FIELD DEMONSTRATION: RESPONSE OF INSTRUMENTED PILES TO DRIVING AND LOAD TESTING

by

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INTRODUCTION

During the ASCE Soil Mechanics and Foundations Division Speciality Conference a demonstration of the "Case Method" of pile capacity prediction was proposed. In planning the demonstration, interest in other related problems caused the demonstration to grow into an ambitious field test project. The occasion of the driving of a load test pile under controlled conditions afforded an opportunity for additional measurements which could not be allowed to escape. The tests as finally constituted and the results obtained from them will be described herein.

The testing was performed at a site on the Purdue University Campus. A soils investigation had been performed in advance of pile driving. Prior to the demonstration, two load test anchor piles were driven. During the driving of the second anchor pile, continuous measurements were made to determine hammer performance over the entire range of conditions. Extensive dynamic measurements were recorded during the driving of the test pile before a group of about 400 conference attendees (1). Near the end of driving the test pile, the Case pile capacity computer with its associated transducer was demonstrated. Shortly after completion of driving, a static load test was performed and finally, on the

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following day, a pull-out test was made. An attempt to measure the residual stresses in the pile at each stage of the driving and testing operation was also undertaken.

Two quite different means to determine pile capacity from the dynamic measurements have been developed at Case Institute of Technology (2, 3). The use of these methods will be discussed and the results obtained from these approaches will be presented. The Texas Wave Equation Analysis was also performed as was a similar analysis modified for the diesel hammer. Finally, the results of all methods are compared.

LOCATION AND SITE CONDITIONS

The field demonstration site is located on the western side of the campus property of Purdue University at the northeast corner of the intersection of McCormick Road (County Road 526) and Third St. Drive. This level site is located on the edge of a large high terrace along the Wabash River. A variable depth of weathered loess covers the stratified sand and gravel layers of the terrace.

The second of the two hollow stem auger borings completed at the site is presented in Fig. 1 which shows the soil profile as well as the standard penetration test blow count versus depth. The soil samples were visually inspected and grain size distribution curves were used to assist in their classification according to the Unified Soil Classification system. The Standard Penetration Test blow count is plotted at the bottom of the 18-inch driving stroke. As usual, the blow count represents the number of blows for the last 12 of 18 inches of penetration. On the right side of Fig. 1, data is presented for D_{50}, the grain size in mm. for 50% passing, for some of the retrieved samples.

The boring log and standard penetration test data appearing in Fig. 1 is not what was presented at the time of the Conference. A second boring (1A) was necessary as the blow counts obtained in the first boring were considerable higher than blow counts of nearby borings made on the terrace area as part of previous site investigations for University structures.

At the time Boring 1 was completed (5/6/72), the ground water table was found to be at -18.5 feet. Boring 1A was completed on 7/17/72 and no ground water table was observed during drilling operations although it was noted that the hole caved to thirty feet after completion.

PILE DRIVING

Three 50-foot long steel H-piles (10HBP57) were driven at the test site. The first two piles driven to a penetration of 46 feet were anchor piles. The first anchor pile was driven within two feet of the test boring. The two anchor piles were driven 12 feet center to center. Prior to driving the instrumented test pile half way between the anchor piles, a pair of 3[4.1 beams were welded on to the web of the test pile to protect the strain gage wires from damage during driving. The test pile was driven approximately 47 feet into the ground.

A DELMAG D-12 diesel hammer was used in driving the three piles. The hammer is an open-end diesel type with a manufacturer's rated energy of 22,500 ft-pounds. The ram weight is 2,750 pounds, the anvil weight 710 pounds, and the total hammer weight was 6,050 pounds. The helmet-cushion assembly weighed a total of 1,030 pounds. A cushion located under the 4 inch thick steel 150 pound anvil plate, consisted of 3 one-half inch aluminum plates alternating with 2 one and one-half inch Conbest sheets.

DYNAMIC INSTRUMENTATION SYSTEM

Two separate instrumentation systems were used for the dynamic measurements. These two data acquisition systems are each associated with different methods of pile capacity prediction. Method one, which involves an extensive one dimensional wave analysis of the pile (CAPWAP), requires the storage of analog data for later evaluation on a large digital computer. Method two utilizes a special purpose computer in the field and displays the static pile capacity immediately following the hammer blow.

The components of the instrumentation system are:

(1) for the analysis method: Two light-weight strain transducers and two accelerometers, all of which are bolted to the pile wall, are used to measure force and acceleration at the pile top. They can be seen in Fig. 2. The strain transducer is the small diamond shaped device and the accelerometer is the small block beside it. A second set of these devices is mounted on the opposite side of the pile to average out any bending effects. A D.C. bridge conditioner and amplifier provides signal conditioning and amplification to the strain transducers. Two battery units supply power to operate

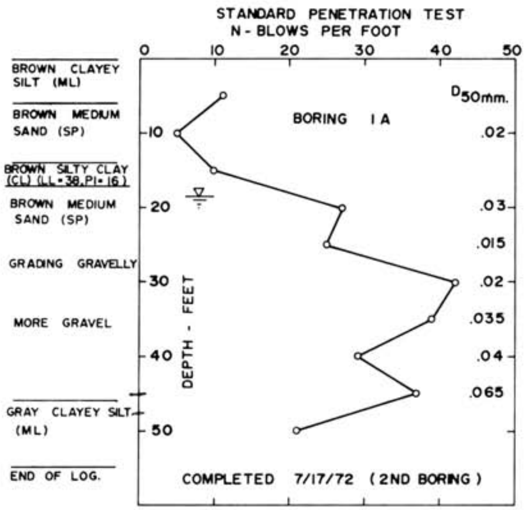


FIGURE 1 - BORING LOG AND STANDARD PENETRATION TEST DATA

FIGURE 2 - INSTALLATION OF FORCE TRANSDUCER

the accelerometers. The signals are recorded on a four channel, portable magnetic tape recorder.

(2) for the special purpose computer method: An H beam force transducer is inserted between the hammer and the pile top. This device can be seen at the top of the pile in Fig. 2. At the time that this photograph was made it was just being bolted to the pile top. An accelerometer is attached to each side of the transducer. The special purpose computer which provides signal condition, analog computation and a digital readout of pile capacity completes this system.

Additional instrumentation, not normally used with the Case Method, was used for purposes of static correlation and for the measurement of additional quantities. The primary components of this additional system were strain gages attached to the pile wall at six locations below grade. At each location a two element resistance strain gage was attached to each side of the pile and connected into a four arm bridge circuit. This installation was protected by 3-inch channel sections that also covered the lead wires. The lead wires can be seen emerging from the pile in Fig. 2. The gage locations are shown later on Fig. 12. Several channels of signal conditioning and amplifying units had to be added to the recording system to accommodate the additional dynamic strain channels. A second magnetic tape recorder was used to make possible the recording of a total of seven channels of data.

Set-rebound measurements were obtained for anchor pile 2 and the test pile. Paper was fastened to the pile and a horizontal bar was anchored to the ground. The pile displacement was then drawn on the paper by guiding a pen slowly along the bar during driving. This operation is shown in Fig. 3.

DATA ACQUISITION AND PROCESSING

Anchor Pile No. 1 was driven on June 12, 1972. The only measurement made during driving was the recording of the blow count.

