Determining Pile Bearing Capacity by Some Means Other Than The Engineering News Formula

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

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.

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

FINAL REPORT

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December 1989
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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

This study recommends that the current *Louisiana Standard Specifications For Roads And Bridges* (1) be revised and expanded to include other methods for evaluating the dynamic performance of piles in addition to the presently specified Engineering New York formula. Familiarity and use of the wave equation analysis for design and construction should be encouraged. Field personnel should be instructed on the importance of duplicating the conditions on which the dynamic analysis is based and should be provided the means for systematically conducting such an analysis in the field. Dynamic measurements should also be considered as a means for supplementing or eliminating static load tests. It is expected that increased use of these advanced techniques will lead to more accurate predictions of pile capacity and cost savings for foundations.
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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 (2). 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>
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<th>Upper and Lower Limits of Safety Factor</th>
<th>Nominal Safety Factor</th>
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<td></td>
<td>Safety Factor $= \frac{P_u}{P_d^b}$</td>
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<td>Pile Capacity Range, kips</td>
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<tr>
<td></td>
<td>0 - 200</td>
<td>200 - 400</td>
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<td>Engineering News</td>
<td>1.1 - 2.4</td>
<td>0.9 - 2.1</td>
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<td>1.1 - 4.2</td>
<td>3.0 - 6.5</td>
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<td>PCUBC</td>
<td>2.7 - 5.3</td>
<td>4.3 - 9.7</td>
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<td>Redtenbacher</td>
<td>1.7 - 3.6</td>
<td>2.8 - 6.6</td>
</tr>
<tr>
<td>Eytelwein</td>
<td>1.0 - 2.4</td>
<td>1.0 - 3.8</td>
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<td>0.8 - 3.0</td>
<td>0.2 - 2.5</td>
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<td>1.3 - 2.7</td>
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<td>5.1 -11.1</td>
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<td>Modified ENR</td>
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<td>1.6 - 5.2</td>
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<tr>
<td>Gates</td>
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<td>2.5 - 4.6</td>
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<tr>
<td>Rabe</td>
<td>1.0 - 4.8</td>
<td>2.4 - 7.0</td>
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</table>

* After Housel (1966)

$P_u$ = ultimate test load

$P_d$ = design capacity, using the nominal safety factor recommended for the equation.
### TABLE 2
SUMMARY OF STATISTICAL ANALYSES

<table>
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<tr>
<th>Material</th>
<th>Standard Deviation on R</th>
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<th>Number of Load Tests</th>
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<td>A 0.78</td>
<td>26.0</td>
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<td></td>
<td>F 0.70</td>
<td>17.5</td>
<td>5.8</td>
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<tr>
<td>Bygden</td>
<td>S &amp; H 0.27</td>
<td>3.8</td>
<td>1.4</td>
<td>50</td>
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<td></td>
<td>F 0.37</td>
<td>10.1</td>
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<tr>
<td>St. Louis</td>
<td>S &amp; H 0.25</td>
<td>3.6</td>
<td>2.3</td>
<td>78</td>
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<td></td>
<td>F 0.22</td>
<td>3.2</td>
<td>2.0</td>
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<td>3.8</td>
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<td>78</td>
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<tr>
<td></td>
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**Legend:**
- **S & H** = Sorensen and Hansen (1957)
- **A** = Agerschou (1962)
- **F** = Flaate (1964)
- **O & F** = Olsen and Flaate (1967) (steel piles in sand)
- Nominal safety factor = 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. File 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 $R = \frac{P_{\text{measured}}}{P_{\text{estimated}}}$ were performed on the data reported by Sorensen and Hansen (5), Agerschou (6), and Olsen and Flaate (9). The standard deviation $s$ 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 ($P_{\text{ENR}} / P_{\text{TEST}}$) 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 ($P_{\text{TEST}} / P_{\text{WAVE}}$) 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>
<thead>
<tr>
<th>Formula</th>
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<th>$s_{(ln \mu)}$</th>
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<td>Denver</td>
<td>0.30</td>
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<tr>
<td>Fract</td>
<td>0.41</td>
<td></td>
<td>123</td>
<td>A</td>
</tr>
<tr>
<td>Sign</td>
<td>0.12</td>
<td>0.14</td>
<td>97</td>
<td>Goble et al. (1981)</td>
</tr>
<tr>
<td>Sign</td>
<td>0.11</td>
<td></td>
<td>19</td>
<td>Skov (1988)</td>
</tr>
<tr>
<td>Sign</td>
<td>0.14</td>
<td></td>
<td>14</td>
<td>Present Investigation</td>
</tr>
<tr>
<td>Sign</td>
<td>0.13</td>
<td>0.16</td>
<td>17</td>
<td>Goble et al. (1981)</td>
</tr>
<tr>
<td>Sign</td>
<td>0.22</td>
<td></td>
<td>26</td>
<td>Different sources</td>
</tr>
<tr>
<td>Sign</td>
<td>0.10</td>
<td></td>
<td>10</td>
<td>Skov (1988)</td>
</tr>
<tr>
<td>Sign</td>
<td>0.13</td>
<td></td>
<td>14</td>
<td>Present Investigation</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Type of Parameter</th>
<th>Type</th>
<th>Quake (Q)</th>
<th>Damping (J)</th>
<th>Hammer Efficiency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soil</td>
<td>Inches</td>
<td>Sec/ft</td>
<td>Side</td>
</tr>
<tr>
<td>Minimum</td>
<td>Clay</td>
<td>0.30</td>
<td>0.20</td>
<td>0.01</td>
</tr>
<tr>
<td>Average</td>
<td>Clay</td>
<td>0.10</td>
<td>0.20</td>
<td>0.01</td>
</tr>
<tr>
<td>Maximum</td>
<td>Clay</td>
<td>0.05</td>
<td>0.10</td>
<td>0.01</td>
</tr>
<tr>
<td>Minimum</td>
<td>Sand</td>
<td>0.20</td>
<td>0.07</td>
<td>0.20</td>
</tr>
<tr>
<td>Average</td>
<td>Sand</td>
<td>0.10</td>
<td>0.05</td>
<td>0.15</td>
</tr>
<tr>
<td>Maximum</td>
<td>Sand</td>
<td>0.05</td>
<td>0.03</td>
<td>0.05</td>
</tr>
</tbody>
</table>
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>
<thead>
<tr>
<th>PILE</th>
<th>LOAD TEST</th>
<th>CRD</th>
<th>PDA</th>
<th>WEAP</th>
<th>WISDOT FORMULA</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>375(F)</td>
<td>380</td>
<td>245</td>
<td>405</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>360+</td>
<td>180</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>300(F)</td>
<td>310</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>380(F)</td>
<td>330</td>
<td>240</td>
<td>420</td>
<td></td>
</tr>
<tr>
<td>SP</td>
<td>425+</td>
<td>230</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**
- **F** = actual failure load
- **F** = constant-rate-of-penetration
- **WISDOT** = Wisconsin Modified ENR
- **W** = 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.

