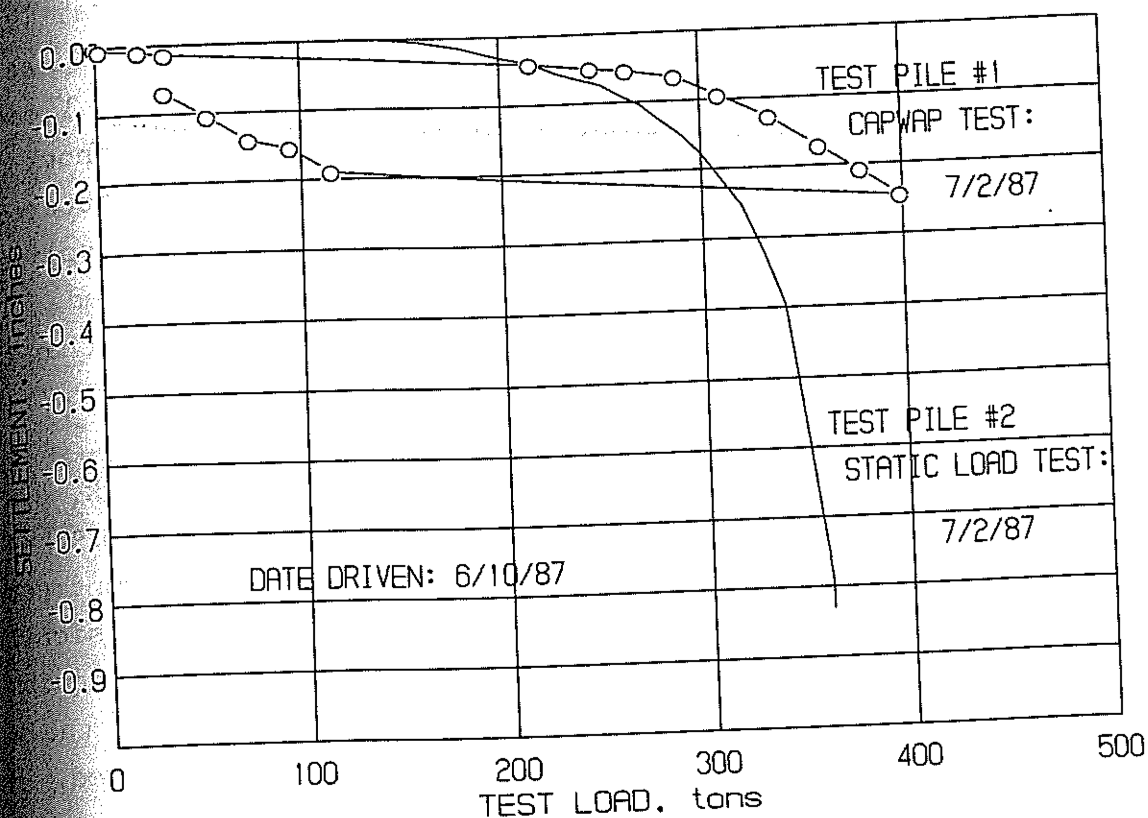


Figure 14. Load-Settlement Curve for TP1 and TP2 after Twenty-Two Days

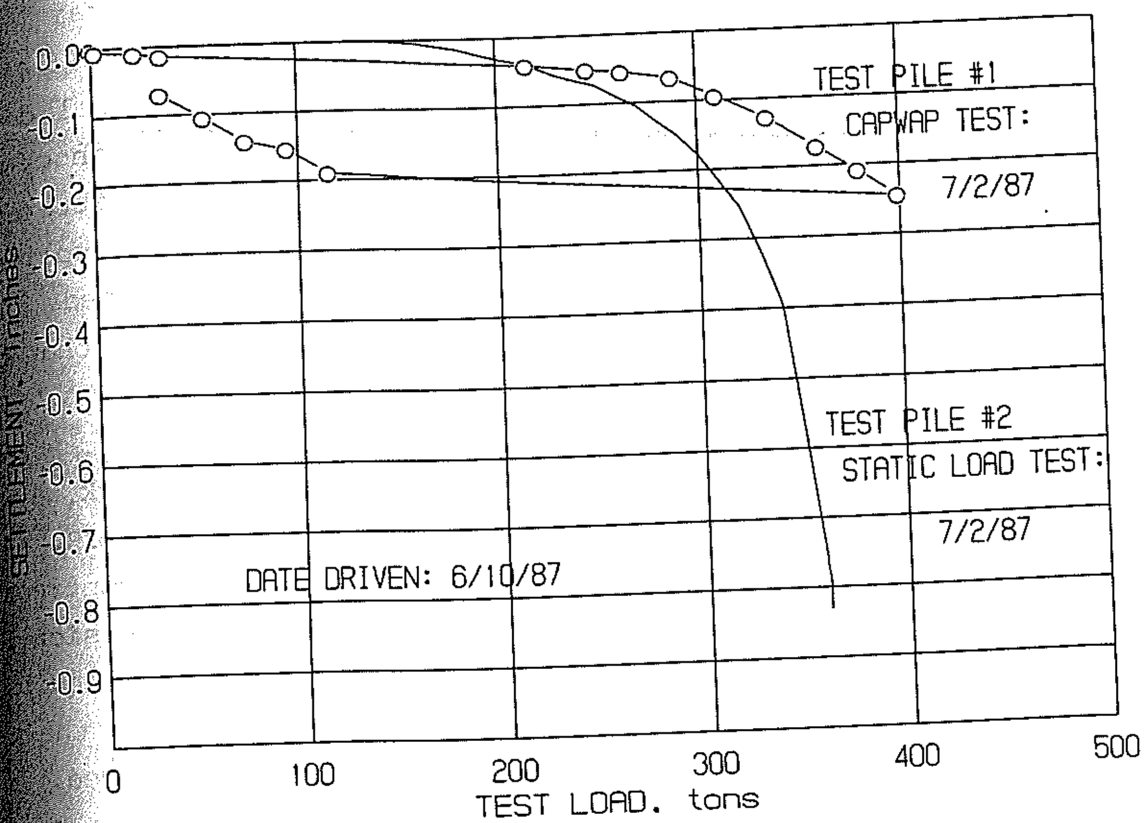


Figure 14. Load-Settlement Curve for TP1 and TP2 after Twenty-Two Days

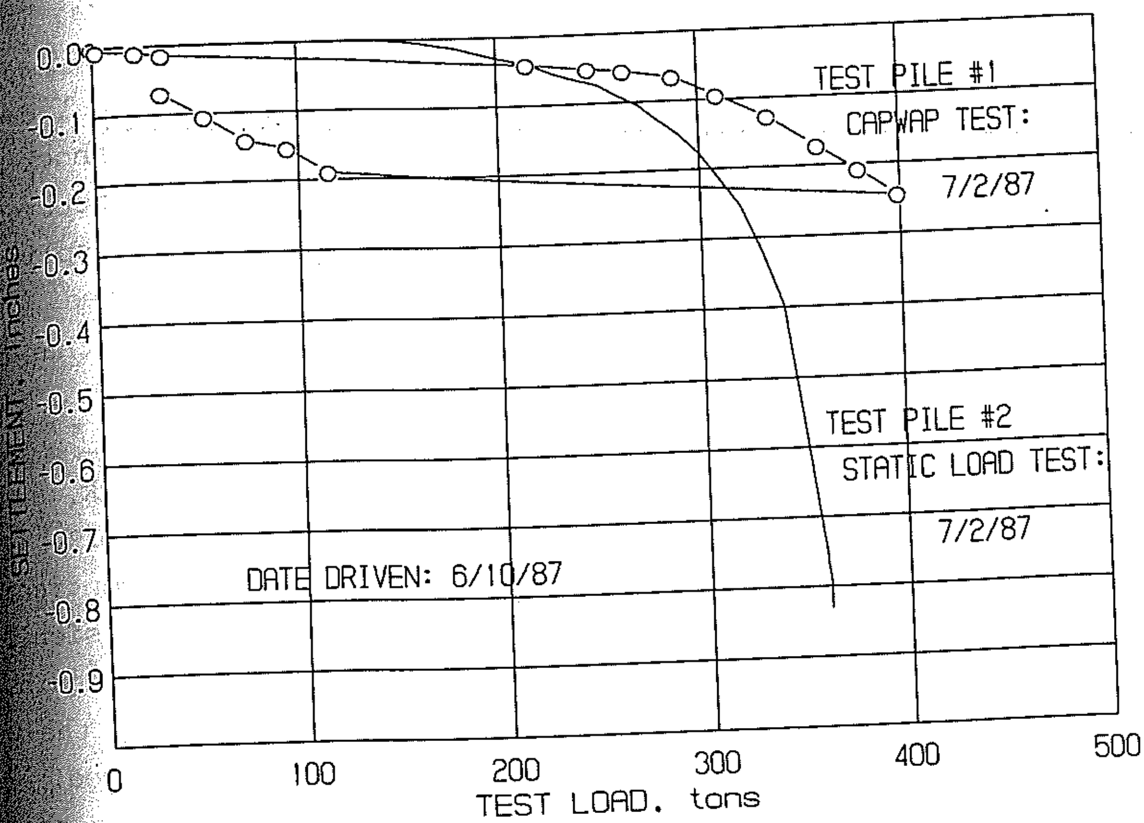


Figure 14. Load-Settlement Curve for TP1 and TP2 after Twenty-Two Days

The maximum compressive driving stress of 3.17 ksi occurred in TP4 on 6/17/87. Driving stresses in all of the other piles were less than 2.5 ksi and in many cases they were less than 2.0 ksi (Table 11). The highest tensile stress of 1.26 ksi occurred in TP6 at the end of driving. The original TP5 experienced structural damage during driving and was replaced. Both the PDA and the shock test measurements had indicated that the original TP5 was shattered at forty feet from the top, approximately 6 feet below the splice. A crack at twenty-six feet below the top was also detected. This pile had been driven with a higher hammer energy setting than that recommended by GRL.