Anchor Pile No. 2 was also driven on June 12. During driving a number of measurements were made to study the performance of the hammer. Force and acceleration were measured at the pile top for every blow from beginning to end of driving using the small transducers which bolt to the pile web. Set-rebound

FIGURE 3 - RECORDING OF SET - REBOUND MEASUREMENTS

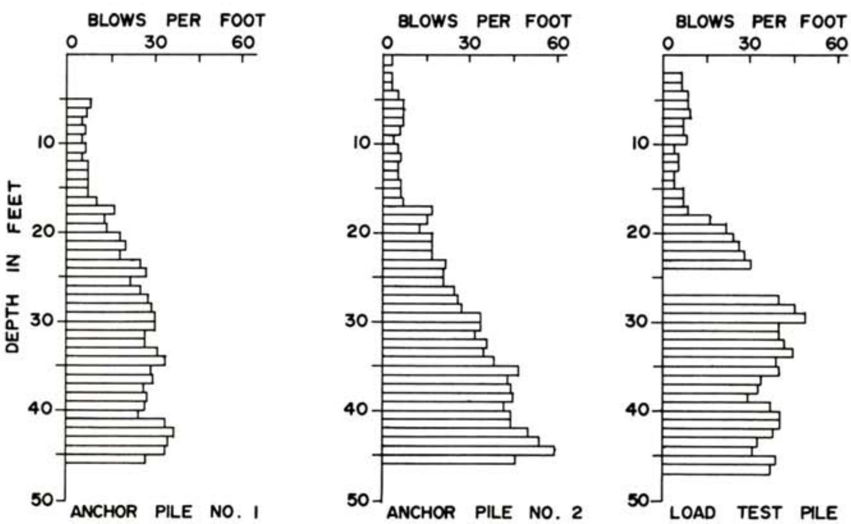


FIGURE 4 - PILE DRIVING RECORD

measurements were also made for every blow. In addition, a video-tape camera held level with the top of the hammer was used to record the ram jump height for the entire driving operation.

The test pile was driven on June 13. It was first driven to a depth of about 25 feet and the blow count was recorded. All of the instrumentation outlined above was then connected except the force transducer and the field computer. After the arrival of the conference participants the pile was driven to a depth of 47 feet 4 inches and continuous measurements were recorded on magnetic tape. Two acceleration and five strain records were recorded on the two tape recorders. In addition, set-rebound measurements were made, and the gross and net set is presented in Table 1 for the last 12 blows of driving. At

TABLE 1

GROSS & NET SETS FOR TEST PILE*

BLOW NO. (1)	GROSS SET (inches) (2)	NET SET (inches) (3)	GROSS/NET Ratio (4)
29	0.46	0.29	1.58
30	-	-	-
31	0.44	0.36	1.24
32	0.45	0.29	1.52
33	0.45	0.32	1.42
34	0.42	0.25	1.68
35	0.50	0.37	1.35
36	0.36	0.29	1.22
37	0.47	0.33	1.43
38	0.41	0.26	1.55
39	0.38	0.24	1.57
40	0.40	0.31	1.29
Average	0.43	0.30	1.43

*Table gives data for the 29th through the 40th blow of the last foot of driving.

this point driving was interrupted, the force transducer was attached to the top of the pile, and the pile capacity computer was demonstrated during the remaining two feet of driving. The driving record for all three piles is shown in Fig. 4, plotted from data presented in the appendix, Table 1A, Driving Record. In Fig. 5, the test pile driving record is compared with the Standard Penetration Test blow count. It can be seen that the

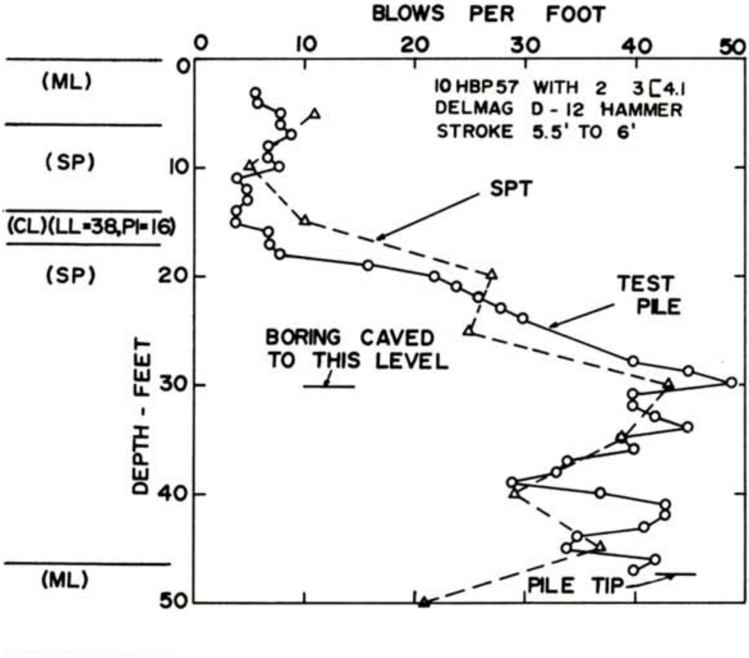


FIGURE 5 - TEST PILE DRIVING RECORD

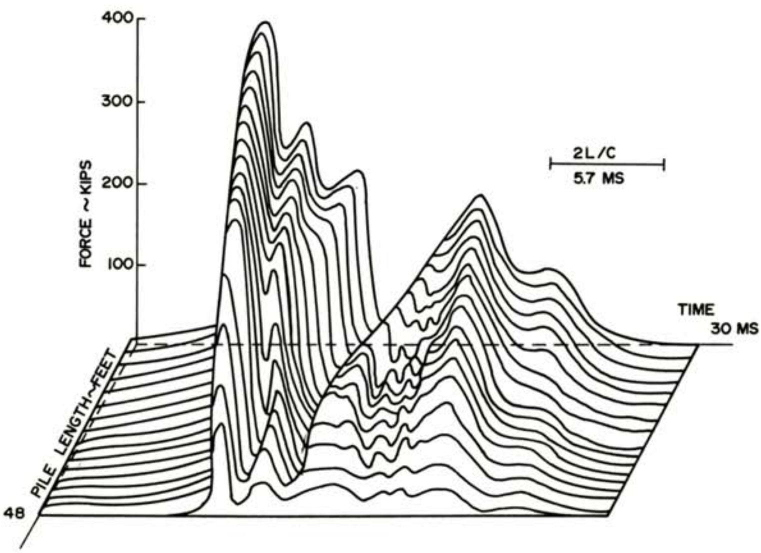


FIGURE 6 - TYPICAL PILE FORCE-TIME RECORD.

two curves follow each other reasonably well.

The pile capacity computer makes a real time computation of predicted pile strength and provides an immediate readout. The only information required to operate the version of the computer demonstrated is the pile length and weight. After these quantities are set on the control panel the computer automatically makes the computations for each blow based on only the force and acceleration signals coming from the top H beam transducer. The strength computation is based on the expression

$$R = \frac{F(t_o) + F(t_o + \frac{2L}{c})}{2} + \frac{Mc}{2L} \left(v(t_o) - v(t_o + \frac{2L}{c}) \right) \quad (1)$$

where R = static capacity, F is the measured force at the pile top, t_o is the time of zero velocity of the pile top, L is the pile length, c is the wave speed in the pile, M is the pile mass and v is the velocity at the pile top. The average value of the computer capacity predictions was 175 kips.

A magnetic tape recorder stores an electronic image of the event measured, and has the capability to recreate it at a later time. This makes possible a fully automatic data processing system. In the laboratory at Case Institute of Technology the analog tape recorder can be converted to digital form, some simple computations can be made on the digital data during the conversion operation, the data can be stored on digital magnetic tape for later conversion and a plotted record can be obtained on a computer controlled plotter. All of the data taken during driving of anchor pile 2 and the test pile was processed in this fashion.

The video tape record of ram jump was replayed and the jump height determined for each blow. This value was then associated with the appropriate digital record of force and acceleration.