<table>
<thead>
<tr>
<th>FORMULA</th>
<th>RATIO MEAN</th>
<th>RATIO COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENR</td>
<td>0.49</td>
<td>0.54</td>
</tr>
<tr>
<td>Hiley</td>
<td>1.11</td>
<td>0.54</td>
</tr>
<tr>
<td>Gates</td>
<td>1.71</td>
<td>0.40</td>
</tr>
<tr>
<td>Janbu</td>
<td>1.24</td>
<td>0.46</td>
</tr>
</tbody>
</table>

**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 appropriate values for many of the input items, such as hammer efficiency, coefficients of restitution, distribution of side friction forces, etc. The PDA allows direct measurement of some of these inputs. However, while the pile analyzer has generally been found to be very successful, it does err considerably in some cases. This continuing uncertainty about the results, together with its significant additional expense, currently prevents universal use of the PDA.
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

The objective of this study was to examine available information regarding the use of dynamic methods employed by the Louisiana Department of Transportation and others and to recommend an approach that will be an improvement over the current dependence on the Engineering News Record formula. Several tasks were identified in an attempt to meet this goal. A literature survey and other inquiries were used to identify dynamic methods used in monitoring pile driving. The identification process included consideration of the philosophies and methods actually employed and those being considered by the Louisiana DOTD and other state transportation departments. Current usage of empirical formulas, the wave equation, and the pile analyzer, as well as previous research efforts by the Louisiana DOTD on this subject, were reviewed and are reported herein.

A comparative study of dynamic methods based on local information and experiences is included. A test pile database was assembled using DOTD files. Computer software was developed for the creation of a computer data file for each pile selected. One objective of this part of the study was to assemble pile load test records that contain at least a bare minimum of the information needed for evaluation of dynamic predictions. Records included the information documenting the hammer, pile, and soil details and apply to piles loaded to failure or to a point sufficient for reasonably accurate determination of pile capacity using accepted methods. The various techniques for interpreting pile capacity from a static load test were reviewed and a consistent method was selected. 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

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

THE 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 records with the analyzer were available for this study. An ton 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) gain 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
- \(S\) = Weight of ram \(W_r\), kips, * height of fall \(H\), inches, for free falling rams
- \(C\) = Loss constant, inches
- \(C = 1.0\) for drop hammers
- \(C = 0.1\) 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
the above formula, a safety factor of 6.0, and conversion to tons. Variations to this formula include application hammer efficiency ratio to the energy and adjustments to the constant.

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

\[
P = \frac{2W_hH}{S + C}
\]

here:

\[
P = \text{Safe bearing capacity, pounds}
W_h = \text{Weight of hammer ram, pounds}
H = \text{Height of fall, feet}
S,C = \text{As defined above}
\]

6-Engineering-News-Record

The ENR (ENRMOD) is presented in *Formulated Pile Loads For Active and Tied Down Methods* (22) a manual of methods for DOTD inspectors. It is an attempt to account for the pile with respect to the weight of the ram. In allowable load format it is written as follows:

\[
P = \frac{2W_hH}{S + 0.1(W_p/W_h)}
\]
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:

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

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.
Gates formula can be expressed as:

\[ P_u = a \times \sqrt{E} \times (b - \log s) \]

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"
- \( \times \) = Abbreviation for "multiply"
- \( \log \) = Abbreviation for base ten logarithm
- \( s \) = Pile set, in., due to final hammer blow

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

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
- \( k_u = C_d(1 + \sqrt{1 + u/C_d}) \)
- \( C_d = 0.75 + 0.15(W_p/W_e) \)
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.

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

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

The information contained on the data extraction forms was transferred into computer files so it could be readily and rapidly accessed and used. Use of a proprietary database program was considered, since the resulting data files would have to be accessed by a data analysis program, it was decided to custom write the data creation software.

A typical computer file for one of the test piles is shown in Figure 3. Every second line in the file contains one piece of information. The preceding line in each case describes the piece of information to follow. This was done to facilitate changes to the file after it has been created and to minimize the chances of a piece of information in the wrong place. The files are standard ASCII files.
Louisiana Department of Transportation and Development
Pile Test Data Bank Entry Form

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
soil strength:

ground soil (1 = sand, 2 = stiff to med clay, 3 = med to soft
soft to very soft clay):

concrete skin friction:

set-up factor from end of driving to start of testing:

pile: cross sections to be input: (number 1 at top)

A of pile with section numbers, x-sector area, sq in,

A of pile (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)

A of soil profile with layer numbers, soil descriptions,

A, ft, total unit weights, pcf, angles of friction, degrees,

and compressive strengths, psf, % moisture content,

dry unit weight, % plasticity index, each layer

weight, kips (Pile cap includes capblock and helmet
noses called hood); alternating layers of 1" micarta

or aluminum generally used)

stiffness, kips/inch (capblock is source of spring):

assumed pile cushion type, if any (cushions generally

only with concrete piles):

joint stiffness, kips/inch:

light kips (anvil only on diesel hammers):

efficient of restitution (cor):

r = 0.85

joint cor (if cushion used):

r = 0.85

cor for formulas:

ing factor:

jumping factor:

or skin friction:

or point resistance:

action distribution type (number from table):

ed Pile Failure Load, kips, by Weap86:

ed WEAP Failure Load, kips using default values

all 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)
DESCRIPTION OF HAMMER

VULCAN NO. 1

HMR TYPE CODE (1=SAAS, 2=DAAS, 3=OED, 4=CED, 5=DROP, 6=OTHER)

RAM NUMBER FROM TABLE

204

WEIGHT OF HAMMER RAM, KIPS

5.00

TOTAL WEIGHT OF HAMMER, KIPS

10.00

MAXIMUM RATED ENERGY, FT-KIPS

15.0

SPEED OF HAMMER RAM, BLOWS/MIN

55.0

MAXIMUM ENERGY EFFICIENCY RATIO

0.670

DESIGN LOAD PER PILE, TONS

57.40

MAXIMUM TEST LOAD, TONS

143.50

RATION OF MAXIMUM LOAD, HOURS

2.00

TOTAL PILE DEFLECTION AT MAXIMUM LOAD, IN

0.19

PILE DEFLECTION AT 50 % OF MAXIMUM LOAD, IN

0.03

PILE DEFLECTION AT 75 % OF MAXIMUM LOAD, IN

0.08

PERMANENT DEFLECTION DUE TO TEST LOAD, IN

0.038

ESTIMATED RATIO TEST LOAD TO FAILURE LOAD AT TIME OF TESTING

0.951

NAME OF METHOD USED TO CALC THIS RATIO

Van der Veen

NUM FREDOM 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

286.000

SIDE FRICTION PERIM, IN, SECTION 1

64.000

MODULUS OF ELASTICITY, KSI, SECTION 1

3640.000

UNIT WEIGHT, PCF, SECTION 1

150.000

Figure 3. (cont)
A FORTRAN computer program, PILLET, was written to allow interactive transfer of information from the data extraction form to the computer file. Upon running PILLET from a terminal, the operator is prompted for each piece of information in the same sequence as it is on the form. A listing of PILLET is available upon request from the authors.

In addition to creating an ASCII file named "LATP.xxx," where "xxx" is the file number, PILLET adds each pile to a cumulative catalogue file named "LATP.CAT," which stores the pile number and certain key information. A listing of the catalogue file is given in Figure 4.