<u>WEAP versus Field Measurements</u> - Prior to beginning work, a wave equation analysis for all test piles was performed by GRL (43). This was submitted to the Bridge Engineer through the contractor, Louisiana Paving Co., Inc., as required in Special Provision ITEM S-105, State Project No. 450-36-06. The pile driving equipment information was provided by the contractor. Based on the wave equation analysis, the pile driving system was approved. The contractor was required to use the approved system. The special provisions for this job required that any variation in the driving system be verified by a revised wave equation analysis and be approved in writing.

In the GRL report, eight wave equation analyses "were performed to investigate the suitability of a Delmag D46-23 hammer on the four different types of test piles." The analyses were conducted twice for each pile type in order to investigate the driving stresses, including tension, that would develop in the concrete piles during easy driving. Each pile was analyzed for driving with the fuel pump setting of the hammer at its highest level and then analyzed for a reduced fuel setting.

The wave equation analyses were performed using WEAP86. Input parameters used are summarized in Table 12. The 54- and 36-inch

TABLE 12

UT VALUES USED IN ADVANCE TEST PILE PROGRAM (Ref. 43)

pes: 54" x 5" cylinders of prestressed concrete
36" x 5" cylinders of prestressed concrete
30" x 30" square prestressed concrete
24" x 24" square prestressed concrete

Model: D46-23
mp Settings: 4 and 2

Cushion Material:	Conbest
Thickness:	1 in.
Diameter:	23 in.
Elastic Modulus:	280 ksi
Stiffness:	116,200 k/in

mping: Skin	0.20 s/ft	(Cohesive Soil)
Toe	0.15 s/ft	(Sandy Soil)

PILE TYPES

	54" x 5"	36" x 5"	30" x 30"	24" x 24"
uake (in)	0.1	0.1	0.1	0.1
ake (in)	0.1	0.1	0.25	0.20
Weight (k)	7.8	5.7	7.0	3.05
ushion Thickness (in)	6.0	6.0	8.0	8.0
ush Elastic Mod. (ksi)	30	30	30	30
ushion Area (in^2)	770	486	900	576
ength (ft)	84	84	84	84
lastic Mod. (ksi)	6000	6000	5000	5000

diameter cylindrical piles were considered to be unplugged during driving. It was assumed that spliced piles behave similar to unspliced piles; thus the splices were not modelled. Damping factors of 0.2 sec/ft (side or skin) and 0.14 sec/ft (toe) were selected for cohesive and sandy soils, respectively. Other input parameters are presented in Table 12.

The soil resistance parameters were determined in the CAPWAPC analyses (40). These are summarized for all seven piles in Table 13. The soil resistance, soil quake, and damping were determined through a trial and error process that matched the measured PDA pile top force and velocity in the CAPWAPC program with the wave equation soil model. Differences between the assumed input parameters of the WEAP analysis and the results of the CAPWAPC analysis can seen by comparing the values of Table 12 with the EOD values of Table 13. A graphical plot of the assumed side and tip values for soil damping and quake with those determined in the CAPWAPC computation are shown in Figures 15 and 16. In some cases, there is a significant difference between the "measured" and the assumed soil parameters. Some of the damping and quake parameters found in the CAPWAPC analyses at this site are much greater than those values commonly assumed in a wave equation analysis. The significant variation in the measured soil resistance values of the clays with setup time is also presented with the restrike soil parameters of Table 13.

The WEAP results were presented in the form of bearing graphs and tables. The variation of predicted ultimate capacities, maximum stresses (compression and tension), energy delivered, and ram

TABLE 13: SUMMARY OF CAPWAPC RESULTS
I-310 APPROACH TO LULING BRIDGE

Pile	Test	Date	Days After Driving	Quakes Skin in	Quakes Toe in	Smith Damping Skin sec/ft	Smith Damping Toe sec/ft	Ultimate Capacity Skin kips	Ultimate Capacity Toe kips	Ultimate Capacity Total kips	Average Unit Skin Friction k/ft²
TP1	EOD	6/10		0.1	0.1	0.203	0.417	119.0	49.6	168.6	.11
	RSTK	6/12	2	0.05	0.16	0.295	0.284	417.0	133.9	551.0	.37
	RSTK	6/19	9	0.1	0.13	0.312	0.395	474.8	183.2	658.0	.42
	RSTK	7/2	22	0.1	0.135	0.463	0.317	604.4	193.0	797.4	.52
TP2	EOD	6/10		0.15	0.20	0.355	0.255	10.6	149.0	159.7	.01
TP3	EOD	6/8		0.55	0.55	0.204	0.405	47.5	12.9	60.4	.07
	RSTK	6/9	1	0.12	0.12	0.112	0.428	175.0	30.0	205.0	.27
	RSTK	6/18	10	0.13	0.18	0.212	0.379	260.5	84.4	344.9	.41
	RSTK	6/26	18	0.19	0.19	0.291	0.277	282.7	94.2	376.9	.44
TP4	EOD	6/8		0.20	0.80	0.423	0.423	27.2	18.2	45.4	.03
	RSTK	6/9	1	0.12	0.75	0.306	0.435	149.7	49.9	199.5	.19
	RSTK	6/12	4	0.20	0.30	0.399	0.363	146.0	146.0	292.1	.18
	RSTK	6/17	9	0.175	0.30	0.463	0.338	134.7	207.2	341.9	.17
	RSTK	6/26	18	0.14	0.37	0.481	0.346	180.2	180.2	360.4	.23
TP5A	EOD	6/25		0.20	0.70	0.101	0.101	15.6	43.6	59.2	.02
	RSTK	6/26	1	0.20	0.70	0.449	0.203	95.5	118.5	214.0	.12
	RSTK	6/29	4	0.20	0.42	0.473	0.333	202.0	113.0	315.0	.25
	RSTK	7/6	11	0.17	0.35	0.490	0.476	252.0	103.0	357.0	.32
	RSTK	7/15	20	0.23	0.375	0.359	0.428	293.2	100.7	393.9	.37

TABLE 13 (Continued): SUMMARY OF CAPWAPC RESULTS
I-310 APPROACH TO LULING BRIDGE

Pile	Test	Date	Days After Driving	Quakes Skin in	Quakes Toe in	Smith Damping Skin sec/ft	Smith Damping Toe sec/ft	Ultimate Capacity Skin kips	Ultimate Capacity Toe kips	Ultimate Capacity Total kips	Average Unit Skin Friction k/ft^2
TP6	EOD	6/15		0.10	0.90	0.159	0.1	75.4	15.4	90.8	.10
	RSTK	6/16	1	0.20	0.55	0.294	0.280	163.1	35.4	198.5	.22
	RSTK	6/19	4	0.15	0.15	0.420	0.277	236.3	43.0	279.2	.31
	RSTK	6/26	11	0.12	0.265	0.337	0.299	310.0	87.2	397.2	.41
	RSTK	7/6	21	0.12	0.285	0.251	0.180	428.0	89.0	517.0	.57
TP7	EOD	6/16		0.15	0.25	0.231	0.176	54.8	47.9	102.0	.07
	RSTK	6/17	1	0.35	0.60	0.273	0.355	160.7	36.1	196.8	.21
	RSTK	6/20	4	0.30	0.32	0.346	0.190	232.0	55.5	287.5	.31
	RSTK	6/26	10	0.22	0.385	0.209	0.309	360.8	64.3	425.1	.48
	RSTK	7/6	20	0.20	0.280	0.252	0.254	421.6	86.4	508.0	.56

