Bearing capacity of piles in soils with time dependent characteristics

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Two case histories are presented on the evaluation of static bearing capacity of piles driven in soils that exhibit time dependent strength changes. Under consideration are a steel and a concrete pile driven into soils generally consisting of silty and clayey fine sand. Pile dynamic measurements and analyses were performed on both piles during initial installation, and also during restrike one week later. This information was used to predict each pile's long term capacity (defined as 14 days after initial drive), which was then compared to that measured with a static load test. Agreement between predicted and measured capacities were remarkably good, with an average error of only 4.5 percent. Two weeks after driving, total pile capacities were three times those at the end of drive. If only the skin friction was considered, however, it increased by an average factor of 16. The increase in pile capacity was determined to be linear over the time period considered.

INTRODUCTION

The planned expansion of a major transportation facility in Central Florida, USA included the construction of a building which encompassed over 3.5 hectares and ranged from four to eleven stories. Anticipated column loads ranged up to 22,000 kN. The highly variable subsurface conditions encountered at the site included strata of very loose clayey sand and very soft clay. A foundation system consisting of driven displacement piles with service loads of 670 kN and 1330 kN was recommended for support of this structure.

A test pile program was recommended prior to final design of the foundations for the structure to verify pile service loads and more accurately predict pile driven lengths. The program incorporated both dynamic and static testing of steel pipe and precast concrete piles. The aim of the dynamic testing was to evaluate pile capacity, efficiency of pile driving systems, and integrity of in-place piles and splices. Static loading tests were performed to verify capacity.

An additional aim of the test program was to quantify the rate of pile set up after initial driving. Local experience indicated that pile capacity increased substantially with time after driving. It was presumed this increase was due primarily to reconsolidation and remolding of soil at the soil-pile interface. Since these mechanisms are expected to affect the soil for only a small distance from the pile, it was anticipated that pile skin friction capacity would increase by a greater percentage than end bearing capacity.

In order to evaluate the rate of pile capacity increase, dynamic measurements were taken at the end of initial driving and during restrike one week later. Subsequent static loading tests performed on the piles provided pile capacity data at a third point in time. A linear relationship between pile shaft friction and time was assumed at the outset of the test program and test results substantially verified this for the time period considered. This paper briefly describes the test program with regard to the time dependent pile capacity characteristics for two of the test piles and presents our evaluation of the results.

DESCRIPTION OF TESTING PROCEDURES

The steel pipe and precast concrete piles under study were both initially driven in place with a Vulcan 80C hammer. One week later, the piles were restruck several blows with a Vulcan 010 hammer in order to obtain additional dynamic measurements. The lower energy 80C was used to drive the piles in order to evaluate its driving efficiency (as it was anticipated this hammer would be used during production driving). The 010 was used during restrike so that the greater hammer energy would be available to mobilize the anticipated increased pile capacity during the dynamic testing. A summary of pile and driving system specifications is shown in Table 1.

TABLE 1

	Pile Data
355 mm	Square precast concrete pile
	Concrete compressive strength: 45,000 kN/m^2
	Prestress magnitude: 5200 kN/m^2
323 mm	Steel pipe pile, driven closed end
	Steel yield strength: 310,000 kN/m^2
	Driving System Data
Vulcan Model 80C, air operated, differential	rated energy: 33,150 J at 41.9 cm stroke
Vulcan Model 010, air operated differential	rated energy: 44,070 J at 99 cm stroke

DYNAMIC TESTING AND ANALYSES

The pile load test program included dynamic pile monitoring both during initial installation and restrikes. Field testing was performed with a Pile Driving Analyzer TM (PDA) according to the Case Method procedures. Subsequent data analysis was done by the Case Wave Analysis Program Continuous version (CAPWAPC). The primary objectives of the tests were the evaluation of the hammer-driving systems performance, pile driving stress (both compressive and tensile), pile structural integrity, pile static bearing capacity, and soil behavior. Detailed descriptions of the field equipment and analytical procedures used may be found in several references (Goble et al, 1980). A brief discussion of the methods is also presented below.

Instrumentation: Dynamic measurements of strain and acceleration were taken near the top of each pile. Strain transducers are reusable frames with four resistance foil gages attached in a full bridge. Acceleration was measured with piezoelectric accelerometers that were mounted on special blocks for electronic and mechanical isolation and for ease of attachment to the piles. Two strain transducers and accelerometers were bolted at opposite sides of each pile to monitor and minimize the effects of non-uniform impacts.