LOAD TRANSMISSION DURING DRIVING

Dynamic records of strains in the pile, as reproduced from the tape recorder on an oscillograph were plotted as a function of both time and pile length. These measurements were available at the pile top and at three locations along the pile. Strain-time relations for other locations were obtained by interpolation producing a surface of pile force as a function of pile length and time. This three-dimensional plot is shown in Fig. 6. A wedge-like compressive force can be seen which results from an impact wave travelling at wave speed, c, to the pile toe and causing there a tensile reflection wave (because of relatively small end bearing forces) which travels toward the pile top. The tensile reflection cancels the downwards propagating hammer compression forces almost completely, thus creating the valley of near zero forces. Therefore, it can be observed that the pile top force decreases at a time approximately 2L/c after impact, i.e., after twice the time that the stress wave requires to travel along the pile length, L. This decrease gives an indication at the pile top of the character of the pile tip resistance.

Another interesting observation can be made from Fig. 6. Note, that the toe force (actually, this record was taken 1.5 feet above the pile tip) first reaches a high value and then declines and oscillates about some constant value (about 30 kips). The high peak is caused by both the inertia of the 1.5 foot pile section below the gages and damping due to the high velocity at this time. The average of the later value is due to the static resistance below that point. In fact, as will be shown later, this value corresponds to the force measured at the same location during the static load test.

It must be emphasized at this point that, in general, soil resistance values cannot be extracted this easily from dynamic force records for locations other than the pile toe. Intermediate pile locations exhibit force effects caused by hammer impact, soil resistance, and pile inertia. Only a detailed dynamic analysis can separate these effects.

HAMMER PERFORMANCE

The Case Methods have sought to replace dynamic formula and, therefore, make hammer energy a much less important parameter in describing hammer performance. However, a considerable interest continues in hammer energy. Since the necessary measurements are available for computation of energy delivered by the driving system to the pile, it is useful to calculate it. Of particular interest is this aspect of hammer performance for an open end diesel hammer since widely differing views are held of its operating characteristics.

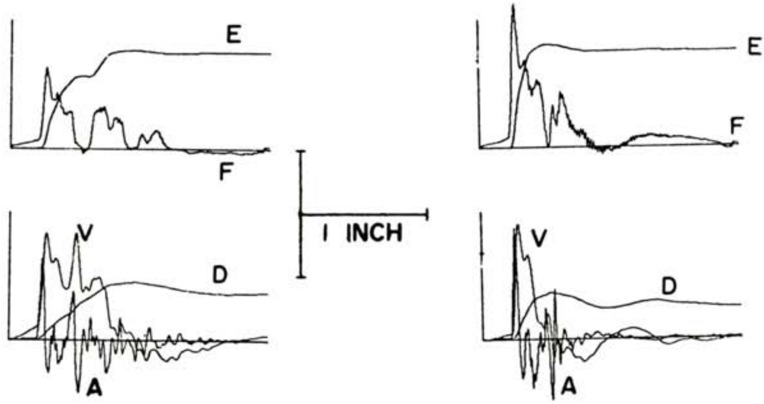
During the driving of Anchor Pile No. 2, ram stroke, force, acceleration and displacement were recorded simultaneously as described above. The video tape records were processed manually to obtain the ram stroke for each blow. It varied from about four feet at the beginning of driving to nearly six feet at the end. The automatic data processing system was used to calculate delivered energy. The processing system operated as follows: The measured analog strain record was converted to digital form and then to force. The two acceleration records were digitized at the same time using a multiplexing system and averaged to obtain a single signal. The digital acceleration and force records for each blow were stored on digital magnetic tape for later processing. Calculation of energy was accomplished by integrating the averaged acceleration once to obtain velocity and a second time to obtain displacement. The final displacement (after 50 milliseconds) was automatically checked with the measured set and if a difference was observed the acceleration zero was adjusted to correct the final displacement. Also, the first velocity peak was checked for proportionality with the first force peak. If this proportionality was seriously violated the results from that blow were ignored. It should be noted that the peak accelerations measured at impact were extremely high; in some cases over 500 g's, where g is the acceleration of gravity.

Energy was calculated from the expression

$$E(t') = \int_{0}^{t'} F(t)\ v(t)\ dt \qquad (2)$$

where E is energy at the pile top expressed as a function of time. Zero time is taken as some time before the beginning of the blow. Thus, an energy-time record can be obtained which can be automatically plotted. Typical plots of such relations are shown in Fig. 7, together with force, velocity, and displacement curves. To demonstrate differences caused by resistance changes, plots of both an early and a late blow are shown. Important to note is the fact that in the early blow the maximum value of energy transfer is reached at a later time after impact and no energy is transferred back to the hammer (the curve stays horizontal). In the later blow a peak value is reached after which the energy curve decreases indicating that some energy is returned to the hammer.

In previous work concerning hammer energy transfer (4), the maximum value was always considered because of its importance regarding the pile driving ability. Here it is intended, however, to consider that value of energy that was actually transferred from the ram to the pile during the blow. Obviously, the total energy transferred is the final value, i.e., a value at a time when the energy curve stays constant. For constant fuel injection with the same combustion efficiency and with the same hammer losses, this value should be constant, or, in case not all of these restrictions were valid, it should be at least independent of the ram stroke.



EARLY BLOW LATE BLOW

Scales

Horizontal:
Time 1 in = 25 msec

Vertical:
A - acceleration 1 in = 400 g
V - velocity 1 in = 10 ft/sec
D - displacement 1 in = 2 in
E - energy 1 in = 10 k-1
F - force 1 in = 400 k

FIGURE 7 - SAMPLE PLOTS OF AUTOMATICALLY PROCESSED DATA.

The results of the measurements on Anchor Pile No. 2 confirm this idea. Consider Fig. 8 where the values of jump height are plotted vs. the measured final energy value. Although there is considerable scatter in these values there is certainly no correlation between stroke and energy. In fact, it seems that there is an average value of about 7.5 kip-ft around which individual values scatter independent of the stroke level.

Also shown in this graph is the line that is frequently assumed for hammer energy computations, i.e., ram weight times stroke. The measured values are all on the lower energy side of this line, a result that is expected since only actually delivered energy is considered.

A brief statistical investigation produced a mean energy value of 7.45 kip-ft and a linear correlation coefficient between stroke and energy of 0.146 which means that there is only a 3 percent chance that the two variables are related. The scatter of the energy values about their mean as expressed by the coefficient of variation was 22 percent.

PILE LOAD TEST

A constant rate of penetration static load test was performed on the center test pile at the rate of 0.02 inches per minute. A photograph of the load test set-up is presented in Fig. 9. The test pile was loaded by a 200 ton capacity jack located between the test pile and the horizontal reaction frame. Four inch diameter rods, welded to the webbs of the two anchor piles, passing through a pair of wide flange beams provided the anchoring system.

Actually the loading was not completely continuous as it was applied by manual jacking. The jacking rate was maintained by visual observation of the two displacement dial gauges set perpendicular from the reaction frame as shown in Fig. 9. Jack pressure and pile top displacement were recorded each minute. The pile top displacement was averaged for the two displacement readings taken.