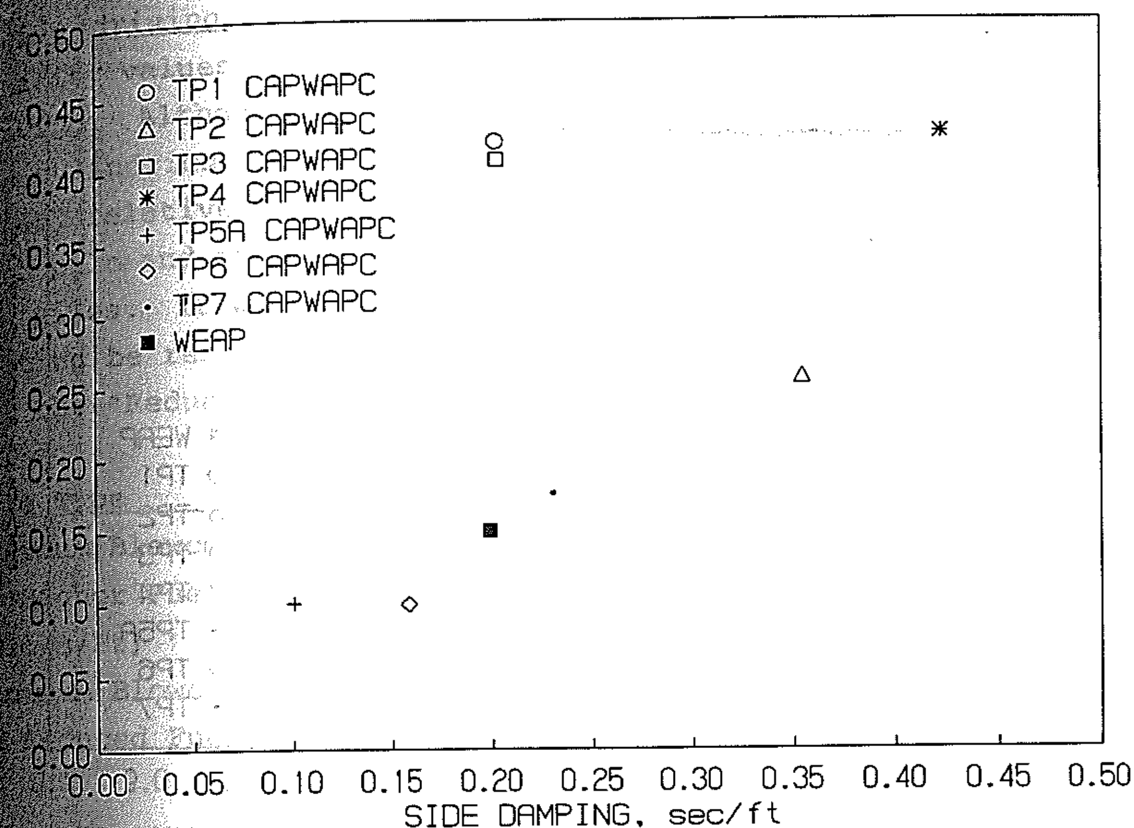


Figure 15. CAPWAPC/WEAP Damping Values

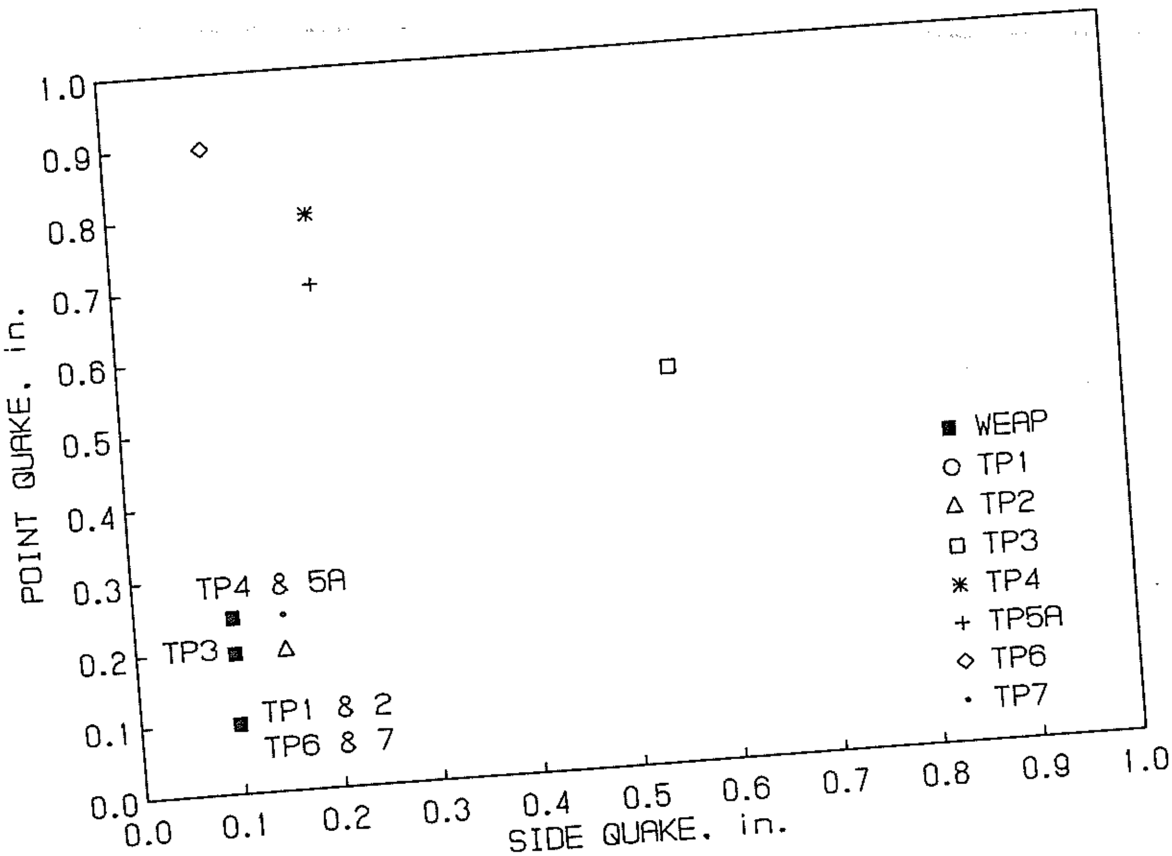


Figure 16. CAPWAPC/WEAP Quake Values

stroke versus the blowcount were included. It was noted in the GRL report that the static resistance of the pile may not be as high during driving as after a waiting period but that the ultimate capacity values used in the wave equation pertained to the time of driving. Although a 6-inch-thick plywood cushion was modelled for the cylindrical piles and an 8-inch-thick cushion for the square piles, relatively high tensile stresses were predicted for the 54-inch pile and it was recommended that an 8 inch cushion be used on all piles. It was further recommended that the hammer's fuel pump setting be reduced until the blowcount reached the minimum values for the "Reduced Fuel Setting" shown below:

Pile Type	54"x5"	36"x5"	30"x30"	24"x24"
Minimum Blowcount at Higher Fuel Setting (Blows/Ft)	30	25	25	20
Minimum Blowcount at Reduced Fuel Setting	60	40	33	25

The wave equation analyses were performed more as an investigation of the driving performance of the hammer and pile than as a predictor or guide for pile capacity. However, the analysis did require specification of the static capacities of the piles and it did precede the actual driving of the piles. Therefore, this analysis was used herein to compare wave equation predictions with the actual PDA measurements by GRL.

Information documented in the pile driving records for these test piles was typical of other DOTD test piles. The only information concerning the hammer operation was an estimate of the ram stroke. The type or thickness of cushion was not included. The WEAP predicted energy delivered was compared to the energy measured by the PDA and CASE methods, Figure 17. Two sets of data points, one set for each fuel setting, are plotted in this figure. The WEAP energy values for the cylindrical piles driven with the reduced