The PDA is a state-of-the-art, user friendly, field digital computer. Basically, it computes some 40 different dynamic variables in real time between hammer blows after providing signal conditioning, amplification, filtering and calibration to the measured signals. Pile strains are converted to forces and accelerations to velocities as a function of time for each hammer blow. Force and velocity records are assessed for data quality and are evaluated according to Case Method equations. Dynamic variables are computed and are available for display and printing for each hammer blow. This data may include maximum pile driving forces, maximum pile top hammer transferred energy, and ultimate pile static capacity.

The analog signals of forces and velocities along with voice annotation were recorded on a seven-channel FM instrumentation tape recorder for future reference. An oscilloscope was used to monitor signals for data quality and possible pile damage.

Case Method: The techniques most widely employed today for both measurement and analyses of pile dynamic events were developed starting in the 1960's at Case Institute of Technology (now Case Western Reserve University) and hence, collectively referred to as the Case Method. This name actually covers a wide range of equations and procedures.

The Case Method of closed form solutions requires the measurement of force and velocity histories of the pile under a hammer blow. Using wave propagation theory and assuming a uniform elastic pile, the PDA applies the Case Method for determining some 40 dynamic variables in real time. The most interesting of these quantities are:

* Case Method axial pile capacity (Rausche et al, 1985)
* Maximum energy delivered to the pile, ram impact velocity, and hammer or pile cushion stiffness for hammer-driving system performance evaluation (Likins, 1982)
* Maximum pile compressive and tensile forces during driving to check for potentially damaging stresses
* Indication of location and extent of pile damage for pile integrity evaluation (Rausche and Goble, 1978)

For piles with little shaft resistance, a static toe resistance force displacement graph may be obtained from pile top measurements with one dimensional wave theory considerations.

The Case Method computes total driving resistance and relies on a damping factor for static capacity determination. This damping factor (called Case Damping) is dependent on the soil type and behavior under dynamic loading. In the field, a Case Damping factor has to be selected to represent the soil conditions prevalent at each site.

CAPWAPC: This is an analytical procedure performed interactively between the engineer and a computer program using a micro-computer. It was developed to compute soil resistance forces and their distributions using pile top force and velocity measurement recorded in the field in a wave equation type procedure (Rausche, 1970).

In order to perform the CAPWAPC analysis, the pile below the point where the gages were attached is modeled in the form of a series of segments of equal stress wave travel time. The soil reaction forces are passive and are assumed to consist of a static (elasto-plastic) and a dynamic (linearly viscous) component, both along the shaft and below the pile tip. In this way, the soil model has at each point three unknowns: elasticity, plasticity, and viscosity. To start the analyses, a complete set of wave equation type constants is assumed and entered into the computer model. Then in a dynamic analysis, the hammer model is replaced by the measured velocity imposed at the top pile element and CAPWAPC calculates the force necessary to induce the imposed velocity. The measured and calculated forces are both plotted as a function of time; if they do not agree, the soil model is changed and the analysis repeated. This iterative procedure is repeated until no further improvements between measured and computed forces can be obtained. Alternatively, the force may be imposed as the boundary conditions and the velocity computed.

Results from a CAPWAPC analysis include comparisons of measured with the corresponding computed force/velocity curves. Numerically for each segment of the pile, ultimate static resistance, soil quake and damping factors are tabulated. Also included in the results is a pile load-set curve from static test simulation.

Because they are calculated during the analysis, forces, velocities, displacements, and energies may be printed or plotted as a function of time for any pile segment.

Like static loading tests, dynamic pile testing computes pile capacity at the time of testing. Since it is possible to determine skin friction distribution and end bearing values from CAPWAPC, analyses done on data during restrike may be compared to that from end of driving for soil resistance change determination. This procedure was followed during this project to assess time dependent soil strength characteristics.

STATIC LOADING TESTS AND ANALYSES

Static axial compressive loading tests were performed on each of the described test piles in accordance with ASTM D 1143-81. Reaction frames were assembled and anchored to piles driven to resist the reaction load. The piles were loaded using a hydraulic jack in increments of 25% of the proposed service load, with the top of pile deflection being recorded by dial gauges. Successive incremental loads were applied only after the rate of penetration was measured at less than 0.25 mm/hr. Each pile was loaded in this manner to 200% of the service load, this load was then held constant for 12 hours. The pile was then incrementally unloaded. The load was then incrementally reapplied up to the previous maximum load. The load was then increased in increments of 10% of the service load until failure. Failure was defined as a total pile top deflection of 10% of the pile diameter for the purposes of this test.