**Test Pile Failure Loads**

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

There are many alternate methods of determining "failure" load from a given load versus deflection curve for a test pile. It is difficult to say which of these often very different failure loads should correlate with the predicted ultimate load of a given dynamic method. There is some logic to assuming that the test load corresponding to a deflection equal to the penetration of the final hammer blow should correlate to this prediction. This deflection may be considerably more or less than what actually constitutes 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  

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Figure 4. (cont)
Several methods for determining pile capacity were considered. These included the Van der Veen (27), Mazurkiewicz-Davisson (28), Chin (29), AASHTO, and Swedish Ninety percent Criterion technique. After reviewing these methods and existing load tests, it was determined that several could be used. Many of the test piles had not been loaded far enough to produce a load-settlement curve required by some of the methods, the requirements of the method did not fit test procedures, conditions, etc. The Van der Veen and Mazurkiewicz were considered possible candidates.

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

Van der Veen proposed the following relation between applied load and 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 the load-deflection curve, \( Q_u \) and \( r \) can be determined.
Specifications [1] require a load test when the bearing capacity computed by the ENR formula is less than twice the design capacity. The test loading consists of the sum of incremental static loads on the pile and measuring the corresponding settlement. Test piles are loaded to failure or 20 times the design load is reached. The test pile is considered to have failed when the permanent settlement at the top of the pile is 1/4 inch (regardless of pile size). The Van der Sloot method worked very well for most of these test pile records.

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

In southeast Louisiana, most piles increase significantly in capacity during the first few weeks after driving, as much as 400 to 500 percent [16]. Logically, the end-of-driving cannot be expected to predict the pile capacity at the time of driving, it cannot predict a significantly different capacity at the time of the load test (or the time of designing for the typical production pile). In recognition of this, the practice of "restriking" is growing. Restriking refers to driving 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.
should be noted that practically all evaluations of dynamic methods to date have used end-of-driving blowcounts to predict pile capacity. This capacity has been compared to the load test result regard to setup. It might be argued that any time-dependent changes are considered indirectly through the customary safety factors that have been settled upon through the observation of satisfactory results. However, it is likely that these safety factors are higher than would be required if the time-dependent effects in each pile could be better quantified and included in the prediction process.

should also be noted that the Gates formula included in this study is different from the other methods in that it was derived by a statistical correlation between end of driving blowcounts and load test results. Thus, it includes the effect of setup that occurred before the load test on the average for that group of piles on which the formula development was based. Because of the extreme variation in amounts of setup that occurs, it is unlikely that this method of dealing with setup can be widely accurate.

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

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

\[
SUF = S(P_s) + 1.0(P_e)
\]
where:

\[ P_s = \text{Fraction of total pile resistance coming from side friction} \]

\[ P_t = \text{Fraction of total pile resistance coming from tip bearing} \]

\[ S = 1.0 \text{ if predominant side soil has high permeability (sand or gravel)} \]
\[ = 2.0 \text{ if predominant side soil is medium to stiff clay} \]
\[ = 3.0 \text{ if predominant side soil is soft to medium clay} \]
\[ = 4.0 \text{ if predominant side soil is very soft to soft clay} \]

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

\[ P_s = 0.95 \text{ if the final blowcount is less than 3.5 times the average blowcount} \]
\[ = 0.75 \text{ if the final blowcount is between 3.5 and 4.0 times the average blowcount} \]
\[ = 0.50 \text{ if the final blowcount is greater than 4.0 times the average blowcount} \]

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 soil is hard clay, sand, gravel, or rock, this is probably an assumption. For soft clays, it is probably a good assumption for soft clays also since excess pore pressure can effectively shear the axial compressive stresses that occur at the pile tip. Thus, the capacity of a pile tip in soft submerged clay is very less than 5% of the total capacity, and factors relating to the setup of that tip capacity are not very small.

Setup factors for pile side friction were decided upon through the literature. Lowery recommended setup factors of 3 for loose clays, 2 for firm and stiff clays, and 1 for firm clays. Under the assumption that their wave equation was accurately predicting end-of-driving capacities, Lee gave pile setup factors in the form of the "R" cited in the literature review above.

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

The end of estimating percent skin friction that was not dependent on analysis 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
values of each dynamic method were compared to test pile
load data (4). Because of the large geometric similarity
between test piles, good load vs. capacity relations
were obtained. Over-all, the wave equation requires
by far the most input variables. While the literature
contains guidelines 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
determine the effect of "reasonable" variations in these
inputs on predicted blowcount vs. capacity relation (31).
Results of study indicated that reasonable variations in
hammer energy, coefficients of restitution, damping factors, and quake
amplitude can have a very significant effect on this relation.

Input Selected

On how some of the WEAP87 input values were selected for
the project are given below. Complete copies of the input data
for the selected test piles are available by request from the

Input for the other formulas is discussed in a later
section...
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.
side quake values used were 0.1 inch; point and side soil factors (Smith damping) used were 0.15 seconds per foot.

In Ratios

Ratios were calculated to compare the performances of the formulas. The R numbers are ratios of the measured pile loss to predicted pile capacities. There are three different predicted capacities for each pile; namely, 1) the actual maximum load, 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 pile are 1) the theoretical predicted ultimate capacity of a formula and 2) twice the customary allowable load given by nla. Thus there are six combinations of measured and predicted capacities.

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

\[ R1 = \frac{\text{Maximum Test Load}}{\text{Formula Predicted Capacity}} \]

\[ R2 = \frac{\text{Maximum Test Load}}{\text{Formula Allowable times 2.0}} \]

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

At R ratios, R3 and R4, compare the failure load as predicted by the Van der Veen method with the predictions of the C formula. These ratios constitute the "without consideration of setup" comparison.

\[ R3 = \frac{\text{Test Failure Load}}{\text{Formula Predicted Capacity}} \]

\[ R4 = \frac{\text{Test Failure Load}}{\text{Formula Allowable times 2.0}} \]

At two R ratios attempt to account for setup between the time the pile was driven and the time of the load test.
R5 = Test Failure Load divided by Setup / Formula Predicted Capacity
R6 = Test Failure Load 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 EMBEDMENT, 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 DEF, IN, AT 50% MAX LOAD = 0.030
PILE DEF, 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 FRICITION, 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
The page contains detailed engineering specifications and calculations. Here is the plain text representation:

**PLASTICITY INDEX = 11.0**

**CLAYEY SILTY SAND**

**TOTAL UNIT WT, PCF = 120.0**

**FRICTION, DEGREES = 30.0**

**COMPRRESSIVE STRENGTH, PSF = 0.0**

**TENT% = 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**

**DAMPING FACTOR, SEC/FT = 0.05**

**TYPING FACTOR = 0.15**

**ACTION QUAKE, IN = 0.10**

**POINT QUAKE = 0.13**

**ACTION DISTRIBUTION TYPE NUMBER = 3**

**LOAD, KIPS, PREDICTED BY WEAP66 = 99.000**

**LOAD, KIPS, WEAP WITH DEFAULT INPUT = 106.000**

**TEST LOAD, TONS = 143.50**

**SURE LOAD, TONS, AT TESTING TIME = 150.89**

**SURE LOAD, TONS, AT END OF DRIVING = 150.89**

**PORION SKIN FRICTION AT EOD BASED ON**

**IN BETWEEN AVG AND FINAL BLOW COUNTS = 0.950**

**FACTOR BASED ON PREDOM SIDE SOIL AND ESTIM**

**K FRICTION = 1.000**

**KIPS = 0.932E+06**

**SIDE FRICT TO TOTAL BASED ON SOIL PROFILE**

**TERM1 = .090 END OF DRIVING = 0.696**

**END OF DRIVING ULTIMATE CAP, TONS = 99.30**

**LONG TERM ULT CAP, TONS = 99.30**

**------------------------------**

**RED NOM SF CUST ADJ P ULT PPSF ADJ PPSF ALLOW, T**

<table>
<thead>
<tr>
<th>Depth</th>
<th>Nom SF</th>
<th>Cust</th>
<th>Adj P</th>
<th>Ult</th>
<th>PPSF</th>
<th>Adj PPSF</th>
<th>Allow, T</th>
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<td>11.8</td>
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</table>

**Figure 5. (cont)**

51
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

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<th>R3</th>
<th>R4</th>
<th>R5</th>
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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 ? 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)
Out for files 092 and 093 similar, omitted for brevity

### PRIMARY STATISTICS

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<th>R6</th>
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<td>1.11</td>
<td>1.572</td>
<td>0.71</td>
<td>1.11</td>
</tr>
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</table>

### RAW DATA OUTPUT, SHORT TONS

**Y** = VAN DER VEE LOAD TEST ULT LOAD

**D** = VDV LOAD REDUCED TO END OF DRIVING BY EST SETUP

NUMBERS REFER TO METHODS AS NUMBERED ABOVE

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**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
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:

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

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

**I-310 Advance Test Pile Program** - 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
and in-place dynamic tests of those piles. The "Special
or" of the construction contract for this job required the
or to submit a wave equation analysis of all test piles
ith the approved hammer to the Bridge Design Engineer prior
sing work. The piles were driven with a Delmag D46-23
cing diesel hammer to an 80-foot tip penetration.
valuations of pile capacity, driving stresses, and hammer
ence 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
iles. All test piles except one were statically loaded
re at a time interval of approximately 14 days from the
ik test. A quick-load test was used for testing the
ile capacity (35). Results of the test sequences used for
astic 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.

MENT OF IMPLEMENTATION SOFTWARE

The tasks of the project was the creation of a field use
allow convenient application of a superior dynamic
ion 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
cription 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. (12) 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
switch to wave equation analyses due to the resulting
increase in accuracy. Comparative studies of pile driving formulas
conducted by some states were found to be "either quite old...or
obsolete." A few states had previously conducted a comparative
study of pile load tests with formulas and/or the wave equation.
Four states were conducting such a study at the time of the survey,
while one was considering such an investigation in the near future.

Although their study was not complete, Pennsylvania transportation
ingeers indicated that they were finding that the wave equation
and pile analyzer underpredict pile capacity if the pile does not
exhibit relaxation.

Of the 47 states responding to the Washington State survey indicated they
use the wave equation. Only two states indicated regular use of
the pile analyzer, but they were very satisfied with it. It was
generally noted in the report that although "these methods clearly are more
difficult to implement, and require more highly trained personnel,
the intermediate step, using a more sophisticated equation, does
not seem to have been considered."

The Washington State Department of Transportation procedure for
construction control of pile foundations, as presented in the
McAsey et al. report (20), is similar to that used by many other
states. The Engineering News Record is used for estimation of pile
capacity and construction control of small pile driving jobs. This
includes the majority of pile driving projects. For interstate
construction and larger projects, the wave equation and pile
analyzer are used. Outside contractors are used when the pile
analysts do not possess the in-house capability for this dynamic test.

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

\[ P_{allow} = \frac{2WH}{S + 0.2} \]

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
In the predicted design capacities. New York DOT initially estimates the pile type and lengths with sampling and testing information (21). Pay items for static tests and dynamic load tests are established. The contractor is the responsibility for determining the lengths of piles to be driven. All piles not driven to refusal or bedrock must be tested to a penetration that satisfies the ENR formula. After driving experiences at the site, a decision is made between static tests (or both). Static load tests are generally performed only for relatively large projects. The Ohio DOT has been utilizing dynamic tests since the mid-1970s. A Pile Driving Research project became the property of the Ohio DOT at the conclusion of a research project on pile capacity at Case Western Reserve University. Since that time, the usage of and reliance on dynamic testing has greatly increased relative to static testing. The wave equation has been used on selected projects.

with Carolina DOT (38) found that the wave equation analysis was a means of not only increasing the Engineer's confidence in required capacity is achieved but that the pile is not overstressed and that the pile driving hammer is capable of driving the pile to the desired depth. Since 1977, the WEAP program has been used unofficially on pile projects to experience. Methods of correlating the wave equation computer with piles, load tests were practiced on at least four projects per year. The wave equation has been used since 1980 to predict pile driving. However, the ENR formula is still generally used on bridges with steel H-piles driven to rock. As part of 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:
<table>
<thead>
<tr>
<th>Construction Control Method</th>
<th>Recommended Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static Load Test</td>
<td>2</td>
</tr>
<tr>
<td>Dynamic Measurement coupled</td>
<td>2.5</td>
</tr>
<tr>
<td>with Wave Equation Analysis</td>
<td></td>
</tr>
<tr>
<td>Wave Equation Analysis</td>
<td>3</td>
</tr>
</tbody>
</table>

PERFORMANCE OF DYNAMIC FORMULAS WITH LOUISIANA TEST PILES

EVALUATION OF CANDIDATE METHODS

The database test piles were grouped in certain categories 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:

\[ R_3 = \frac{\text{Test Failure Load}}{\text{Formula Predicted Capacity}} \]

\[ R_4 = \frac{\text{Test Failure Load}}{\text{Formula Allowable} \times 2.0} \]

\[ R_5 = \frac{\text{Test Failure Load divided by Setup}}{\text{Formula Predicted Capacity}} \]

\[ R_6 = \frac{\text{Test Failure Load divided by Setup}}{\text{Formula Allowable} \times 2.0} \]
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 WEAAP87, 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

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

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

<table>
<thead>
<tr>
<th>Mean</th>
<th>COV</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>R3</td>
<td>R4</td>
<td>R3 and R4</td>
<td>R5</td>
</tr>
<tr>
<td>0.436</td>
<td>11.308</td>
<td>0.30</td>
<td>0.389</td>
</tr>
<tr>
<td>1.429</td>
<td>22.143</td>
<td>0.24</td>
<td>1.280</td>
</tr>
<tr>
<td>1.643</td>
<td>21.464</td>
<td>0.24</td>
<td>1.471</td>
</tr>
<tr>
<td>1.570</td>
<td>63.758</td>
<td>0.24</td>
<td>1.497</td>
</tr>
<tr>
<td>2.056</td>
<td>84.112</td>
<td>0.24</td>
<td>1.841</td>
</tr>
<tr>
<td>1.728</td>
<td>11.728</td>
<td>0.24</td>
<td>1.553</td>
</tr>
</tbody>
</table>