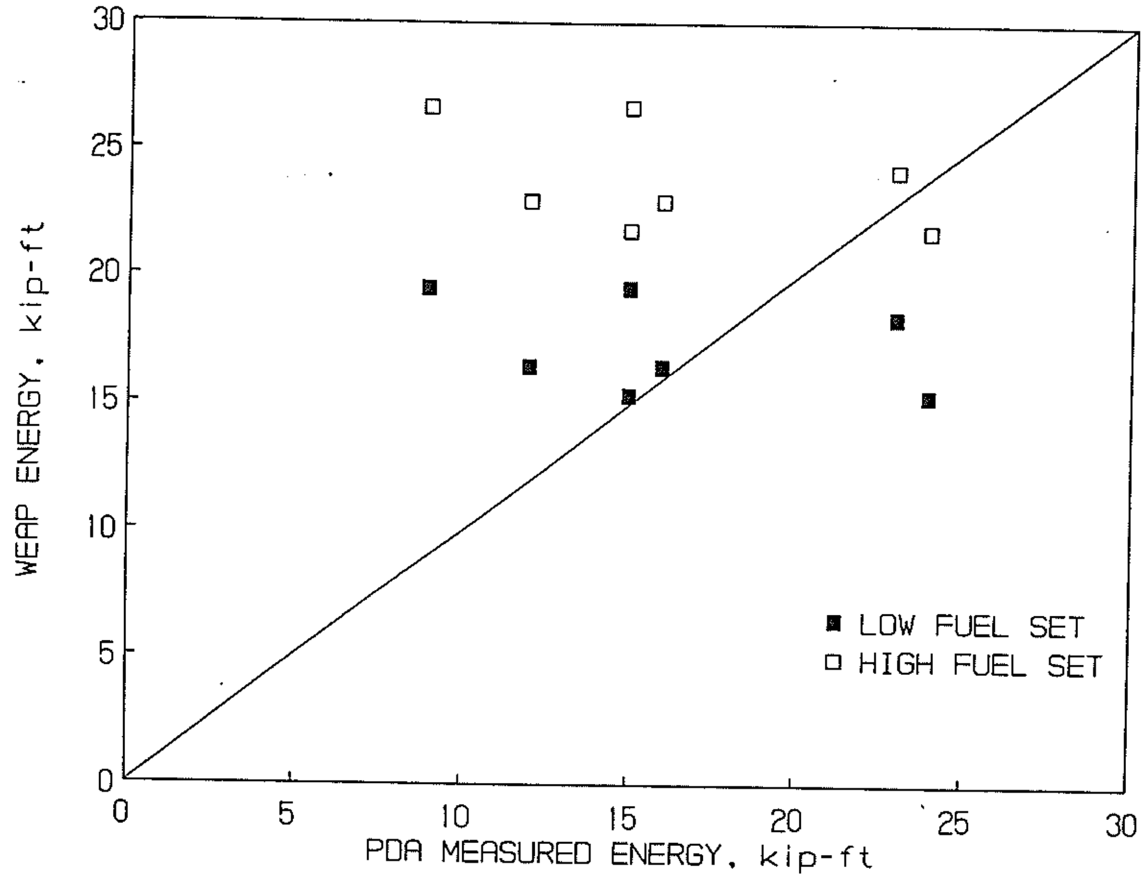


Figure 17. WEAP Predicted and PDA Measured Energy

l setting are in better agreement than are the WEAP energy ues for the higher fuel setting. This may indicate that the tractor was concerned with the potential for pile cracking and refore used care in driving these large cylindrical piles (TP1, , TP6, and TP7), as recommended by GRL. However, in examining energy input as predicted by WEAP with that measured by the SE method for the square piles (TP3 and TP4), the higher fuel ting is in better agreement. This is not the case for test pile A; however, TP5A was a replacement for TP5, which cracked during ving. DOTD records indicate that the TP5 pile was driven by the ntractor at a "high hammer energy which was against their commendation in a report sent to the contractor by his company RL) recommending that the low energy be delivered to the pile n the resistance of the soil is weak." The average PDA-measured rgy that was delivered in driving the replacement pile, TP5A, s in close agreement with the WEAP prediction for the reduced l setting of the hammer, i.e., 15 kip-ft for the PDA and 15.4 p-ft in the WEAP analysis.

e WEAP pile capacities were also compared with the CASE pacities, i.e., PDA measurements. Bearing graphs for the piles e reproduced in Figures 18, 19, 20, and 21. Pile capacities are esented for each of the test piles at the end of driving in gure 22. The WEAP capacities correspond to the hammer being erated at the reduced, designated "2", and high, designated "4", el setting. The operation of the hammer at the reduced fuel tting, resulted in a higher blowcount requirement to attain a rticular soil resistance since less energy was being put into the stem. The range of predicted pile capacities for each pile and mmer fuel setting are shown. In examining Figure 22, the WEAP-edicted capacities are in most cases more than twice those termined by the CASE method at the end of driving. There is an en greater difference when comparing the WEAP analyses at the gher fuel setting to the CASE capacities.