Pile capacities were evaluated from the results of the loading test by the method suggested by Davisson (Davisson, 1970). In this procedure, the load versus pile top deflection data resulting from the loading test is plotted. The theoretical linear elastic compression of the pile is also plotted and a parallel to this plot is constructed at a distance of 3.81 + D/3048 (where D is the pile diameter in millimeters). The point of intersection of the constructed parallel and the loading test curve is defined as the pile capacity.

SUBSURFACE CONDITIONS

The geology of Central Florida is characterized by strata formed during three distinct geologic periods. The surficial stratum is composed of undifferentiated Holocene/Pliestocene/Pliocene series sands containing varying amounts of silt and clay. These sediments were deposited on terraces at the bottom of shallow seas during interglacial times when sea levels were higher than at present. The thickness of these surficial deposits typically range from about 9 to 18 m below ground surface. The Hawthorne formation, a marine deposit of Miocene age, underlies these terrace deposits. The Hawthorne is typically composed of green clay, clayey sand, sandy limestone and dolomite. In the area of the project site, it is estimated the Hawthorne is about 40 m thick. The Hawthorne formation

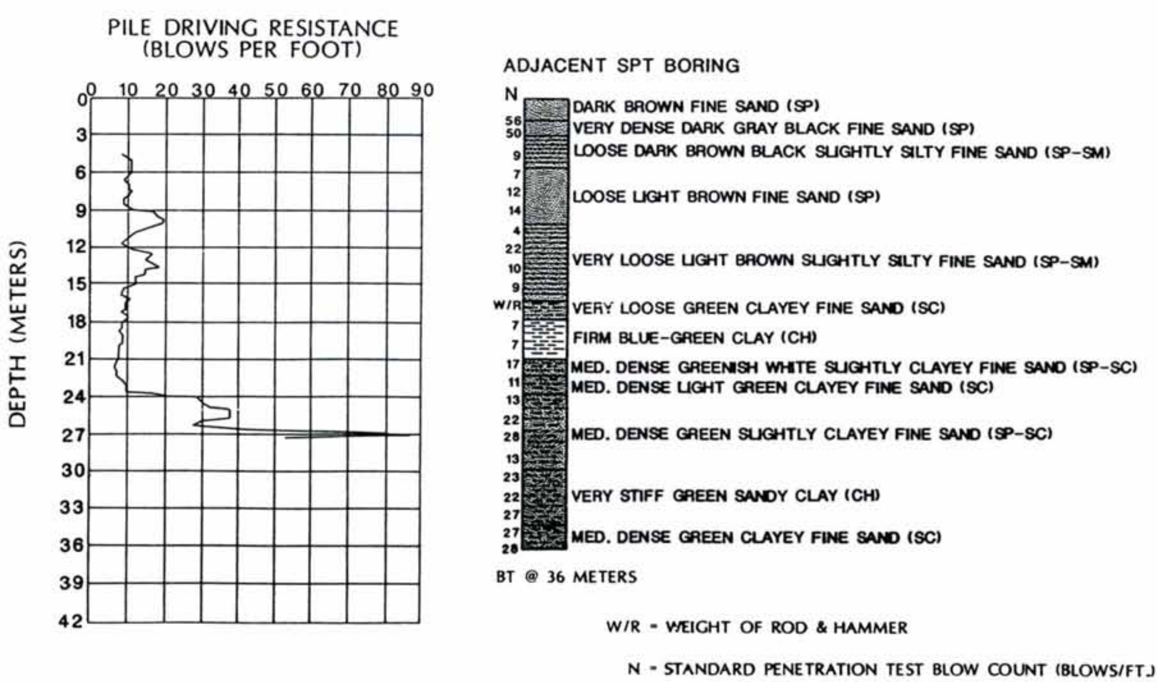


FIGURE 1. BORING LOG / DRIVING RECORD – 355 mm SQUARE CONCRETE PILE

is in turn underlain by the Ocala limestone, of Eocene age. It consists of cream to tan fine grained limestone and is estimated to be about 35 m thick at the study site.