Strain was also measured at the six resistance strain gage points along the pile. However, gage number one did not function properly and results from it are not included. Difficulties were also encountered with gage number 5. It exhibited continual drift when reading, a performance that can probably be attributed to an inadequate resistance to ground. A regular procedure

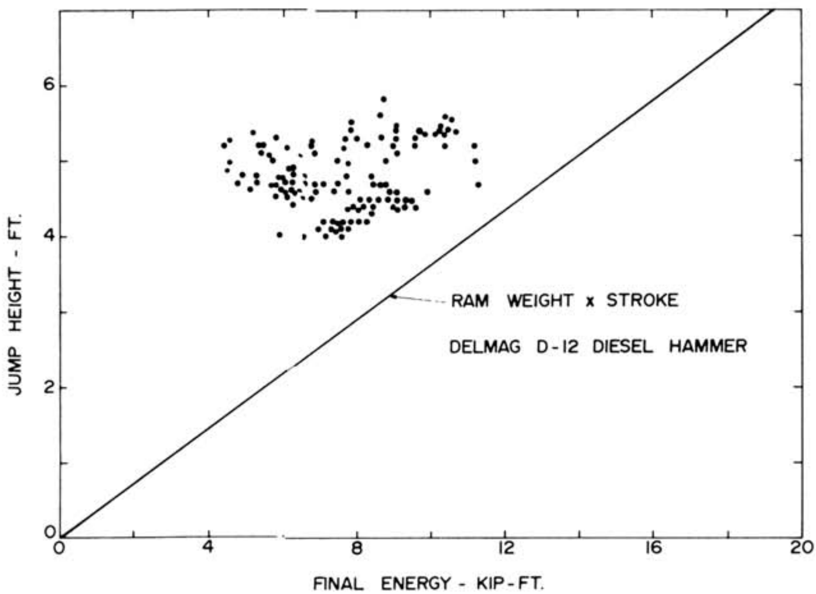


FIGURE 8 - RAM STROKE vs. FINAL DELIVERED ENERGY.

FIGURE 9 - LOAD TEST SET UP

was used in reading this gage and it produced satisfactory results for short term readings but inadequate results for the long term quantities. The short term measurements are included here while the long term values are dropped. Strain measurement was made using a Binary Electronics self balancing strain indicator, thus, the strains could be read very quickly causing only a very short interruption in the loading operation. The strains were converted into force and are summarized in Table 2A in the appendix. Force is presented for various gage numbers for given displacement of the pile top. The forces tabulated during the load test represent forces measured after pile driving and do not include any residual forces or stresses after the driving of the pile.

Residual stresses were measured along the pile length at 5 times during the two day load tests. Prior to pile driving zero readings of the strain gages were made. Shortly after driving and just prior to the load test, the residual readings were taken. Shortly after the load test, residual readings were again taken on June 13, 1972. The following day, June 14, 1972, readings were again taken prior to the pullout test. Residual readings were also obtained after the pullout test was completed. This information is summarized in Table 3A in the appendix and is plotted in Fig. 10. In Fig. 10, curves A through D represent the four sets of residual load readings as discussed above. After driving, the residual stresses are given by curve A. It can be seen that the stresses left in the pile after the load test, curve B, are higher than prior to driving. Between the time that the residual stresses were measured after driving, curve B, and just prior to the pullout test the following morning, curve C, an elapsed time of approximately 16 hours, the residual stresses decreased to approximately the initial values prior to load testing. Readings at gage location 1 are questionable. Residual stresses prior to the pullout test in the upper 30 feet of the pile relaxed even further. Finally, after pullout, the residual stresses are given by curve D where a 16 kip tensile force is noted at gage location 6.

TEST RESULTS

Load test data summarized in Table 4A, appendix, is plotted in Fig. 11, Applied Load vs. Pile Top Penetration. For clarity, not all the points given in Table 4A were plotted. It can be seen in Fig. 11 that a linear relationship existed between applied load and penetration up to approximately 140 kips. After the pile yielded, the load test was stopped at approximately 0.8 inches penetration. After unloading the elastic rebound was

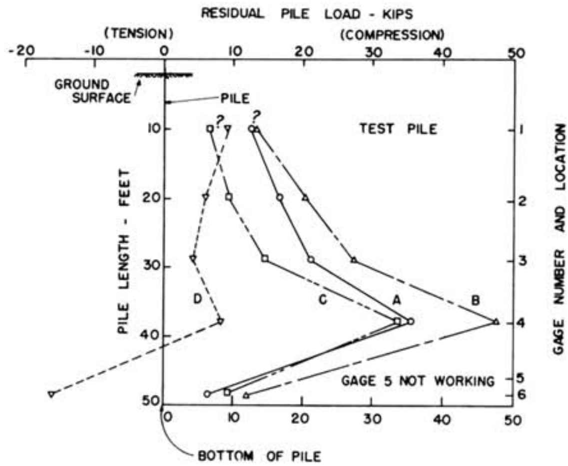


FIGURE 10 - RESIDUAL PILE LOAD vs. LOCATION.

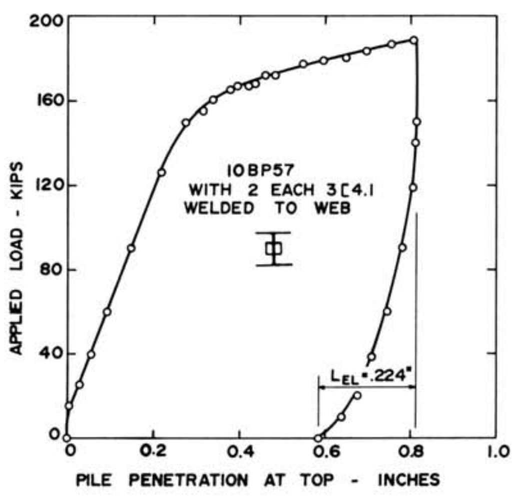


FIGURE 11 - APPLIED LOAD vs. PENETRATION

found to be approximately 0.22 inches. The H-pile and channel beam configuration is also shown in Fig. 11.

Data from Tables 2A and 4A, is presented in Fig. 12 which shows the relationships between force in the pile at various locations versus pile top displacement. In addition to the upper load test curve which has been replotted to a different scale from Figure 11, the force versus penetration relationship is shown for gages 2 through 6, for the loading sequence only. It can be seen at locations 2 and 3, the upper portion of the pile received considerable force in contrast to locations 5 and 6 which received less than a quarter of the load applied to the pile top.

By cross plotting the data presented in Fig. 12, relationships between pile force distribution at any given pile top displacement can be made. Such relationships are presented in Fig. 13 for seven different pile top displacements. Gage locations are shown on the right side of the graph. A linear relationship was assumed between the top of the pile and gage 2 as gage number 1 no longer functioned. Also, it should be mentioned that residual stresses after driving, are not included in Fig. 13.

However, if residual stresses are included in the force distribution curve, Fig. 14 results. When residual stresses are included, the force distribution appears to be almost linear with depth, at least for the top 80% of the pile. Also shown in Fig. 14 is curve B which has been replotted from Fig. 10, the residual stress after load test. Approximately 16 hours later the forces in the pile reverted to those shown by curve C, which is again replotted from the Fig. 10.

PULLOUT TEST

A pullout load test was also applied to the pile on the following day of the load test. The test setup was quite similar to the compression test except that the loading jack was removed from the pile top as shown in Fig. 9 and placed on top of the loading beams. Heavy straps were bolted to the pile top and attached to either end of a short beam passing over the top of the jack; thus when the jack was extended it placed the straps and the test pile in tension. The testing procedure and instrumentation system were the same as those for the previous load test. In the pullout test, jack pressure, pile top deflection and strain were read during the test. The jack pressure was converted to force and all of the pullout test data is presented in Table 5A, appendix.