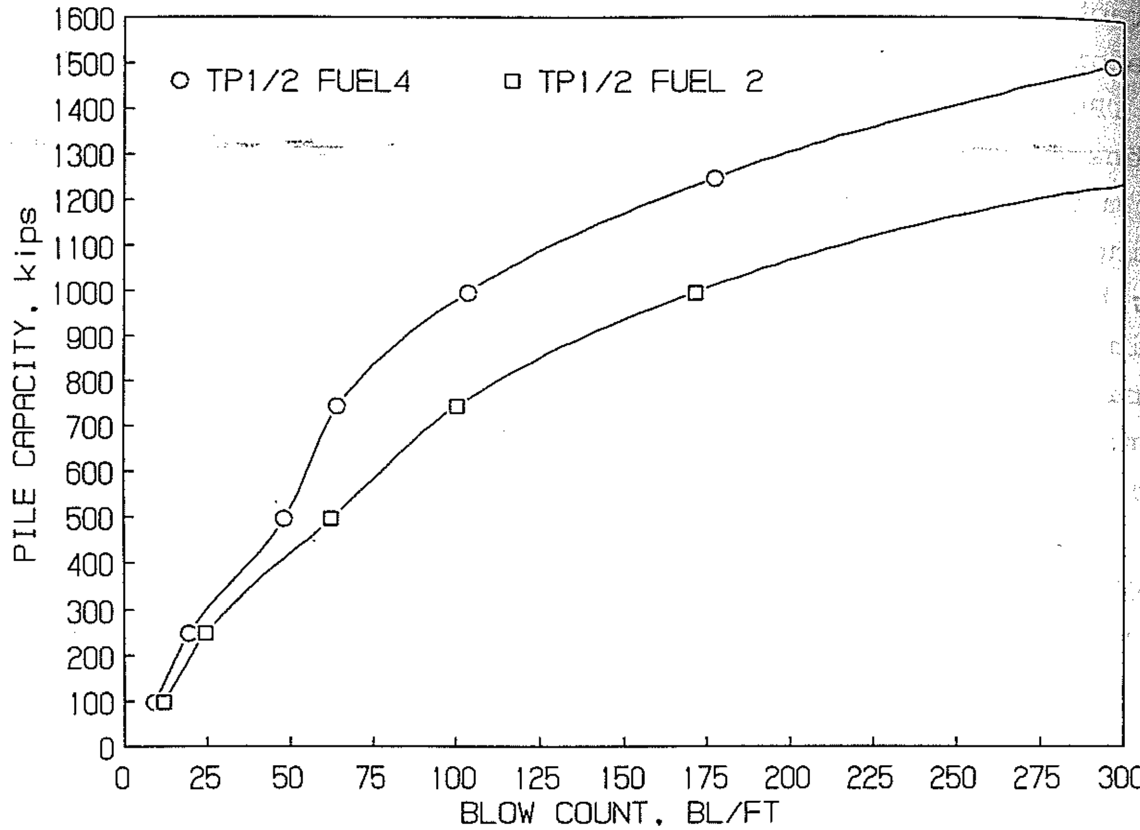


Figure 18. WEAP Analysis TP1 and TP2: 54" x 5" Prestressed Concrete Cylinder

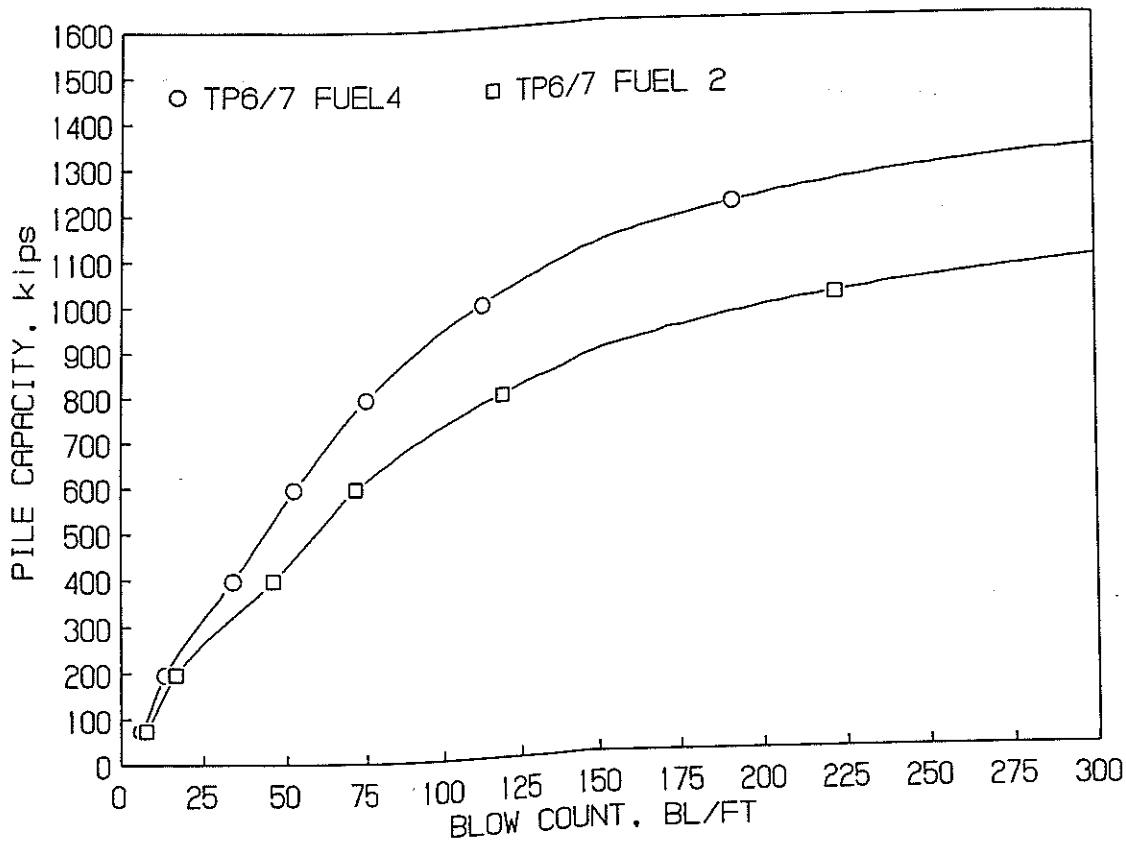


Figure 21. WEAP Analysis TP6 and TP7: 36" x 5" Prestressed Concrete Piles

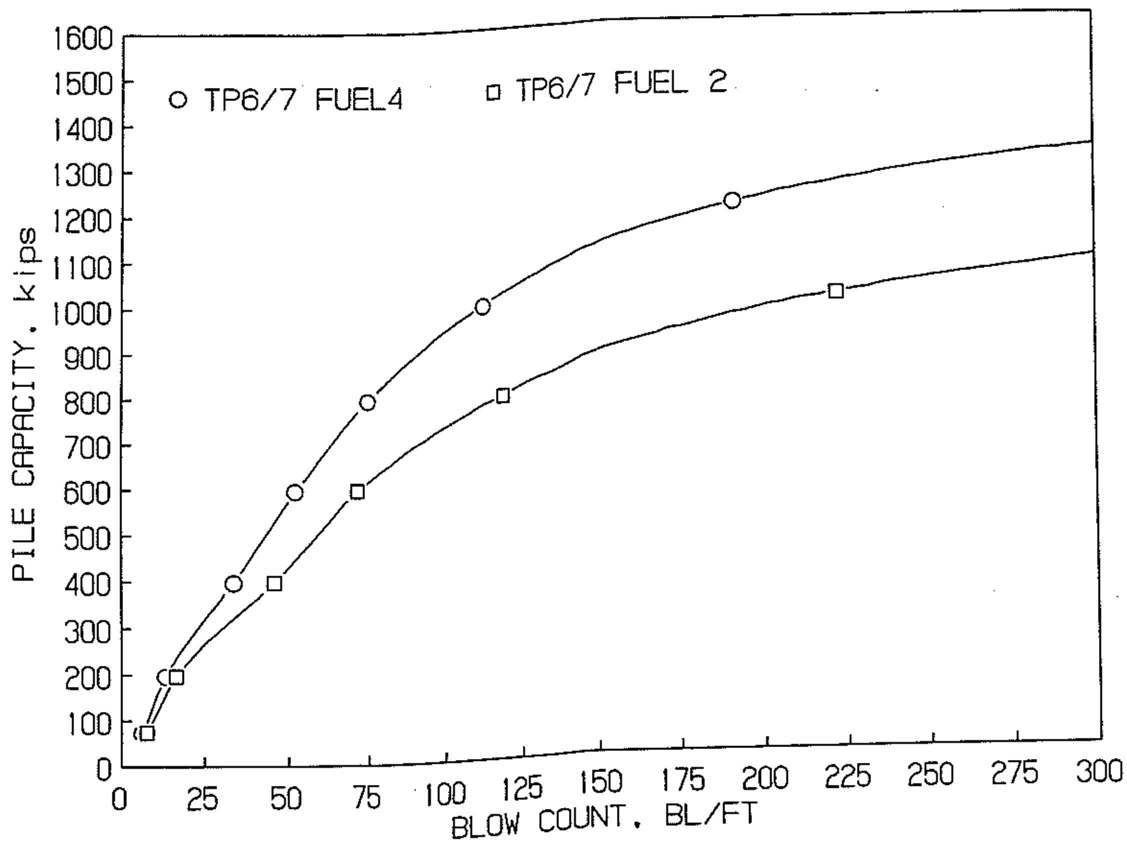


Figure 21. WEAP Analysis TP6 and TP7: 36" x 5" Prestressed Concrete Piles

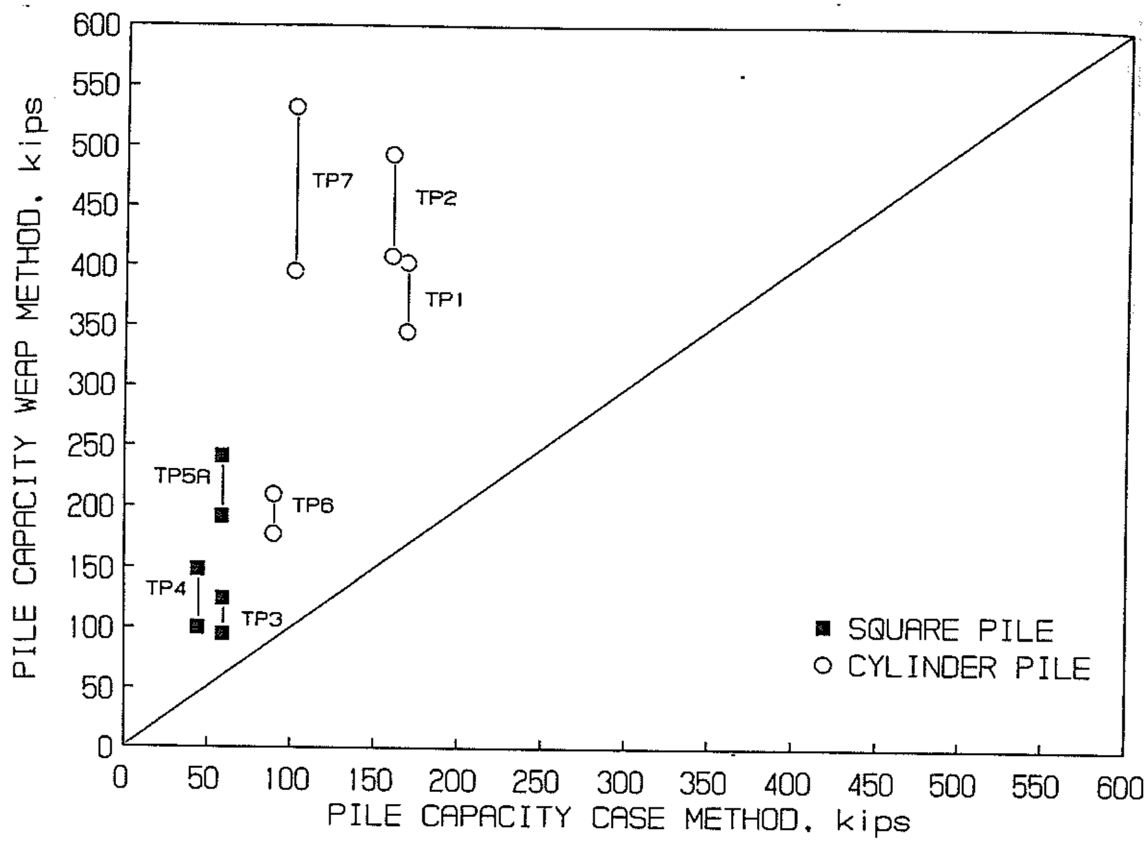


Figure 22. WEAP Predicted and PDA Measured EOD Pile Capacities

ure 23 compares the WEAP capacities corresponding to the count at the end of driving to the load tested capacities. The tic load tests were performed at times ranging from 22 to 35 s after the piles were initially driven as shown in Table 11. in, the range of WEAP-predicted pile capacities are shown for h pile with the hammer operating at two different fuel settings. nsideration must be given to the fact that this WEAP analysis conducted mainly as a means of determining hammer acceptability driving performance of the pile.) In reviewing the predicted formance of the hammer and pile, the analysis of the energy livered and the potential for pile damage seem to have been rly accurate. The predicted pile capacities do not appear to ree as well with those measured in the CASE method or with load st values. However, there probably was little effort to ensure at many of the WEAP input parameters were matched by actual field iving conditions. The fact that the pile cushions and details of e operation of the hammer are not documented supports this ossibility. Additionally, the hammer was reportedly operated nder conditions contrary to those recommended; this is possibly he cause of the cracking of TP5. Since a wave equation analysis equires more details on the pile driving system, additional care n monitoring and directing the field operations would assist in ts proper application.

Formula-Predicted versus Measured Pile Capacities - The PILCAP program was used to generate predictions of test pile capacities by the dynamic formulas. These were compared to those capacities measured in the PDA tests at the end of driving and the static load tests at the end of the series of tests for each pile. Figure 24 is a scatter plot of the formula-predicted pile capacities with corresponding CAPWAP values that were computed with the end of driving PDA measurements. All of the pile capacities computed by the formulas exceed those determined with the PDA readings. This is quantified with the computation of the R5 ratios of the Failure Load at the End of Driving to the Formula Predicted Capacity in