The subsurface exploration of the project site consisted of performing a number of Standard Penetration Test (SPT) borings to depths of up to 54 m and Electronic Piezocone Penetrometer soundings to depths of up to 40 m below ground surface. Split spoon samples and undisturbed tube samples were obtained from the borings and were subjected to laboratory testing including: sieve analyses, unit weight, natural moisture content, Atterberg limits, and consolidation tests.

Subsurface soil conditions across the site were found to be quite variable. The results of SPT borings performed in the vicinity (within 15 m) of each test pile, along with the pile driving records, are shown on Figures 1 and 2. Descriptions of the soils encountered in the borings are accompanied by the Unified Soil Classification Symbol (SP, SC, etc.) based on visual examination and laboratory testing.

At both the test pile locations, the borings encountered fine sand (SP) and slightly silty fine sand (SP-SM) to depths of about 15 m. The boring adjacent to the concrete test pile encountered these sands primarily in a loose (SPT blow count N=4-10) to very loose (N=0-4) condition. Adjacent to the steel test pile, these sand strata were encountered in a medium dense (N=10-30) condition. Below these surficial strata the borings encountered about 2 m of very loose (N=0-4) clayey fine sand (SC) grading to a firm (N=4-8) to very soft (N=0-2) clay (CH) extending about 5 m. Below this, strata of medium dense (N=10-30) to dense (N=30-50) clayey fine sand (SC) and slightly clayey fine sand (SP-SC) were encountered to depths of 30 m or more.

The majority of the cohesive soils tested at the site were found to be overconsolidated by ratios ranging from 1.5 to 4. The natural water content of the clays ranged between 40% and 50% and void ratios between 1.05 and 1.35. Liquid limits and plastic limits ranged from 27 to 190 and 19 to 52, respectively.

DISCUSSION OF DYNAMIC AND STATIC TESTING RESULTS, CONCRETE PILE

The concrete pile was initially driven with the Vulcan 80C hammer to a penetration of 27 m and a driving resistance of 68 blows per foot. The end of driving pile top compressive stress and transferred energy averaged 13,790 kN/m^2 and 11.5 kJ, respectively. The pile static capacity was calculated by CAPWAPC to be 1,135 kN (205 kN in skin friction and 930 kN in end bearing), most of the skin friction (65%) was acting along the bottom 2 m of pile. Plotted CAPWAPC results for this analysis are presented in Figure 3.