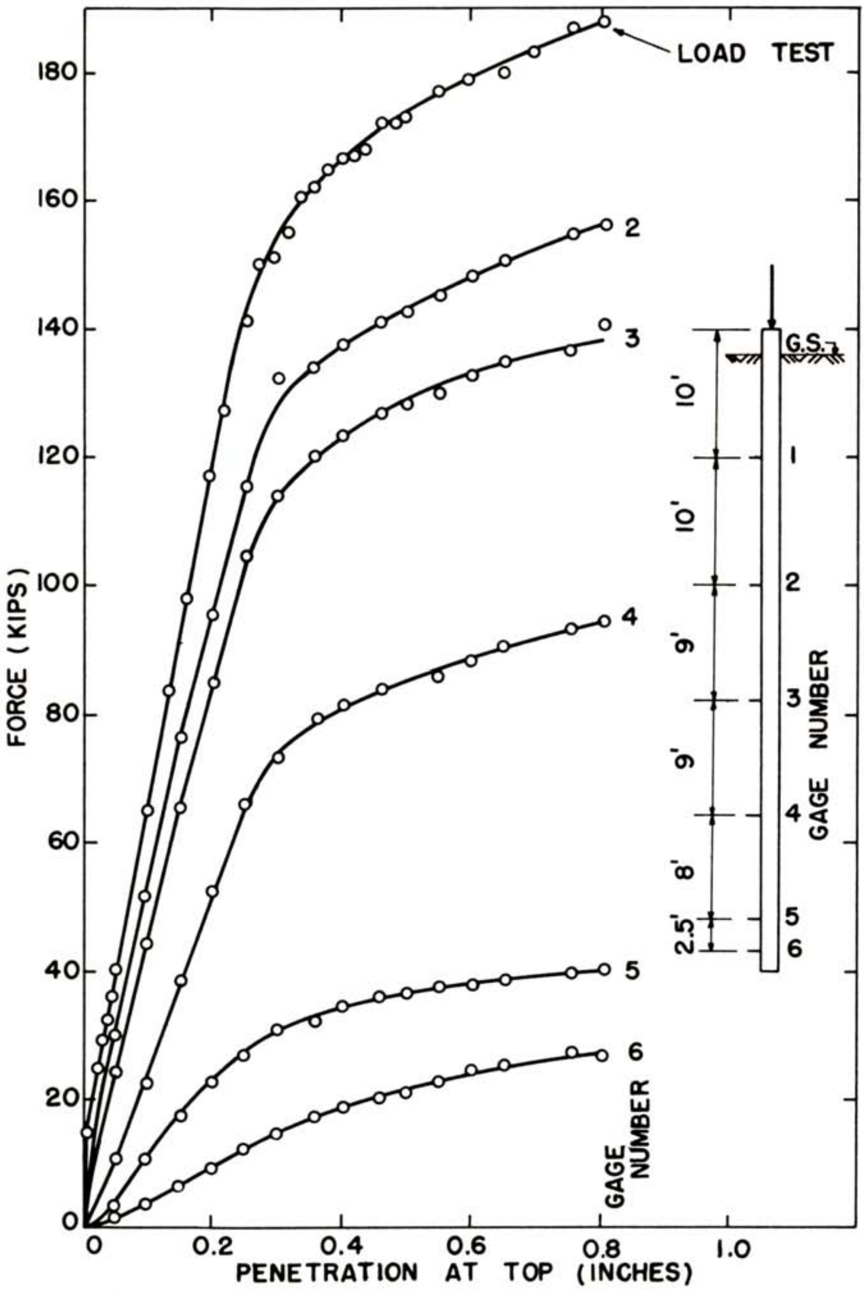


FIGURE 12 - FORCE vs. PENETRATION.

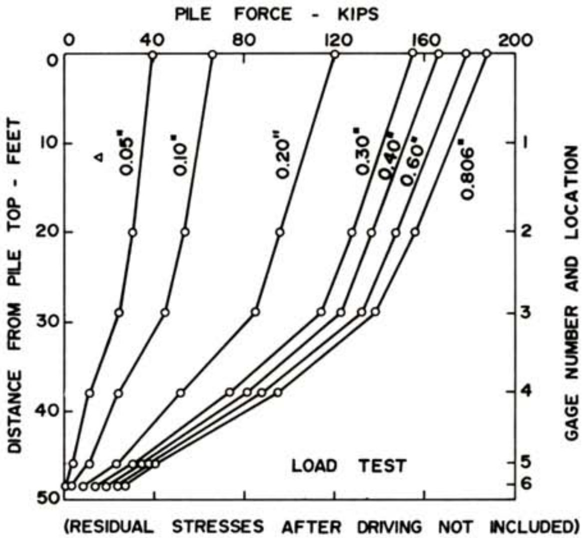


FIGURE 13 - PILE FORCE DISTRIBUTION vs. DEPTH

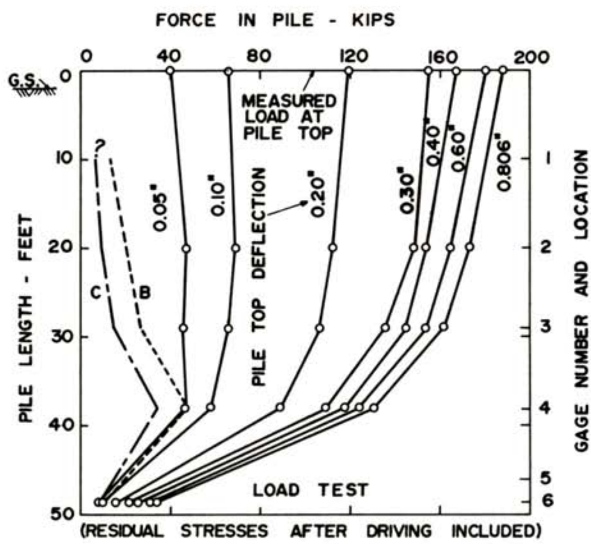


FIGURE 14 - PILE FORCE DISTRIBUTION vs. DEPTH

The data in Table 5A has been plotted in Fig. 15, Force versus Pile Top Displacement, for the top of the pile as well as for five locations along the pile. Cross plotting Fig. 15 leads to the force distribution curves shown in Fig. 16, which includes the residual stresses prior to the pullout test. Curve C, which is reproduced from Fig. 10, represents the residual stresses prior to pullout. Curve D represents the residual stresses after the pullout test was completed. The residual compressive stresses reported after the pullout test represent approximately 15 microinches of strain and probably are due to some zero drift.

PREDICTED CAPACITY

The capacity of the test pile was established by several methods including the actual load test measurement. These methods include the Case Pile Capacity (field) Computer, the Case Pile Wave Analysis Program (CAPWAP), the standard Texas wave equation method, and finally the Texas wave equation method modified for the diesel hammer. The methods and their results are described below. Predicted pile capacity was not computed using other static or dynamic pile driving formulas.

PREDICTION OF STATIC CAPACITY USING "CASE" METHODS

The same simple computation that is performed in the field using the special purpose computer can be performed automatically during the analog-digital conversion operation. The special purpose computer, demonstrated in the field, utilized the Phase 2A method given by Equation (1). A number of other similar methods have been developed during the Case project. They provide refinements in the prediction for various driving conditions. Since this was the first test for H-piles in granular material, it is valuable as it provides an insight into the performance of various predictive methods under an unfamilar set of conditions.

The primary difference between the various similar methods is the time t_o at which the computation begins. For pipe piles in cohesionless soils, experience has shown that t_o should be taken at the time of maximum velocity, (P4). The Phase 2A method described above has been effective for pipe piles in cohesive soils. Another approach (PL2c) has been to take t_o at a time 2L/c after maximum velocity.