*Test Failure Load / Formula Predicted Capacity*

*Ultimate Failure Load / Formula Allowable times 2.0*

*Ultimate Failure Load divided by Setup / Formula Predicted Capacity*

*Ultimate Failure Load divided by Setup / Formula Allowable times 2.0*
### TABLE 8
PILE FORMULA CAPACITIES

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

* 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**

<table>
<thead>
<tr>
<th>Method</th>
<th>Mean R3</th>
<th>Mean R4</th>
<th>R3 and R4 COV</th>
<th>Mean R5</th>
<th>Mean R6</th>
<th>R5 and R6 COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENR</td>
<td>0.619</td>
<td>1.858</td>
<td>0.60</td>
<td>0.360</td>
<td>1.079</td>
<td>0.53</td>
</tr>
<tr>
<td>Hiley</td>
<td>1.609</td>
<td>2.414</td>
<td>0.42</td>
<td>0.993</td>
<td>1.489</td>
<td>0.50</td>
</tr>
<tr>
<td>Gates</td>
<td>2.032</td>
<td>3.048</td>
<td>0.46</td>
<td>1.259</td>
<td>1.888</td>
<td>0.54</td>
</tr>
<tr>
<td>Janbu</td>
<td>2.204</td>
<td>4.959</td>
<td>0.49</td>
<td>1.298</td>
<td>2.921</td>
<td>0.48</td>
</tr>
<tr>
<td>ECUBC</td>
<td>2.828</td>
<td>5.657</td>
<td>0.47</td>
<td>1.678</td>
<td>3.355</td>
<td>0.49</td>
</tr>
<tr>
<td>NEAP87</td>
<td>2.728</td>
<td>2.728</td>
<td>0.60</td>
<td>1.529</td>
<td>1.529</td>
<td>0.48</td>
</tr>
</tbody>
</table>

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

---

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### TABLE 10
**PILES BEARING IN SAND: PILE FORMULA CAPACITIES**

#### R - Ratios

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

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

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

VIEW OF I-310 TEST PILE PROGRAM

The I-310 North Approach to the Luling Bridge is an elevated roadway that crosses environmentally sensitive wetlands. In order to confine the construction activity and to cause the least disturbance to this area, an end-on construction technique was selected. In this method, the elevated roadway is advanced by building off the previously completed section. The customer's decision to perform static load tests of roadway bent pile is not permitted for environmental reasons. Thus, a higher th...
usual reliance on dynamic methods was required. The methods considered for monitoring the pile performance and load transferred included the pile driving analyzer (PDA) and the Shock Measured Response Transit (Transient Dynamic Response Testing Technique). A correlation of load test results with electronic cone penetrometer tests (ECPT) was also considered. The ECPT soundings were to be used as a means to establish pile tip elevations at the bent locations.

In order to verify the proposed pile evaluation techniques, an advance test pile program was conducted at a nearby accessible site. The test site was located in St. Charles Parish at the intersection of US 61 and I-310, North Approach to the Luiz Bridge. The location and arrangement of the piles are shown in 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 the 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
Figure 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
using the initial pile installation and subsequently in a
scheduled restrikes. The restrike tests were conducted
mous setup periods that varied from 1 to 22 days. For
static quick-load tests were conducted at the end of
period. A series of static load tests were conducted on
different setup periods.

The shock test requires placement of a small loadcell centrally
ile and a geophone near the circumference of the pile head.
A geophone and loadcell are connected, the loadcell is
with a light, hand-held hammer. According to the Special
for this job, "data is obtained for determining the pile
action," Results of this test did not support its
ition for determination of pile capacity. However, during
fact, 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
ments also indicated that TP5 cracked during driving.
 tests were conducted by CEBTP Limited, 2201 Wisconsin
Suites 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
ving stresses, and hammer performance were conducted
the CASE Method with the PDA measurements. Results were
to the pile driving contractor, Louisiana Paving Company
, La. CAPWAPC (microcomputer version of CAPWAP) analyses
also 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
ted 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 GRI (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
<table>
<thead>
<tr>
<th>Pile Test</th>
<th>Date</th>
<th>Blow Count</th>
<th>Average Energy Transfer</th>
<th>Average Max Measured Compressive Force* Stress</th>
<th>Computed Tensile Force* Stress</th>
<th>PDA** Load Test</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP1 EOD</td>
<td>6/10</td>
<td>38</td>
<td>16</td>
<td>1200 1.56 kips/ft kips ksi</td>
<td>300 0.39 kips ksi</td>
<td>169 84.5 kips tons kips tons</td>
<td>54&quot; cylinder</td>
</tr>
<tr>
<td>RSTK</td>
<td>6/12</td>
<td>(42/1&quot;)</td>
<td>12</td>
<td>1100 1.43 kips/ft kips ksi</td>
<td>160 0.21 kips ksi</td>
<td>551 275.5 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>6/19</td>
<td>(43/1.25&quot;)</td>
<td>17</td>
<td>1520 1.97 kips/ft kips ksi</td>
<td>120 0.61 kips ksi</td>
<td>658 329 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>7/2</td>
<td>(43/0.25&quot;)</td>
<td>16</td>
<td>1465 1.91 kips/ft kips ksi</td>
<td>41 0.05 kips ksi</td>
<td>797 398.5 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>TP2 EOD</td>
<td>6/10</td>
<td>48</td>
<td>12</td>
<td>930 1.21 kips/ft kips ksi</td>
<td>330 0.49 kips ksi</td>
<td>160 80 kips tons kips tons</td>
<td>54&quot; cylinder</td>
</tr>
<tr>
<td>Static</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>6/19</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>7/2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP3 EOD</td>
<td>6/8</td>
<td>10</td>
<td>23</td>
<td>820 1.77 kips/ft kips ksi</td>
<td>300 0.65 kips ksi</td>
<td>60 30 kips tons kips tons</td>
<td>24&quot; square</td>
</tr>
<tr>
<td>RSTK</td>
<td>6/9</td>
<td>(6/1&quot;)</td>
<td>25</td>
<td>950 2.05 kips/ft kips ksi</td>
<td>260 0.56 kips ksi</td>
<td>205 102.5 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>6/18</td>
<td>(12/3&quot;)</td>
<td>18</td>
<td>950 2.04 kips/ft kips ksi</td>
<td>183 0.40 kips ksi</td>
<td>344 172 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>6/26</td>
<td>(21/1.75&quot;)</td>
<td>15</td>
<td>996 2.15 kips/ft kips ksi</td>
<td>226 0.43 kips ksi</td>
<td>376 188 kips tons kips tons</td>
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</tr>
<tr>
<td>Static</td>
<td>7/29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP4 EOD</td>
<td>6/8</td>
<td>14</td>
<td>24</td>
<td>1400 2.24 kips/ft kips ksi</td>
<td>640 1.02 kips ksi</td>
<td>45 22.5 kips tons kips tons</td>
<td>30&quot; square</td>
</tr>
<tr>
<td>RSTK</td>
<td>6/9</td>
<td>(16/8.5&quot;)</td>
<td>34</td>
<td>1750 2.80 kips/ft kips ksi</td>
<td>600 0.96 kips ksi</td>
<td>200 100 kips tons kips tons</td>
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</tr>
<tr>
<td>&quot;</td>
<td>6/12</td>
<td>(9/2&quot;)</td>
<td>36</td>
<td>1900 3.04 kips/ft kips ksi</td>
<td>570 0.91 kips ksi</td>
<td>292 146 kips tons kips tons</td>
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</tr>
<tr>
<td>&quot;</td>
<td>6/17</td>
<td>(12/0.5&quot;)</td>
<td>32</td>
<td>1980 3.17 kips/ft kips ksi</td>
<td>560 0.90 kips ksi</td>
<td>341 170.5 kips tons kips tons</td>
<td></td>
</tr>
<tr>
<td>&quot;</td>
<td>6/26</td>
<td>(21/1.75&quot;)</td>
<td>25</td>
<td>1310 2.09 kips/ft kips ksi</td>
<td>260 0.42 kips ksi</td>
<td>360 180 kips tons kips tons</td>
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</tr>
<tr>
<td>Static</td>
<td>7/9</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