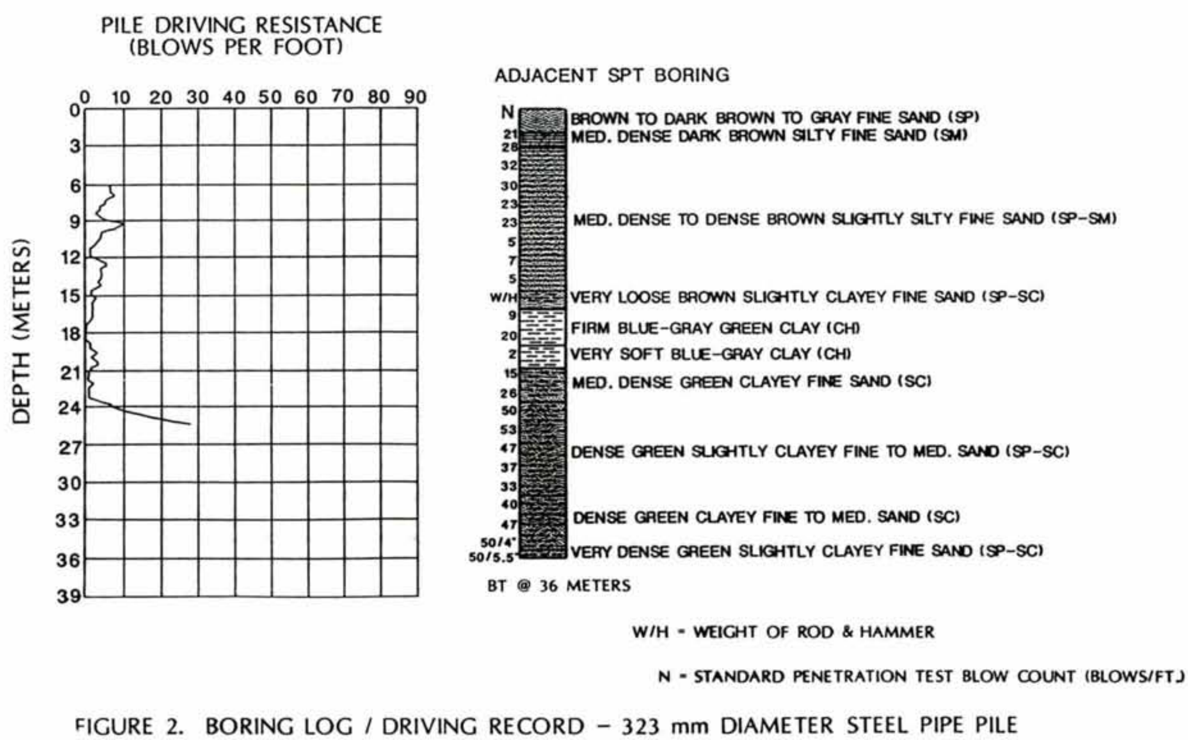


FIGURE 2. BORING LOG / DRIVING RECORD – 323 mm DIAMETER STEEL PIPE PILE

One week later, this pile was restruck with the Vulcan 010 hammer. The driving resistance, transferred energy, and pile top compressive stress were 13 blows for 50 mm, 26 kJ, and 22,130 kN/m^2, respectively. Data representing the second blow of restrike was analyzed with CAPWAPC which computed an ultimate static capacity of 2,310 kN. As expected, all the capacity increase (1,170 kN) was added skin friction. Restrike values represent a doubling in the total pile capacity and a 6.7 times increase in skin friction from those at the end of driving. CAPWAPC analyses results are summarized in Table 2 below.

TABLE 2
SUMMARY OF CAPWAPC RESULTS
FOR CONCRETE PILE

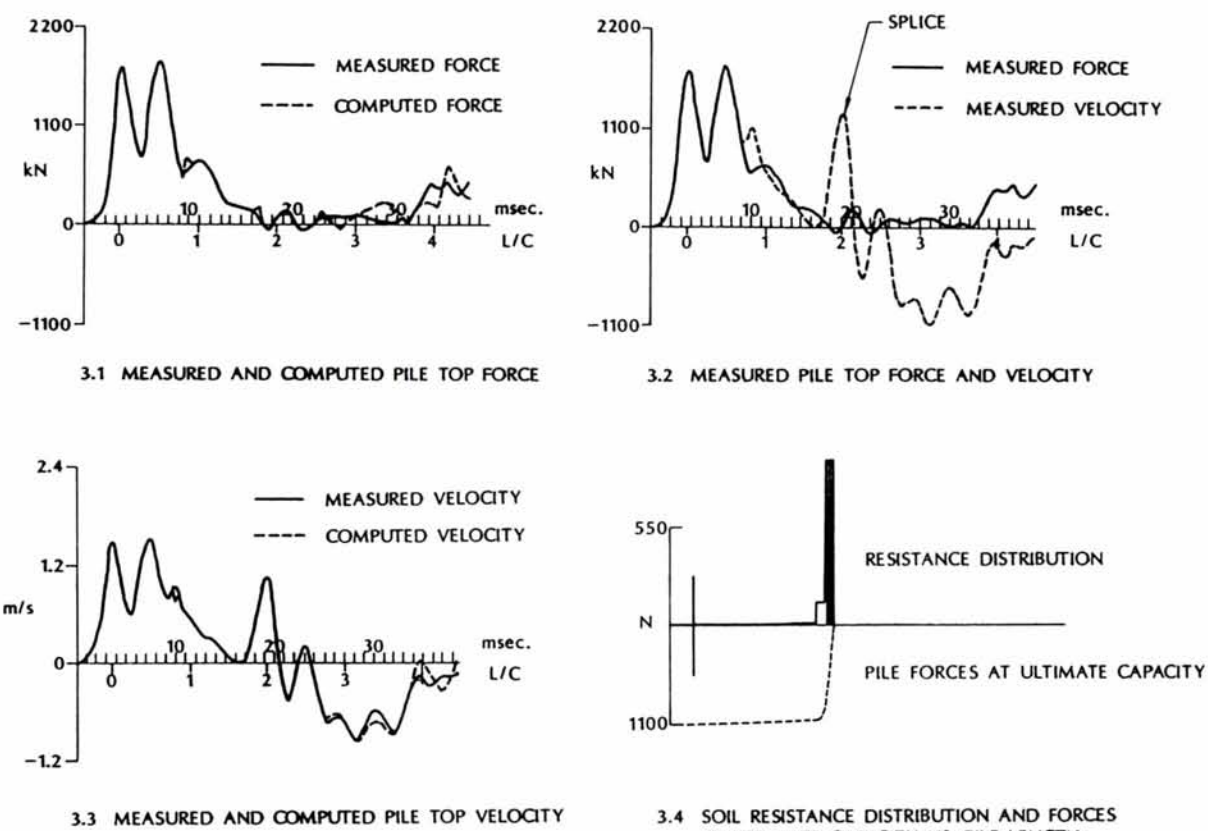
Event Analyzed	Pile Capacity: Skin Friction (kN)	Pile Capacity: End Bearing (kN)	Total (kN)
End of Initial Driving	205	930	1135
2nd Blow of Restrike	1375	935	2310