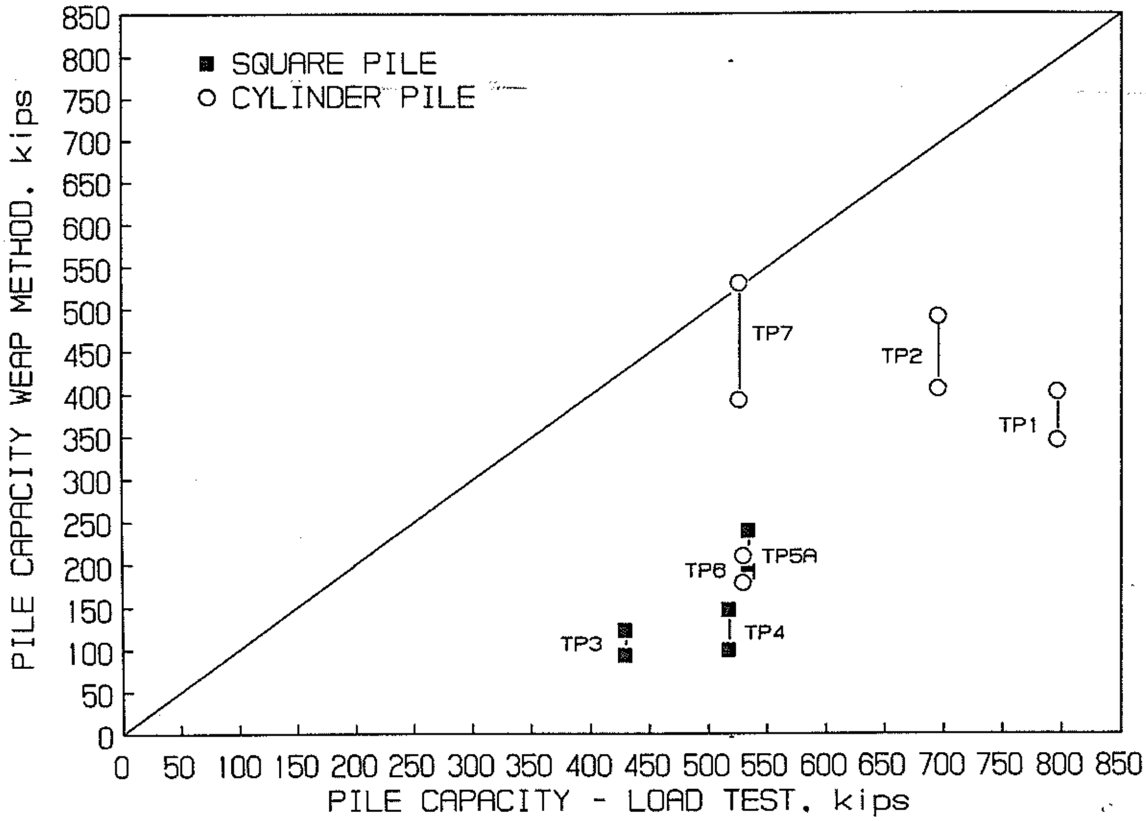


Figure 23. WEAP Predicted versus Static Load Test Capacities

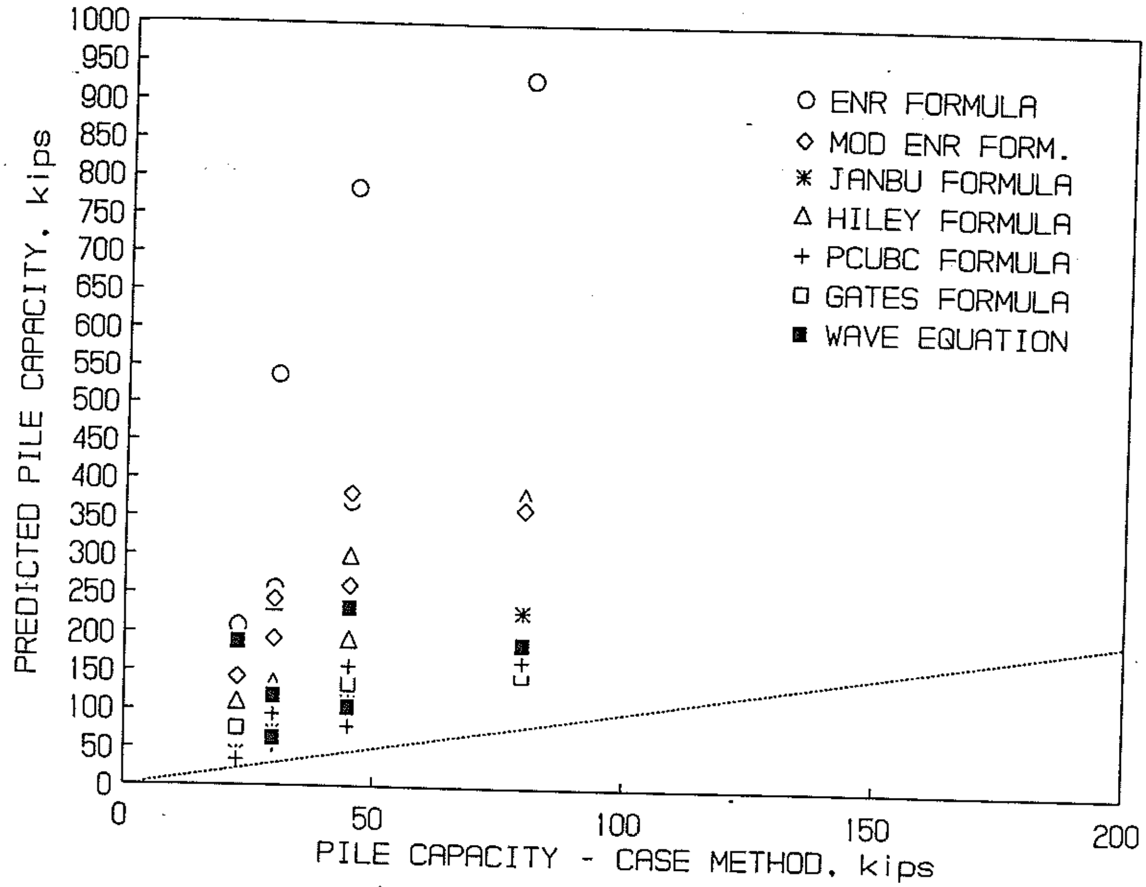


Figure 24. PDA End of Driving versus Formula Predicted Pile Capacities

Table 14. The end of driving capacity should correspond to the predicted ultimate capacity. However, all of the formulas overpredict capacities measured with the PDA. The ENR gives the greatest deviation from the measured EOD capacity.

A comparison of four of the formula predicted capacities to the failure load measured in a static test is shown in Figure 25. In some cases there is better agreement when comparing the measured, loaded capacities that have developed over the longer setup time. This is not compatible with the intent of the formula to model the dynamic resistance of the pile capacity corresponding to that which exists at the time of driving. It is, however, consistent with studies that have compared load test results with the performance of the pile driving formulas applied without restrike blowcounts. The ENR capacities again overpredict the measured failure loads much more than the other methods. In this comparison, the modified ENR shows the best agreement.

DEVELOPMENT OF IMPLEMENTATION SOFTWARE

SELECTION OF METHODS

Development of microcomputer software suitable for field execution of one or more dynamic methods was one of the main objectives of this project. Although the described evaluation failed to identify one formula that was greatly superior to the others at predicting historical load test results, the authors believe that a reevaluation of the methods using a yet unavailable high quality database would indicate a preference for the wave equation approach. This opinion is based on the greater flexibility of the wave equation (more input options), its sounder theoretical base, and its successful use by many others. Therefore, it was decided that one of the project tasks would be to facilitate field use of the WEAP87 program. There is an interactive data file creation program which accompanies WEAP87; however, it is not sufficient for use in the environment intended herein. It was decided to also

TABLE 14

SUMMARY STATISTICS FOR I-310 ADVANCE TEST PILE STUDY

R - Ratios*

Method	Mean R3	Mean R4	COV R3 and R4	Mean R5	Mean R6	COV R5 and R6
ENR	0.674	2.021	0.52	0.091	0.274	0.32
ENR MOD	1.127	3.380	0.35	1.158	0.473	0.23
Hiley	1.406	2.109	0.41	0.194	0.291	0.23
Gates	2.539	3.808	0.20	0.369	0.553	0.27
Janbu	2.740	6.164	0.64	0.359	0.808	0.34
PCUBC	3.628	7.257	0.60	0.480	0.961	0.31
WEAP87	2.113	2.113	0.38	0.317	0.317	0.46

* R3 = Test Failure Load / Formula Predicted Capacity
R4 = Test Failure Load / Formula Allowable times 2.0
R5 = Test Failure Ld divided by Setup / Formula Predicted Capacity
R6 = Test Failure Ld divided by Setup / Formula Allowable times 2.0