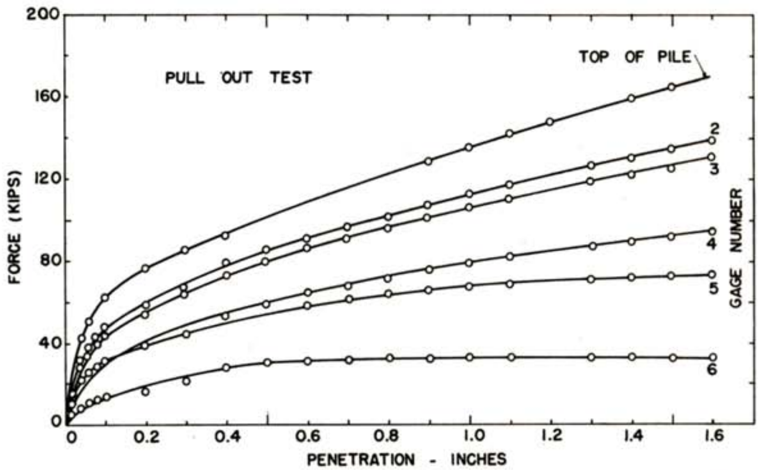
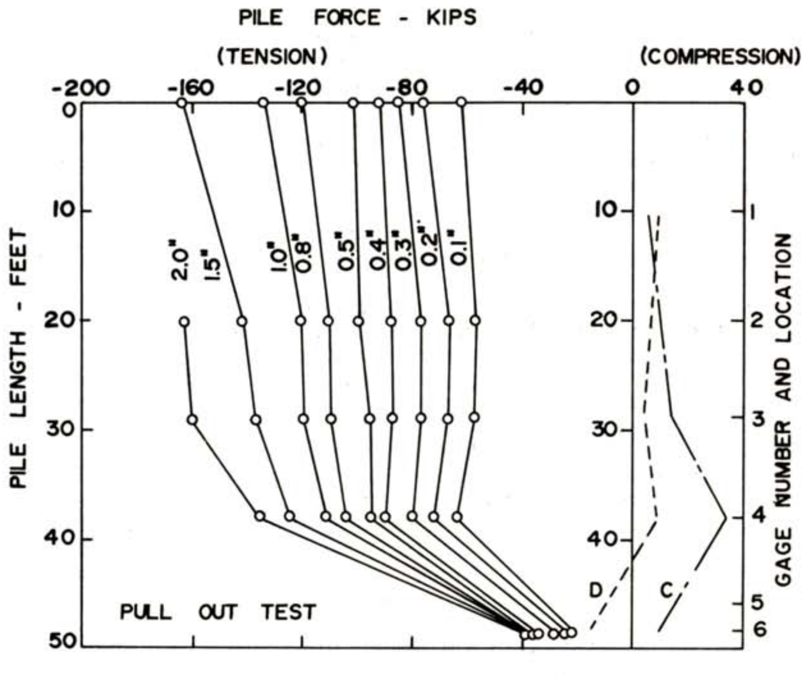


FIGURE 15 - PILE FORCE (TENSION) vs. DISPLACEMENT.



(RESIDUAL STRESSES AFTER LOAD TEST INCLUDED)

FIGURE 16 - PILE FORCE DISTRIBUTION vs. DEPTH.

For the measurements reported herein, the average prediction from Phase 2A and PL2c were about the same. However, the scatter of the Phase 2A results was somewhat greater. In fact all predictions exhibited an unusually large scatter. The results from the P4 and PL2c methods for the last 120 blows of the test pile are shown in Fig. 17, Predicted Pile Capacity vs. Blow Number. The P4 method gives results which are too high and probably is only appropriate when the pile is a Displacement type. On the average, the PL2c method agrees very well with the load test results.

Averages from the PL2c method were computed for three groups of twenty consecutive blows. These averages are 152, 164, and 153 kips. No trend can be seen for this limited number of blows. The average for the last 120 blows was 159 kips. Note that some decrease in pile capacity can be observed towards the end of driving.

PREDICTIONS FROM CAPWAP

A few of the blows recorded were also analyzed utilizing the Case Pile Wave Analysis Program (CAPWAP) that predicts among other quantities, the static soil resistance distribution and the static load test curve. This technique, reported by Rausche et al (5), uses the measured velocity as an input and determines the resistance distribution (both static and dynamic) so that the calculated force curve matches the measured force. The necessary computations are carried out on a large digital computer and the determination of the resistance is completely automatic.

A typical result was obtained for blow No. 120 taken in the last series of blows. This blow was one of the last blows recorded. The measured (M) force and velocity (multiplied by EA/c, i.e., Young's Modulus times cross sectional area divided by the wave speed) records are shown together with the analytically computed (C) curve in Fig. 18. The agreement of the measured and computed force curves shows some deficiency at about time 2L/c after impact but describes the overall behavior very well. The differences that arise can be attributed to the inadequacy of the soil model. A constant force of 16.2 kips was subtracted from the total record. This force is present before impact because of combusion chamber compression.

The force distribution curve predicted by CAPWAP is presented in Fig. 19a by the solid line. The measured values are shown by circles. In order to provide a common basis for distribution comparisons, the measured values were taken at the 155 kip level of the load test; i.e., when the pile began yielding. In general, the predicted

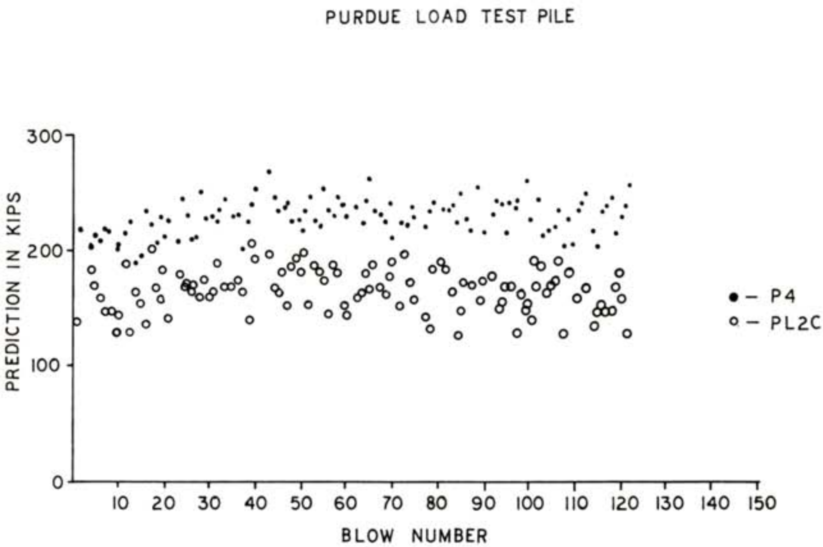


FIGURE 17 - "CASE METHOD" STRENGTH PREDICTIONS

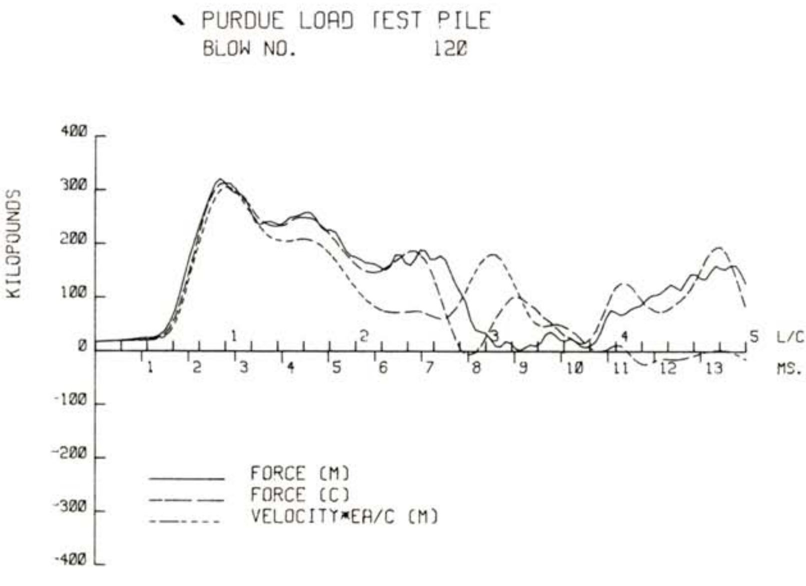


FIGURE 18 - TYPICAL FORCE, VELOCITY MATCHING CURVE.

force distribution curve agrees well with the measured values and there was relatively little change from earlier to later blows. Fig. 19b shows both the predicted and the measured load test curve. The predicted curve was obtained automatically by performing a static analysis using the resistance distribution obtained in the CAPWAP analysis.