A simple linear extrapolation was employed to predict the pile's capacity 16 days after initial installation. Numerically, the process was done as follows:

Capacity after 16 days =
$935 + 205 + 16(1375-205)/7) = 3814$ kN

It assumes a uniform constant rate of skin friction increase of 167 kN/day, which adds up to a total skin friction value of 2880 kN; an increase of 14 times over that at the end of driving. This prediction procedure is illustrated graphically in Figure 4.

Sixteen days after initial driving, this pile was subjected to a static load test that indicated an ultimate pile capacity of 3692 kN according to the Davisson's failure criteria (see Figure 5 below). The difference between the linearly extrapolated dynamic capacity prediction and that of the static load test result was 3 percent.



3.1 MEASURED AND COMPUTED PILE TOP FORCE

3.2 MEASURED PILE TOP FORCE AND VELOCITY

3.3 MEASURED AND COMPUTED PILE TOP VELOCITY

3.4 SOIL RESISTANCE DISTRIBUTION AND FORCES AT ULTIMATE CAPACITY VS. PILE LENGTH

FIGURE 3. RESULTS OF DYNAMIC TESTING FOR CONCRETE PILE

DISCUSSION OF DYNAMIC AND STATIC TESTING RESULTS, STEEL PIPE PILE

Utilizing the Vulcan 80C hammer, the steel pipe pile was driven to a depth of 25 m and a driving resistance of 27 blows per foot. Towards the end of driving, the maximum pile top compressive stress averaged 150,600 kN/m^2, the maximum transferred energy averaged 7.5 kJ, and the CAPWAPC ultimate capacity was 630 kN (80 kN in skin friction and 600 kN in end bearing). One week later, the pile was restruck with the Vulcan 010 hammer. The pile exhibited a driving resistance of 8 blows for 50 mm of penetration. The pile top compressive stress and transferred energy averaged 196,500 kN/m^2 and 27 kJ respectively. Data from the first blow of restrike was analyzed with the CAPWAPC program and it showed an ultimate capacity of 1230 kN (795 kN in skin friction and 435 kN in end bearing). This represents a 9.8 times increase in skin friction. It was believed that due to the minimal pile set under the first blow, the restrike indicated end bearing value was only a mobilized and not an ultimate value.

CAPWAPC analyses results are summarized in Table 3 below.

TABLE 3
SUMMARY OF CAPWAPC RESULTS
FOR STEEL PILE

Event Analyzed	Pile Capacity Skin Friction (kN)	End Bearing (kN)	Total (kN)
End of Initial Driving	80	600	680
1st Blow of Restrike	795	435	1230

A linear extrapolation was used to predict the total pile capacity 14 days after initial driving. The following numerical process was used assuming end bearing from end of drive and a similar increase in skin friction during the second