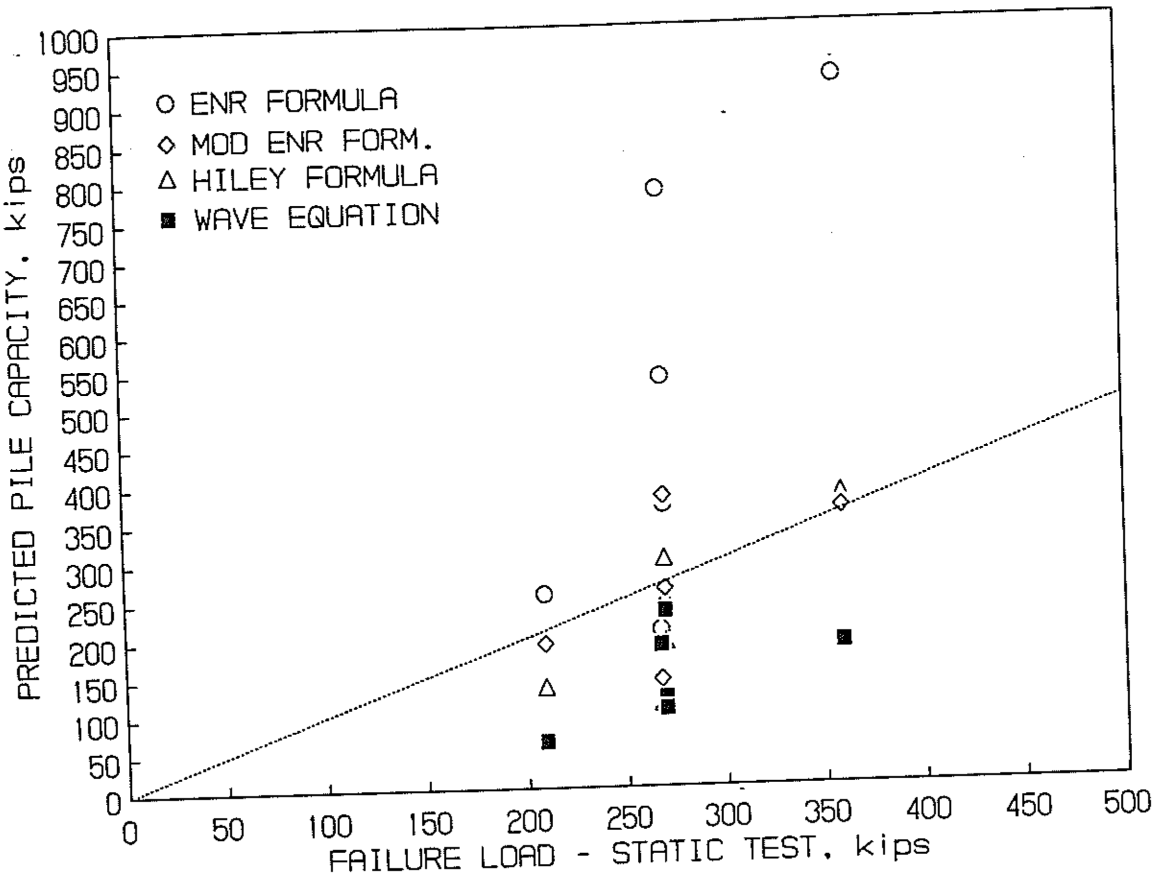


Figure 25. Formula Predicted versus Static Test Pile Capacities

provide for field microcomputer execution of the ENR and Hiley formula predictions of pile capacity. It was very simple to include these formulas and they provide a continued basis for comparison.

PROGRAM PCAP

The "batch" file, which controls the execution of the various programs, is named PCAP (Pile CAPacity). This program is begun by simply typing "PCAP" from the directory in which the programs reside. The essential four lines of this file are given below.

```
PILINP
WEAP87
PILOUT
PRINT PILE.OUT
```

Program PILINP requests input of information on the pile, driving hammer, and soils, either interactively using keyboard and screen or from a previously created data file named PILINP.DAT. If input is given interactively, the program will create file PILINP.DAT for possible later editing and/or repeated use without having to re-input.

Interactive Data Entry Program

When interactive data entry is selected, screen prompts are sequentially given for input of required information. First, the user is prompted for several pieces of information regarding location of the pile, project number, and date of driving. Next, a classification of the pile as timber, precast concrete, steel, composite, mandrel-driven, or other is requested. Other information specific to the classification is then requested so that a complete description of pile properties is accomplished.

Following the pile description, the user is requested to input information on the driving hammer and accessories. Air/steam and diesel hammers are handled by the program; however, diesel hammers must be selected from those listed in the WEAP87 hammer data file.

Information requested includes the hammer rated energy; ram weight; hammer efficiency; pile cap weight, stiffness, and coefficient of restitution; and pile cushion stiffness and coefficient of restitution. Next, the final blowcount is requested.

Following blowcount input, an estimated setup factor, or information needed for the program to compute setup factor, is requested. Finally, some information needed to complete the WEAP87 input is requested. This includes quake and damping factors.

Creation of WEAP87 Input File

The information input is used to create two information files. One file is simply a listing of all the information with descriptive headings, named PILINP.DAT. The other is a standard input file for WEAP87, called WEAPIN.DAT, which incorporates all the input specifications of pile, hammer, accessories, etc. Several candidate ultimate capacities are program-calculated using the ENR prediction as a "ballpark" estimate. WEAP87 calculates the blowcounts corresponding to these ultimate capacities. Then, the output program uses curve fitting to these (capacity, blowcount) points to determine the WEAP87 predicted ultimate capacity for the actual final blowcount.

Output

Program PILOUT produces screen and printed output showing the predicted ultimate pile capacities by WEAP87, the Hiley, and the ENR methods. These are capacities corresponding to the time at which the input final blowcount was recorded (generally at end of driving). Using the input or calculated setup factors, "long term" capacity predictions of the three methods are also calculated and output. Safety factors may be applied to these capacities to obtain allowable design loads. PILOUT also outputs all of the input pile and hammer descriptive information accompanying the predicted capacities.

Hardware

Program PCAP was created and run entirely on an IBM AT compatible microcomputer with a 20 megabyte hard drive and 512 kilobytes of RAM. This is the recommended hardware for field use of this software. A dot matrix printer is sufficient for output.

CONCLUSIONS

Until recently, the only dynamic analysis employed in the driving of piles by the Louisiana Department of Transportation seems to have been the Engineering News Record formula. This simple formula has been and is currently used by the field engineer as a guide for monitoring the driving installation of piles and validating their soil bearing capacity. It is the only dynamic method formerly specified in the **Louisiana Standard Specifications For Roads and Bridges**. Computation of the ENR allowable capacity is systematically computed for each foot of penetration and included in the field pile-driving record. The ENR-predicted pile capacity becomes an issue only if the specified depth of penetration is not achieved or if the pile's ENR-computed capacity at the specified penetration is less than design requirements.

During this study, many individuals within the Louisiana DOTD have expressed their thoughts regarding the limitations and shortcomings of this reliance on the ENR and the need for a more comprehensive program utilizing more modern dynamic methods. This need becomes even more obvious on a job such as the I-310 Luling Bridge Approach where static load tests are either very difficult to conduct or not possible. The Louisiana DOTD move toward these advanced dynamic methods is current with the efforts being made by many state departments of transportation. The results of these efforts have recently begun appearing in the literature. Many of the conclusions that were formed through this study were probably anticipated by some. However, it is hoped that this study will formalize these views and provide an impetus for locally improving the dynamic program in pile driving.

To say that the evaluation of static capacity and the dynamic analysis of a driven pile is "complex" is an understatement.

Additionally, the necessity for reliance on historical data in the evaluation of methods further complicates the process. Available information is very incomplete and often hard to interpret. Based on the evaluations of the pile driving records and literature reviewed, the following observations and conclusions are made:

1. Most state departments of transportation are at this time using the ENR formula in one form or another (although there is significant interest and desire to move towards a more consistent method).

2. Most of the available historical data files of test piles are missing much of the information needed to completely describe and accurately analyze the dynamic performance of the hammer and pile.

3. Based on comparative analyses of various pile driving formulas using historical data from the Louisiana DOTD files, none of the studied dynamic formulas stands out as being more reliable than any of the others.

4. Most of the studies reviewed in the literature that involved the dynamic analysis of driven piles generally emphasize the superiority or desirability of an analysis based on the wave equation.

5. The hammer-pile-soil model of the wave equation provides a better representation of the real system. The wave equation analysis provides an accurate assessment of the hammer and pile drivabilty. Its ability to predict pile capacity is not as consistent. However, predicted pile capacity is improved by a complete follow through in the field to insure that the conditions of the equipment and operation of the hammer are the same as those on which the analysis was based.

6. The wave equation analyses that were conducted for the historical data files did not perform much better than the other dynamic formulas; its performance varied. However, much of the information required for the wave analysis was missing and had to be assumed.

7. Locally, past utilization of dynamic analyses for pile foundations has been limited in scope. A dynamic analysis should be included in the design and selection of the pile and hammer and should also be used as a tool for monitoring and verifying the pile capacity and integrity during its installation.

8. The pile driving analyzer (PDA) performed well in predicting and/or measuring pile capacity for the I-310 Advance Test Pile Study. It was also very accurate in identifying damage due to pile driving and in monitoring pile and hammer performance. It has been promoted as being able to provide a simulated static load-settlement curve also, but the results derived from the I-310 data were inconclusive and should be used with caution. The PDA does have the potential for complementing or replacing static load tests. Operation of field equipment and interpretation of the measurements require skilled personnel.

9. Setup was found to significantly affect the pile capacity of the piles in the I-310 Advance Test Pile Program. Setup values exceeding those commonly suggested in the literature, and as high as 11, were estimated. Pile capacities of piles driven in soils with high setup potential are difficult to predict using dynamic formulas. A program including a series of pile restrikes and/or static load tests can be used to determine the setup characteristics of a site. There were some indications that the pile type and size also influence the pile setup relationship.

RECOMMENDATIONS

In order to enhance the design synthesis and quality control in the construction of pile foundations, it is recommended that the Louisiana DOTD formally develop a more comprehensive pile foundation program that will include the various dynamic methodologies. The following specific items are proposed as a means for achieving this goal:

1. Use greater detail in documenting test pile driving accessories and hammer operation. A formal end of driving report should be required. With the availability of more complete test pile data files, the creation of a quality database for future review and evaluation of dynamic methods can be continued. Test piles should be loaded at least to three times the design load, and preferably to failure.

2. Use of the wave equation should be increased and systematically included in the selection of the pile types, selection and control of the hammer, and in planning the inspection program. Pile driving contractors should be required to submit a wave equation analysis that verifies the ability of their equipment to adequately drive the piles. The construction specifications should require that driving equipment and methods employed in the field match the assumptions made in the submitted wave equation analysis.