WAVE EQUATION ANALYSIS

A standard Texas wave equation analysis was used to predict the pile capacity. In addition, the pile capacity was also predicted by a modified Texas wave equation incorporating diesel hammer characteristics. The results of these two analyses are shown on Fig. 20. In Fig. 20, the ultimate dynamic resistance, R_u is shown versus hammer blows per inch. The results of the Diesel - I program are presented by the solid line while results of the Standard wave equation analysis is presented by the dashed line. In both cases the assumed distribution of soil resistance was 20% of R_u at the tip and 80% of R_u along the side of the pile. Additional information on the assumed conditions for the two wave equation analyses is presented in Table 6A in the appendix. The Standard wave equation analysis predicts an ultimate capacity of 184 kips while the wave equation modified for the diesel hammer predicts an ultimate capacity of 166 kips.

DISCUSSION OF RESULTS

The summary of the results of all the dynamic prediction methods is given in Table 2 for comparison with the static load test value. When such comparisons are attempted, the inadequacy of static load test evaluation procedures becomes obvious. The ultimate capacity of the load test, shown in Fig. 11, has been selected as 160 kips. This value represents the authors' interpretation. It is essential that some definite procedure be accepted for defining failure.

All of the dynamic methods show good agreement with the static load test. The closest agreement with the measured load test, comes from the Case method from processed Analog Record. However, all methods show very good agreement.

Some comments on the relative advantages of these methods are appropriate. The wave equation solution has the advantage of not requiring field measurements. It can also be used to study the selection of driving equipment. However, it requires the use of a large digital computer and the assumptions of hammer performance and soil characteristics. The Case method field computer does require

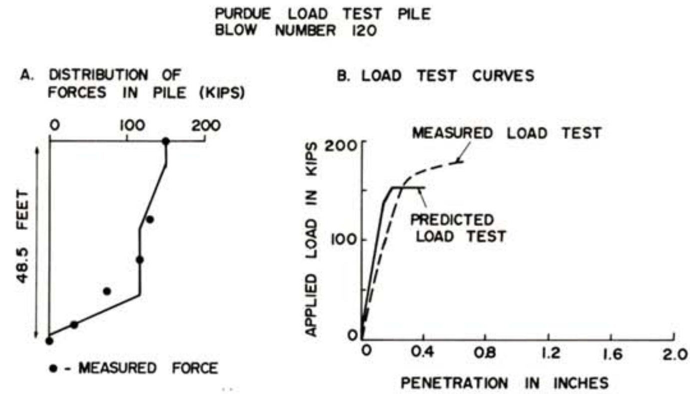


FIGURE 19 - CAPWAP RESISTANCE DISTRIBUTION AND STATIC LOAD TEST

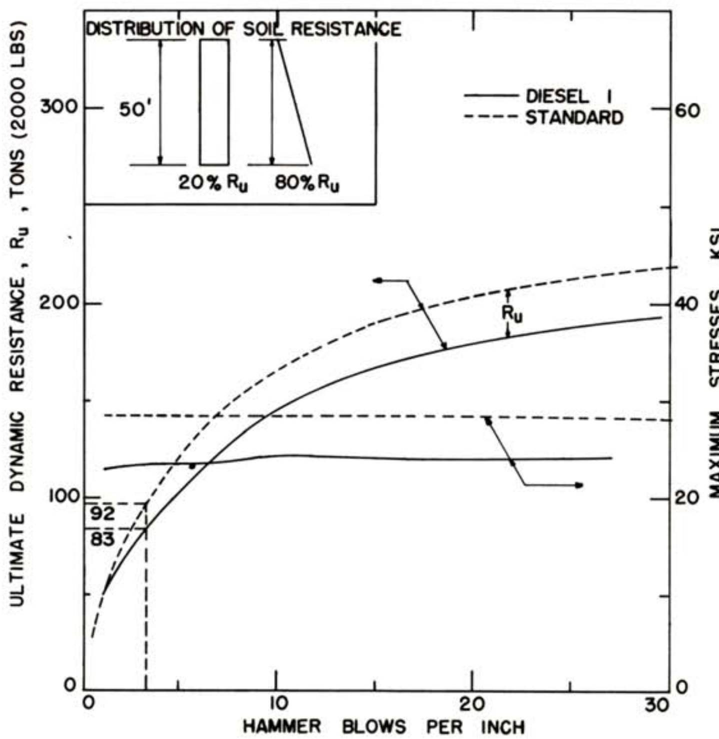


FIGURE 20 - WAVE EQUATION ANALYSIS RESULTS

field measurements but does not require any assumptions of either hammer performance or soil characteristics. It gives an immediate answer in the field. The CAPWAP method

TABLE 2

PILE CAPACITY COMPARISON

TEST METHOD (1)	PILE CAPACITY (Kips) (2)
Load Test	160*
Case Methods:	
Field Analog Computer	175
Processed Analog Record	155
CAPWAP (Resistance Distribution)	159
Texas Wave Equation (Standard)	184
Texas Wave Equation (Modified for Diesel Hammer)	166

*Possible interpretation from Fig. 4.

also requires field measurements and a large digital computer for processing. It does not provide an immediate answer. However, without making any assumptions on hammer performance or soil characteristics, the soil resistance and its distribution along the pile length is determined automatically. In view of the variety in hammer performance which has been observed in the field, it seems to be desirable to avoid assumptions on hammer performance. This test represented the first opportunity to try the Case method on an H pile in granular soil. It appears that the PL2c method is the best method under these field conditions.

The residual stress measurements presented here were quite challenging; particularly since they were made as part of the demonstration to a large group of people. Some difficulty was encountered in making these measurements due to the stiffness of the pile. A smaller cross section of pile would probably have provided a larger strain output.

An examination of the driving record raises an interesting question. Does the proximity of the drill hole effect the driving resistance? It can be observed that as the distance from the pile to the drill hole increases, the driving resistance also increases. Frequently, test piles are driven into or very near old drill holes. Based on these data this practice should be avoided. (Additional data from the entire testing program may be obtained from the authors.)

CONCLUSIONS

The following conclusions are supported by this test program:

1. All dynamic methods tested provided good results for this H pile in granular material.
2. The measurements for the Case methods are of such a character that they can be made routinely in the field.
3. Force and acceleration measurements can be made routinely either for immediate or later processing.
4. The PL2c method appears to be the best method for predicting the ultimate capacity of H piles in granular soils.
5. Final hammer energy is fairly independent of ram stroke for an open end diesel hammer.

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Piling - The American Iron and Steel Institute Piling Committee

All driving equipment and crew - The Foundation Equipment Corporation, Newcomerstown, Ohio

Load test equipment - The A. Bentley and Sons Co., Toledo, Ohio.

Wave Equation Analysis - David M. Rempe, Doctoral candidate, The University of Illinois, Urbana, Illinois.