Figure 4 – TIME VS. CAPACITY

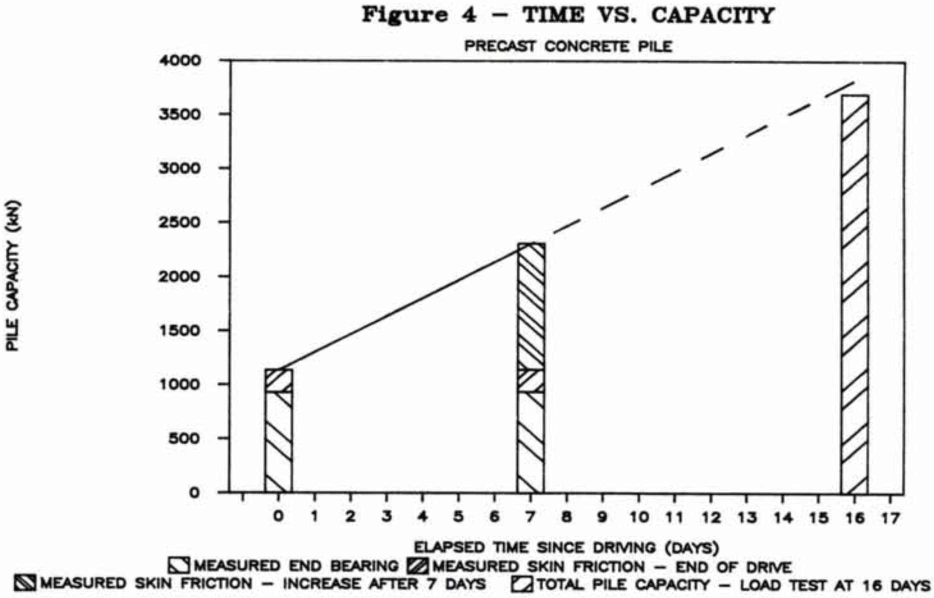
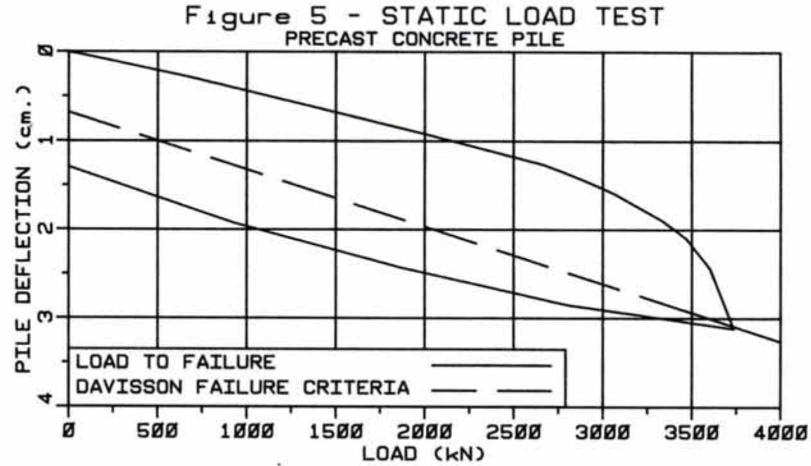


Figure 5 - STATIC LOAD TEST



7 day period as that experienced on the first 7 days:

Capacity after 14 days =
600 + (80 +2(795 - 80)) = 2110 kN

This represents a 3.1 times increase in total pile capacity; if only skin friction is considered, it increased by a factor of 19 times. This is shown graphically on Figure 6.

A static load test performed 14 days after initial driving indicated an ultimate pile capacity of 2240 kN, according to Davisson's failure criteria (see Figure 7). The difference between predicted and verified pile capacity was 6 percent.

CONCLUSION

Analysis of the results of the pile test program yielded data significant to the design of this project and suggested a general approach for estimating the capacity of piles with time dependent characteristics. First, an increase in pile capacity with time after initial driving was observed in the two case histories presented (it was also observed in all of the 6 other test piles involved in the study). Second, the magnitude of this increase was significant, a two to three-fold increase in total pile capacity. Third, the increase appeared to occur almost entirely in skin friction capacity. And fourth, the capacity increase was found to be linear over the time period considered (14 to 16 days).

Another observation can be made by comparing the rate of capacity increase of the steel and concrete piles. In the case histories presented, this rate was 167 kN/day for the concrete and 102 kN/day for the steel. The differing rate may be attributable to differences in the subsurface conditions at the pile locations. Alternatively, the greater rate of increase for the concrete pile might be related to its absorption capacity (allowing more rapid reconsolidation) and higher coefficient of surface friction.