3. LADOTD field personnel should be provided with bearing graphs from dynamic analyses conducted for the pile(s) and hammer(s) to be used on the job. These graphs should include documentation concerning the equipment or other conditions on which it is based. The field engineer should have the means to produce alternate graphs in case variations in occur. Movement toward more familiarity and reliance on capacities predicted by the wave

equation is recommended but will require a field computer. A computer program for use in the field, PCAP, was developed during this study. PCAP includes the application of WEAP87, the ENR and Hiley Formulas for field computations.

4. The pile driving analyzer should be given further consideration for complementing or eliminating static load tests. A detailed analysis of the I-310 Luling Bridge Approach pile driving program should be conducted and formally reported. An approach utilizing the PDA in restrike tests should be developed for assessing setup.

REFERENCES

1. Louisiana Department of Transportation and Development, Office of Highways, <u>Louisiana Standard Specifications for Roads and Bridges</u>, 1982 Edition, DOTD 03-07-7000, , Baton Rouge.

2. Chellis, R., <u>Pile Foundations</u>, Second Edition, McGraw-Hill, New York, N.Y., 1961.

3. Smith, E., "Pile-Driving Analysis by the Wave Equation," ASCE Transactions, Volume 127, Part 1, 1962.

4. Poulos, H. G. and Davis, E. H., <u>Pile Foundation Analysis and Design</u>, John Wiley and Sons, Inc., 1980.

5. Sorensen, T. and Hansen, B., "Pile Driving Formulae, An Investigation Based on Dimensional Considerations and Statistical Analysis," Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1957.

6. Agerschou, H. A., "Analysis of the Engineering News Pile Formula," <u>Journal of the Soil Mechanics and Foundation Engineering Division</u>, ASCE, 88, SM5, 1962.

7. Flaate, K. S., "An Investigation of the Validity of Three Pile Driving Formulae in Cohesionless Material," Pub. No. 56, Norwegian Geotechnical Institute, 1964.

8. Housel, W. S., "Pile Load Capacity-Estimates and Test Results," <u>Journal of Soil Mechanics and Foundation Engineering Division</u>, ASCE, Vol. 92, SM4, 1966.

9. Olsen, R. E., and Flaate, K. S., "Pile-Driving Formulas for Friction Piles in Sand," <u>Journal of Soil Mechanics and Foundation Engineering Division</u>, ASCE, Vol. 93, SM6, 1967.

10. Lowery, L. L., et.al., "Use of the Wave Equation to Predict Soil Resistance on a Pile During Driving," 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969.

11. Bowles, J. E., <u>Foundation Analysis and Design</u>, McGraw-Hill Book Co., 4th Ed., New York, 1988.

12. Rausche, F.; Goble, G.; and Moses, F.; "A New Testing Procedure for Axial Pile Strength," Paper 1482, Offshore Technology Conference, Houston, Texas, 1971.

13. Rausche, F.; Moses, F.; and Goble, G. G.; "Soil Resistance Predictions From Pile Dynamics," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 98, SM9, September 1972.

14. Denver, H., and Skov, R., " Investigation of the Stress-Wave Method By Instrumented Piles," Third International Conference, Application of Stress-Wave Theory to Piles, ISMEFE and CGS, Ottawa, May 1988.

15. Poplin, J. K., "Preliminary Evaluation of Test Pile Records For Highway Structures in Louisiana," Engineering Research Bulletin No. 106, Div. of Engineering Research, Louisiana State University, Baton Rouge, 1971.

16. Blessey, W. E., and Lee, P. Y., "Report on Wave Equation Analysis of Pile Driving in the New Orleans Area," Pile Driving Contractors Association of New Orleans Inc., March 1980.

17. Briaud, J. L, and Tucker, L. M., "Measured and Predicted Axial Response of 98 Piles," Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 9, September 1988.

18. Whited, G. C., and Laughter, C. N., "Pile Foundation - From Preliminary Boring to Production Driving," Transportation Research Record 884, Transportation Research Board, Washington, D. C., 1982.

19. Fragaszy, R. J.; Argo, D.; and Higgins, J. D.; "Comparison of Formula Predictions with Pile Load Tests," Paper No. 880214, Transportation Research Board, 68th Annual Meeting, Washington, D. C., January 1989.

20. Fragazy, R. J.; Higgins, J. D.; and Lawton, E. C.; "Development of Guidelines for Construction Control of Pile Driving and Estimation of Pile Capacity (Phase I)," Report No. WA-RD-68.1, Washington State Department of Transportation, June 1985.

21. Engel, R. L., "Discussion of Procedures for the Determination of Pile Capacity," Transportation Research Record 1169, Transportation Research Board, Washington, D. C., 1988.

22. Louisiana Department of Highways, Formulated Pile Loads for Single Acting and Approved Differential Hammers, LDH 03-10-8000, Louisiana Department of Highways, January 1973.

23. Goble, G. G., and Rausche, F., Wave Equation Analysis of Pile Driving - WEAP Program, FHWA-IP-76-14.1, Federal Highway Administration, July 1976.

36. Fuller, F., and Hoy, H., "Pile Load Tests Including Quick-Load Test Method, Conventional Methods, and Interpretations," Highway Research Record No. 333, Highway Research Board, Washington, D. C., 1970.

37. Bailey, P. F., and Sweeney, S. E., "NYSDOT's Construction Control of Pile Foundation with Dynamic Pile Testing," Transportation Research Record 1169, Transportation Research Board, Washington, D.C., 1988.

38. Ledbetter, J. F., Jr., "Use of the Wave Equation by the North Carolina Department of Transportation," Transportation Research Record 1169, Transportation Research Board, Washington, D. C., 1988.

39. Vanikar, S., Manual on Design and Construction of Pile Foundations, FHWA-DP-66-1, U.S. Dept of Transportation, Federal Highway Administration, Washington, D.C., April 1986.

40. Goble Rausche Likins and Associates, Inc., "Dynamic Pile Tests Performed During June and July, 1987 for Advance Pile Test Program: Louisiana D.O.T. Project No. 450-36-06," a report submitted to Louisiana Paving Company, Inc., Kenner La., August 1987.

41. Hannigan, P.J., and Webster, S.D., "Comparison of Static Load Tests and Dynamic Pile Testing Results", Second International Symposium, Deep Foundation Institute, Luxembourg, May 4-7,1987.

42. Skov, R., and Denver, H., "Time-Dependence of Bearing Capacity of Piles," Third International Conference on Application of Stress-Wave Theory to Piles, ISSMFE and Canadian Geotechnical Society, May 1988.

43. Rausche, F., "Wave Equation Analysis for D.O.T." Project No. 456-36-06, GRL Job No. 871057, Goble Rausche Likins and Associates, Inc., April 16, 1987.

APPENDIX A
LISTING OF PILE GROUP FILE NUMBERS

Square Concrete Piles

There are 56 prestressed, precast square concrete piles in the database. Most are 14 or 16 inch prismatic piles without holes. Pile unit weight was taken as 150 pounds per cubic foot; pile modulus of elasticity was taken as 4000 kips per square inch. The following pile numbers are included in this group: 011, 012, 013, 014, 015, 017, 018, 019, 026, 027, 028, 030, 031, 032, 036, 037, 038, 039, 040, 041, 043, 046, 048, 050, 051, 055, 056, 057, 058, 059, 060, 061, 062, 063, 064, 065, 074, 075, 076, 077, 078, 079, 080, 086, 087, 088, 089, 090, 091, 092, 093, 094, 095, 096, 097.

Timber Piles

There are 12 timber piles in the database, mostly class B piles about forty to sixty feet long. Pile unit weight was taken as 60 pounds per cubic foot; pile modulus of elasticity was taken as 1800 kips per square inch. The following pile numbers are included in this group: 053, 054, 066, 067, 068, 069, 070, 072, 082, 083,084, 085.

Piles Driven with Single Acting Air/Steam Hammers

There are 61 piles in the database which were driven with single acting air/steam hammers. The following pile numbers are included: 012, 013, 016, 017, 018, 019, 027, 028, 030, 031, 032, 034, 035, 036, 037, 038, 039, 040, 041, 046, 048, 050, 051, 053, 054, 055, 056, 057, 058, 059, 060, 061, 062, 063, 064, 065, 066, 067, 068, 069, 070, 072, 073, 074, 075, 082, 083, 084, 085, 086, 087, 091, 092, 093, 094, 095, 096, 097, 099, 100, 101.

Piles Bearing in Clay

There are 43 piles in the database which are bearing in clay and have clay side soils. The following pile numbers are included: 013, 017, 031, 032, 034, 035, 041, 046, 050, 051, 053, 054, 055, 058, 060, 061, 062, 064, 065, 066, 067, 068, 069, 070, 072, 079, 080, 082, 083, 084, 085, 086, 087, 088, 089, 090, 092, 093, 096, 097, 098, 100, 101.

Piles Bearing in Sand

There are 12 piles in the database which are bearing in sand and have side soils which are sand and/or clay. The following pile numbers are included: 011, 014, 015, 016, 026, 028, 039, 043, 056, 059, 081, 091.

Summary statistics for the five pile groups described above are given in Tables 6 -10. Ratios R1 and R2 are not included because they involve the maximum applied test load which, unless equal to the pile failure load, would not be expected to correlate with final blowcount. The covs for R3 and R4 are always equal, as are the covs for R5 and R6.