* Force measured 4 ft below pile top
** Calculated with Pile Driving Analyzer Measurements
<table>
<thead>
<tr>
<th>Pile</th>
<th>Test</th>
<th>Date</th>
<th>Blow Count</th>
<th>Average Energy Transfer</th>
<th>Average Max. Measured Compressive Force* Stress</th>
<th>Maximum Computed Tensile Force* Stress</th>
<th>Axial Bearing Capacity</th>
<th>PDA** Load Test</th>
<th>Remarks</th>
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<tr>
<td>TP5A</td>
<td>EOD</td>
<td>6/25</td>
<td>23</td>
<td>780 1.25</td>
<td>670 1.07</td>
<td>59 29.5</td>
<td>30&quot;</td>
<td></td>
<td></td>
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<tr>
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<td>RSTK</td>
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<td>(44/9&quot;)</td>
<td>1110 1.77</td>
<td>430 0.69</td>
<td>214 107</td>
<td>157.5</td>
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<td>spliced replacement</td>
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<td></td>
<td>6/29</td>
<td>(24/3&quot;)</td>
<td>1150 1.84</td>
<td>300 0.48</td>
<td>315 178.5</td>
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<td></td>
<td></td>
<td>7/6</td>
<td>(36/4.75&quot;)</td>
<td>1231 1.97</td>
<td>273 0.44</td>
<td>357 178.5</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7/15</td>
<td>(27/1.5&quot;)</td>
<td>1130 1.81</td>
<td>284 0.46</td>
<td>393 196.5</td>
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<td></td>
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<td></td>
<td>7/29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>534 267</td>
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<td>TP6</td>
<td>EOD</td>
<td>6/15</td>
<td>15</td>
<td>910 1.87</td>
<td>615 1.26</td>
<td>90 45</td>
<td>36&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td>(34/10&quot;)</td>
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<td>580 1.19</td>
<td>199 99.5</td>
<td>139.5</td>
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<tr>
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<td></td>
<td>6/19</td>
<td>(24/4.5&quot;)</td>
<td>1120 2.30</td>
<td>330 0.68</td>
<td>279 139.5</td>
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<td></td>
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<tr>
<td></td>
<td></td>
<td>6/26</td>
<td>(18/2&quot;)</td>
<td>1120 2.30</td>
<td>260 0.53</td>
<td>397 198.5</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>7/6</td>
<td>(33/3.5&quot;)</td>
<td>1170 2.40</td>
<td>198 0.41</td>
<td>517 258.5</td>
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</tr>
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<td></td>
<td>7/20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>530 265</td>
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<td>660 1.35</td>
<td>413 0.77</td>
<td>102 51</td>
<td>36&quot;</td>
<td></td>
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<td></td>
<td>RSTK</td>
<td>6/17</td>
<td>(20/7.5&quot;)</td>
<td>1110 2.28</td>
<td>120 0.25</td>
<td>197 98.5</td>
<td>143.5</td>
<td></td>
<td>cylinder, spliced</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6/20</td>
<td>(17/2&quot;)</td>
<td>1210 2.48</td>
<td>170 0.35</td>
<td>287 143.5</td>
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<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6/26</td>
<td>(22/1.25&quot;)</td>
<td>1250 2.57</td>
<td>150 0.31</td>
<td>425 212.5</td>
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<tr>
<td></td>
<td></td>
<td>7/6</td>
<td>(31/2&quot;)</td>
<td>1250 2.59</td>
<td>213 0.99</td>
<td>508 254</td>
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<td></td>
<td></td>
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<tr>
<td>Static</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>526 263</td>
<td></td>
</tr>
</tbody>
</table>

* 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
poured pile head force waves.)

Tests and predictions were conducted on TP1 concurrent
tests on TP2; both TP1 and TP2 are 84-ft-long, 54 in. x
indical piles, in similar soil environments. At the end
initial 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
ote any regular pattern; however, the test loads for TP2
istently lower than the PDA-predicted capacities for TP1
ime times, Figure 8.

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

viously discussed, setup is a gradual increase in capacity
es in clay or other soils with low permeability. The gain
resistance can continue over long periods of time, with the
id increases generally occurring within the first few days.
values for the test piles of this study also indicate the
of the size and shape of the pile. In comparing setups of different piles, the gains in pile capacities occurring during testing for the 24 and 30 inch square piles were generally larger than the capacity gains for the 54- and 36-inch piles.

In Figure 9, the gains in capacities for these test piles are approximately linear when plotted against the log of time. It is stated that the time-dependent increase in a pile's stabilizes after some time, $t_0$, beyond the initial driving. Estimates of bearing capacity based on measurements from driving or on redriving performed at times $t < t_0$ have proved to be unreliable. Thus the EOD estimates of pile are not included in Figure 9. The resulting linear fits for the seven piles seem to indicate similar patterns of capacity for similar pile types. For example, consider the gains in capacities for the cylindrical piling, TP1, TP2, TP6.

Although there is a difference in the magnitude of the capacities for the different size piles, the rate of increasing capacities is similar. The differences between the TP1 and TP6 linear fits may also be influenced by the different testing i.e., the PDA test of TP1 and the static load test of TP2. The "larger displacement piles" also had a common pattern of increase that was different than the pattern for the smaller piles. The regression formulas for the variations in capacities with the log$_{10}$ of time for the seven piles are as follows:

- **Pile 1 (54"x5"):** $P = 235.32 + 114.34 \log_{10} t$
- **Pile 2 (54"x5"):** $P = 161.83 + 141.86 \log_{10} t$
- **Pile 6 (36"x5"):** $P = 87.85 + 115.69 \log_{10} t$
- **Pile 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 3 for TP1 and TP2, respectively), the displacements of a simulated static test are greater than those of TP1 for ing 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 d supplementing or eliminating the conventional static c. However, it was suggested that "CAPWAPC ultimate pile a and corresponding displacements should be checked the allowable pile head settlement, particularly for tly end bearing piles and in large quake soils (41)."