Figure 6 – TIME VS. CAPACITY

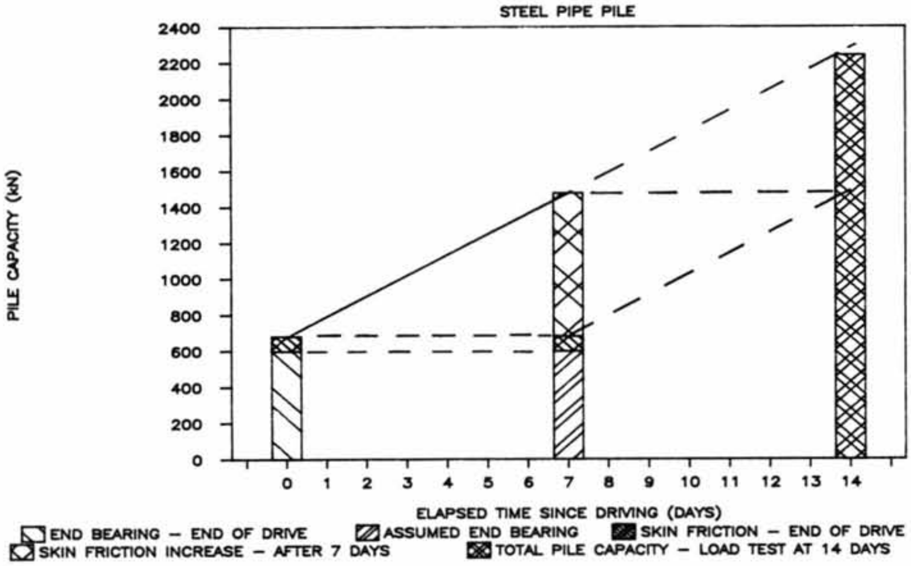
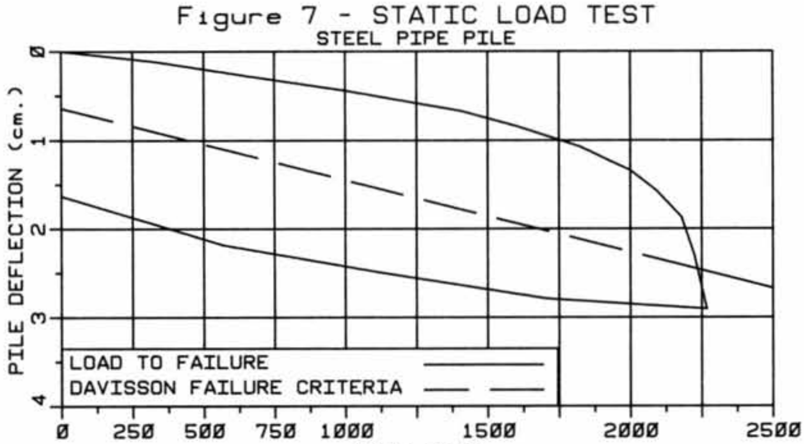


Figure 7 - STATIC LOAD TEST



For this project, the time dependency of the pile bearing capacity was taken into consideration in several aspects of the design and installation of the piles. Due to the highly variable subsurface conditions, it was assumed that pile driven lengths would also vary, and would be quite dependent on the acceptance criteria adopted. The results of our testing indicated that if driving acceptance criteria was adopted based on the pile capacity indicated during initial driving, the piles would be significantly over-driven with regard to the service load. Therefore, it was decided to develop an acceptance criteria based on the indicated capacity during driving plus an assumed increase in skin friction. The assumed increase would be calculated based on the rates observed in our test program and assuming maximum capacity would be achieved at 14 days after installation.

Fourteen days was selected both because the calculated capacities at this time correlated well with our ultimate static capacity analyses and also because it was felt a restrike testing program could be conducted during production installation on this basis without delaying construction unduly.

Recommendations for the production pile installation included restriking about 10% of production piles 10 to 14 days after initial driving. Dynamic measurements would be made during restrike using the PDA. Analysis of PDA results, including CAPWAC, would provide a check on our acceptance criteria so that it could be revised, if necessary, as construction progressed.

Another aspect of the foundation design which was affected by the time dependency of pile capacity was the comparison of the capacity of different pile types. Part of the purpose of the test program was to evaluate the relative cost efficiency of different pile types. From the results of our study, it was apparent that a valid comparison of pile capacities could not be made without taking into account the time between initial driving and static loading tests. Therefore, estimated driven lengths of each pile type were based on the calculated "two-week capacity" described above.

The pile test program described herein provided data regarding the time-dependent capacity characteristics of the soils at this site which were invaluable in developing an efficient foundation design. Where soil conditions and local experience indicate significant set-up, an attempt to quantify the rate of set up may be beneficial. The use of the PDA during initial driving and during subsequent restrikes provides a means to develop capacity versus time relationships. While the linear relationship observed in this study will likely not always be applicable under different conditions, the authors suggest it may be a reasonable assumption for preliminary analyses.

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