APPENDIX 1. - TABLES

TABLE 1A

DRIVING RECORD

Depth (Ft.) (1)	Anchor Pile 1 (B/F) (2)	Test Pile (B/F) (3)	Anchor Pile 2 (B/F) (4)	N(SPT) (B/F) (5)
1	-	-	3	
2	-	-	-	
3	-	6	3	
4	-	6	3	
5	-	8	5	11
6	8	8	7	
7	7	9	7	
8	5	7	7	
9	6	7	6	
10	5	8	4	5
11	6	4	5	
12	5	5	6	
13	7	5	5	
14	7	4	5	
15	7	4	6	15
16	7	7	6	
17	10	7	7	
18	16	8	17	
19	13	16	16	
20	14	22	13	27
21	18	24	17	
22	20	26	17	
23	18	28	17	
24	25	30	22	
25	27	-	21	25
26	22	-	21	
27	25	-	25	
28	28	40	26	
29	29	45	27	
30	30	49	34	42
31	30	40	34	
32	27	40	32	
33	27	42	36	
34	31	45	35	
35	34	39	38	39
36	29	40	47	
37	30	34	43	
38	27	33	44	
39	28	29	45	
40	27	37	42	29
41	25	43	44	
42	34	43	44	
43	37	41	50	
44	35	35	54	
45	34	34	59	37

TABLE 1A (cont'd)

DRIVING RECORD

Depth (Ft.) (1)	Anchor Pile 1 (B/F) (2)	Test Pile (B/F) (3)	Anchor Pile 2 (B/F) (4)	N(SPT) (B/F) (5)
46	27	42	46*	
47		40		
48				
49				
50				21

*38/10"

TABLE 2A

FORCE DISTRIBUTION

Set (in.) (1)	FORCE (Kips)* GAGE NUMBER No. 1** (2)	No.2 (3)	No. 3 (4)	No. 4 (5)	No. 5 (6)	No. 6 (7)
0.052		30.2	24.1	10.5	3.15	1.47
0.102		51.9	44.2	22.3	10.27	3.36
0.15		76.5	65.1	38.2	17.4	6.29
0.20		95.5	85.0	52.7	23.1	9.24
0.25		115.4	104.9	66.0	27.1	12.0
0.30		131.7	113.8	73.2	30.9	14.7
0.358		133.8	120.0	79.4	32.2	17.3
0.40		137.0	123.5	81.7	34.3	18.9
0.46		140.9	127.0	84.0	36.3	20.3
0.50		142.3	127.8	84.0	36.3	20.8
0.55		145.0	129.9	86.0	37.7	22.6
0.60		147.8	132.7	88.3	37.7	24.4
0.650		150.5	134.8	90.4	38.4	25.6
0.755		154.9	136.9	93.0	39.5	27.7
0.806		156.1	140.5	94.4	40.7	26.7

*Without residual forces after driving.
**Gage 1 not functioning.

TABLE 3A

MEASURED RESIDUAL FORCES

TIME	FORCE (Kips)					
	GAGE NUMBER					
	1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Before Driving	0	0	0	0	0	0
A	12.9	16.8	21.1	35.6	-	6.45
B	13.3	20.2	27.3	47.9	-	12.0
C	6.87	9.45	14.6	33.9	-	9.22
D	9.1	6.2	4.3	8.4	-	-15.9

+ = compression - = tension

TIME A = After driving, 6/13/72, approximately 3 p.m.

TIME B = After Load Test, 6/13/72, approximately 5:30 p.m.

TIME C = Prior to pull out test, 6/14/72, approximately 9 a.m.

TIME D = After Pull out test, 6/14/72, approximately 11 a.m.

TABLE 4A

LOAD TEST DATA

TIME (min.) (1)	LOAD (Kips) (2)	AVG. PENETRATION (inches) (3)
0	0	0
1	15	.006
2	-	.020
3	25	.026
4	29	.0325
5	32.5	.0395
6	36	.046
7	40	.052
8	43.5	.060
9	46.5	.065
10	50.5	.0725
11	56	.082
12	60	.092
13	65	.102
14	70	.112
15	76	.1235
16	83.5	.135
17	90	.1465
18	98	.1595
19	110	.177
20	117	.1955
21	127	.2145
22	-	.2325
23	141	.251
24	150	.2725
25	151	.2935
26	155	.3155
27	160.5	.3365
28	162	.358
29	165	.379
30	167	.3995
31	167	.4215
32	168	.439
33	172	.460
34	172	.4805
35	173	.499
36	177	.549
37	179	.5975
38	180	.650
39	183	.6935
40	187	.755
41	188	.806

TABLE 4A (CONT'D)

LOAD TEST DATA

TIME (min.) (1)	LOAD (Kips) (2)	AVG. PENETRATION (inches) (3)
	UNLOADING	
42	188	.815
43	150	.8165
44	140	.8145
45	129	.810
46	119	.806
47	109	.797
48	99	.788
49	90	.781
50	80	.767
51	70	.7575
52	60	.7445
53	50	.726
54	39	.708
55	30	.694
56	20	.675
57	10	.640
58	0	.5875
58-1	0	.582

L_{el} = .806" - .582" = .224"

TABLE 5A

PULL OUT TEST DATA

TIME (min.)	DISPLACEMENT (inches)	FORCE (KIPS) TENSION					
		GAGE NUMBER					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	0.020		21.4	17.2	9.0	14.7	4.2
	0.040		31.9	27.7	17.6	21.4	8.4
	0.060		38.2	34.2	23.5	25.4	10.9
	0.080		43.9	39.5	27.9	28.8	11.9
5	0.100		48.0	43.5	31.3	31.4	13.4
10	0.200		58.1	53.2	39.4	38.1	16.2
15	0.300		67.6	62.8	46.4	43.9	20.6
20	0.400		79.1	73.6	56.7	53.1	27.9
25	0.500		86.5	80.6	61.5	58.8	30.4
30	0.600		91.0	85.5	63.6	58.8	30.4
35	0.700		96.5	90.4	67.6	61.0	31.3
40	0.800		101.7	95.7	71.4	63.6	31.9
45	0.900		107.3	100.8	75.1	65.5	32.4
50	1.00		112.0	105.5	78.4	66.8	32.8
55	1.10		117.5	111.1	82.0	67.7	32.6
65	1.30		124.0	116.1	85.6	70.3	32.8
70	1.40		127.9	120.3	88.1	71.4	32.1
72	1.50		133.5	123.9	91.0	71.4	32.2
74	1.60		138.2	130.0	93.8	72.8	32.7
76	1.70		142.0	133.7	95.9	73.3	32.7
78	1.80		147.4	137.0	99.1	74.4	32.5
80	1.90		150.5	142.0	100.7	75.5	32.5
82	2.00		154.8	146.5	103.0	76.0	31.9

TABLE 6A

TEXAS WAVE EQUATION ANALYSIS ASSUMPTIONS

Pile: 10 HPB 57; 8.2 lbs. added.

Pile Cross Section Area: 19.2 square inches

Pile Length: 50 Feet

Hammer: DELMAG D-12

Hammer Rated Energy: 22,000 ft-lbs.

Hammer Efficiency: $e_f = 1.0$(1)

Hammer Explosive Force: 93,000 lbs(1)

Helmet (Drive Head): 900 lbs.

Efficiency: 0.85, Diesel 1; 0.60, Standard.

Anvil: 966 lbs.

Efficiency: 0.85, Diesel 1; 0.60, Standard.

Cushion (Hammer): Conbest, $k = 20 \times 10^6$ lb./in.

Efficiency: 0.80

$Q_{side} = 0.10$, $Q_{point} = 0.10$

$J_{side} = 0.05$, $J_{point} = 0.15$

C = 1.08 ft. (For Diesel 1). Equivalent Stroke = 4.42 Ft. (5.5 ft. actual stroke).

(1) Not applicable to DIESEL 1 simulation.

APPENDIX 2 - REFERENCES

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