System Performance and Driving Stresses - As measured by the energy actually transferred to the pile was much less D45-23 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 occurred during the 6/12/87 test of TP4, Table 11. However, initial driving, it was necessary to operate the hammer at energies 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 diezel hammer being used, the combustion pressure and stroke increased or decreased. In a "Wave Equation Analysis" (42) prepared by Goble Rausche Likins (GRL) for Louisiana b, the pile driving contractor, it was recommended that 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.

WEAP versus Field Measurements - 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

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

Types: 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

Type Settings: 4 and 2

Cushion Material: Conbest

Thickness: 1 in.
Diameter: 23 in.

Elastic Modulus: 280 ksi
Stiffness: 116,200 k/in

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

<table>
<thead>
<tr>
<th>PILE TYPES</th>
<th>54&quot; x 5&quot;</th>
<th>36&quot; x 5&quot;</th>
<th>30&quot; x 30&quot;</th>
<th>24&quot; x 24&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taste (in)</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Take (in)</td>
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<td>0.1</td>
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<td>0.20</td>
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<tr>
<td>Weight (k)</td>
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<td>7.0</td>
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<tr>
<td>Cushion Thickness (in)</td>
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<td>6.0</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Cushion Elastic Mod. (ksi)</td>
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<td>30</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Cushion Area (in²)</td>
<td>770</td>
<td>486</td>
<td>900</td>
<td>576</td>
</tr>
<tr>
<td>Length (ft)</td>
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<td>84</td>
<td>84</td>
<td>84</td>
</tr>
<tr>
<td>Elastic Mod. (ksi)</td>
<td>6000</td>
<td>6000</td>
<td>5000</td>
<td>5000</td>
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</table>
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 be 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>
<thead>
<tr>
<th>Pile</th>
<th>Test</th>
<th>Date</th>
<th>Days After Driving</th>
<th>Quakes Skin in</th>
<th>Quakes Toe in</th>
<th>Smith Damping Skin sec/ft</th>
<th>Smith Damping Toe sec/ft</th>
<th>Ultimate Capacity Skin kips</th>
<th>Ultimate Capacity Toe kips</th>
<th>Ultimate Capacity Total kips</th>
<th>Average Unit Skin Friction k/ft²</th>
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<tbody>
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<td>0.203</td>
<td>0.417</td>
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<td>168.6</td>
<td>.11</td>
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<td>133.9</td>
<td>551.0</td>
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<td>658.0</td>
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<td>RSTK</td>
<td>7/2</td>
<td>0.135</td>
<td>0.463</td>
<td>0.317</td>
<td>604.4</td>
<td>193.0</td>
<td>797.4</td>
<td>.52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP2</td>
<td>EOD</td>
<td>6/10</td>
<td>0.15</td>
<td>0.20</td>
<td>0.355</td>
<td>0.255</td>
<td>10.6</td>
<td>149.0</td>
<td>159.7</td>
<td>.01</td>
<td></td>
</tr>
<tr>
<td>TP3</td>
<td>EOD</td>
<td>6/8</td>
<td>0.55</td>
<td>0.55</td>
<td>0.204</td>
<td>0.405</td>
<td>47.5</td>
<td>12.9</td>
<td>60.4</td>
<td>.07</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/9</td>
<td>0.12</td>
<td>0.12</td>
<td>0.112</td>
<td>0.428</td>
<td>175.0</td>
<td>30.0</td>
<td>205.0</td>
<td>.27</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/18</td>
<td>0.13</td>
<td>0.18</td>
<td>0.212</td>
<td>0.379</td>
<td>260.5</td>
<td>84.4</td>
<td>344.9</td>
<td>.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/26</td>
<td>0.19</td>
<td>0.19</td>
<td>0.291</td>
<td>0.277</td>
<td>282.7</td>
<td>94.2</td>
<td>376.9</td>
<td>.44</td>
<td></td>
</tr>
<tr>
<td>TP4</td>
<td>EOD</td>
<td>6/8</td>
<td>0.20</td>
<td>0.80</td>
<td>0.423</td>
<td>0.423</td>
<td>27.2</td>
<td>18.2</td>
<td>45.4</td>
<td>.03</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/9</td>
<td>0.12</td>
<td>0.75</td>
<td>0.306</td>
<td>0.435</td>
<td>149.7</td>
<td>49.9</td>
<td>199.5</td>
<td>.19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/12</td>
<td>0.20</td>
<td>0.30</td>
<td>0.399</td>
<td>0.363</td>
<td>146.0</td>
<td>146.0</td>
<td>292.1</td>
<td>.18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/17</td>
<td>0.175</td>
<td>0.30</td>
<td>0.463</td>
<td>0.338</td>
<td>134.7</td>
<td>207.2</td>
<td>341.9</td>
<td>.17</td>
<td></td>
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<tr>
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<td>RSTK</td>
<td>6/26</td>
<td>0.14</td>
<td>0.37</td>
<td>0.481</td>
<td>0.346</td>
<td>180.2</td>
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<td>.23</td>
<td></td>
</tr>
<tr>
<td>TP5A</td>
<td>EOD</td>
<td>6/25</td>
<td>0.20</td>
<td>0.70</td>
<td>0.101</td>
<td>0.101</td>
<td>15.6</td>
<td>43.6</td>
<td>59.2</td>
<td>.02</td>
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<tr>
<td></td>
<td>RSTK</td>
<td>6/26</td>
<td>0.20</td>
<td>0.70</td>
<td>0.449</td>
<td>0.203</td>
<td>95.5</td>
<td>118.5</td>
<td>214.0</td>
<td>.12</td>
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<tr>
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<td>6/29</td>
<td>0.20</td>
<td>0.42</td>
<td>0.473</td>
<td>0.333</td>
<td>202.0</td>
<td>113.0</td>
<td>315.0</td>
<td>.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>7/6</td>
<td>0.17</td>
<td>0.35</td>
<td>0.490</td>
<td>0.476</td>
<td>252.0</td>
<td>103.0</td>
<td>357.0</td>
<td>.32</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>7/15</td>
<td>0.23</td>
<td>0.375</td>
<td>0.359</td>
<td>0.428</td>
<td>293.2</td>
<td>100.7</td>
<td>393.9</td>
<td>.37</td>
<td></td>
</tr>
<tr>
<td>Pile</td>
<td>Test</td>
<td>Date</td>
<td>Days After Driving</td>
<td>Quakes Skin in</td>
<td>Quakes Toe in</td>
<td>Smith Damping Skin sec/ft</td>
<td>Smith Damping Toe sec/ft</td>
<td>Ultimate Capacity Skin kips</td>
<td>Ultimate Capacity Toe kips</td>
<td>Ultimate Capacity Total kips</td>
<td>Average Unit Skin Friction k/ft²</td>
</tr>
<tr>
<td>------</td>
<td>------</td>
<td>-------</td>
<td>-------------------</td>
<td>----------------</td>
<td>---------------</td>
<td>---------------------------</td>
<td>--------------------------</td>
<td>---------------------------</td>
<td>-----------------------------</td>
<td>----------------------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td>TP6</td>
<td>EOD</td>
<td>6/15</td>
<td>1</td>
<td>0.10</td>
<td>0.90</td>
<td>0.159</td>
<td>0.1</td>
<td>75.4</td>
<td>15.4</td>
<td>90.8</td>
<td>.10</td>
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<tr>
<td></td>
<td>RSTK</td>
<td>6/16</td>
<td>1</td>
<td>0.20</td>
<td>0.55</td>
<td>0.294</td>
<td>0.280</td>
<td>163.1</td>
<td>35.4</td>
<td>198.5</td>
<td>.22</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/19</td>
<td>4</td>
<td>0.15</td>
<td>0.15</td>
<td>0.420</td>
<td>0.277</td>
<td>236.3</td>
<td>43.0</td>
<td>279.2</td>
<td>.31</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/26</td>
<td>11</td>
<td>0.12</td>
<td>0.265</td>
<td>0.337</td>
<td>0.299</td>
<td>310.0</td>
<td>87.2</td>
<td>397.2</td>
<td>.41</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>7/6</td>
<td>21</td>
<td>0.12</td>
<td>0.285</td>
<td>0.251</td>
<td>0.180</td>
<td>428.0</td>
<td>89.0</td>
<td>517.0</td>
<td>.57</td>
</tr>
<tr>
<td>TP7</td>
<td>EOD</td>
<td>6/16</td>
<td>1</td>
<td>0.15</td>
<td>0.25</td>
<td>0.231</td>
<td>0.176</td>
<td>54.8</td>
<td>47.9</td>
<td>102.0</td>
<td>.07</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/17</td>
<td>1</td>
<td>0.35</td>
<td>0.60</td>
<td>0.273</td>
<td>0.355</td>
<td>160.7</td>
<td>36.1</td>
<td>196.8</td>
<td>.21</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/20</td>
<td>4</td>
<td>0.30</td>
<td>0.32</td>
<td>0.346</td>
<td>0.190</td>
<td>232.0</td>
<td>55.5</td>
<td>287.5</td>
<td>.31</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>6/26</td>
<td>10</td>
<td>0.22</td>
<td>0.385</td>
<td>0.209</td>
<td>0.309</td>
<td>360.8</td>
<td>64.3</td>
<td>425.1</td>
<td>.48</td>
</tr>
<tr>
<td></td>
<td>RSTK</td>
<td>7/6</td>
<td>20</td>
<td>0.20</td>
<td>0.280</td>
<td>0.252</td>
<td>0.254</td>
<td>421.6</td>
<td>86.4</td>
<td>508.0</td>
<td>.56</td>
</tr>
</tbody>
</table>
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:

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>54&quot;x5&quot;</th>
<th>36&quot;x5&quot;</th>
<th>30&quot;x30&quot;</th>
<th>24&quot;x24&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Blowcount at Higher Fuel Setting (Blows/Ft)</td>
<td>30</td>
<td>25</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>Minimum Blowcount at Reduced Fuel Setting</td>
<td>60</td>
<td>40</td>
<td>33</td>
<td>25</td>
</tr>
</tbody>
</table>

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
setting are in better agreement than are the WEAP energy
uses for the higher fuel setting. This may indicate that the
tractor was concerned with the potential for pile cracking and
therefore 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
g method for the square piles (TP3 and TP4), the higher fuel
setting is in better agreement. This is not the case for test pile
TP5; however, TP5A was a replacement for TP5, which cracked during
driving. DOTD records indicate that the TP5 pile was driven by the
tractor at a "high hammer energy which was against their
recommendation in a report sent to the contractor by his company
P, recommending that the low energy be delivered to the pile
in the resistance of the soil is weak." The average PDA-measured
energy that was delivered in driving the replacement pile, TP5A,
is in close agreement with the WEAP prediction for the reduced
fuel setting of the hammer, i.e., 15 kip-ft for the PDA and 15.4
kip-ft in the WEAP analysis.

WEAP pile capacities were also compared with the CASE
capacities, i.e., PDA measurements. Bearing graphs for the piles
are reproduced in Figures 18, 19, 20, and 21. Pile capacities are
presented for each of the test piles at the end of driving in
Figure 22. The WEAP capacities correspond to the hammer being
operated at the reduced, designated "2", and high, designated "4",
setting. The operation of the hammer at the reduced fuel
setting, resulted in a higher blowcount requirement to attain a
particular soil resistance since less energy was being put into the
system. The range of predicted pile capacities for each pile and
higher fuel setting are shown. In examining Figure 22, the WEAP-
predicted capacities are in most cases more than twice those
termined by the CASE method at the end of driving. There is an
greater difference when comparing the WEAP analyses at the
higher 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
Figure 23 compares the WEAP capacities corresponding to the
load at the end of driving to the load tested capacities. The
load tests were performed at times ranging from 22 to 35
days after the piles were initially driven as shown in Table 11.
In the range of WEAP-predicted pile capacities are shown for
piles with the hammer operating at two different fuel settings.
Consideration must be given to the fact that this WEAP analysis
was conducted mainly as a means of determining hammer acceptability
and driving performance of the pile.) In reviewing the predicted
capacities of the hammer and pile, the analysis of the energy
delivered and the potential for pile damage seem to have been
duly accurate. The predicted pile capacities do not appear to
be as well with those measured in the CASE method or with load

test values. However, there probably was little effort to ensure
that many of the WEAP input parameters were matched by actual field
driving conditions. The fact that the pile cushions and details of
the operation of the hammer are not documented supports this
possibility. Additionally, the hammer was reportedly operated
under conditions contrary to those recommended; this is possibly
the cause of the cracking of TP5. Since a wave equation analysis
requires more details on the pile driving system, additional care
in monitoring and directing the field operations would assist in
its 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
the 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*

<table>
<thead>
<tr>
<th>Method</th>
<th>Mean R3</th>
<th>Mean R4</th>
<th>COV R3 and R4</th>
<th>Mean R5</th>
<th>Mean R6</th>
<th>COV R5 and R6</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENR</td>
<td>0.674</td>
<td>2.021</td>
<td>0.52</td>
<td>0.091</td>
<td>0.274</td>
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<tr>
<td>ENR MOD</td>
<td>1.127</td>
<td>3.380</td>
<td>0.35</td>
<td>1.158</td>
<td>0.473</td>
<td>0.23</td>
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<td>Hiley</td>
<td>1.406</td>
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<td>0.194</td>
<td>0.291</td>
<td>0.23</td>
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<td>Gates</td>
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<td>0.20</td>
<td>0.369</td>
<td>0.553</td>
<td>0.27</td>
</tr>
<tr>
<td>Janbu</td>
<td>2.740</td>
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<td>0.64</td>
<td>0.359</td>
<td>0.808</td>
<td>0.34</td>
</tr>
<tr>
<td>FCUBE</td>
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<td>0.480</td>
<td>0.961</td>
<td>0.31</td>
</tr>
<tr>
<td>WEAP87</td>
<td>2.113</td>
<td>2.113</td>
<td>0.38</td>
<td>0.317</td>
<td>0.317</td>
<td>0.46</td>
</tr>
</tbody>
</table>

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

The PCAP program 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 drivability. 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.

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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 restring tests should be developed for assessing setup.
REFERENCES


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.