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SYNOPSIS

A pile driving and testing program was undertaken at a construction site for a highway bridge in Florida, USA. Fifteen prestressed concrete piles, 762 mm (30 inch) square with a 457 mm (18 inch) circular void throughout their lengths, were driven as probe piles with a single acting air hammer. Subsurface conditions were variable consisting of layers of loose sand, sandy clay and clayey sand underlain by limestone bedrock. All piles were dynamically tested with a Pile Driving Analyzer during initial installation and restrikes. Further analysis of the dynamic data was performed according to the CAPWAP Method. Two of the piles were statically tested, to failure, in compression and uplift. Correlations between dynamically predicted and statically measured pile compression capacities and pile head load-movement relationships agreed well. It was concluded that uplift pile capacities were approximately 60% of the total compression values and an average of approximately 75% of compression shaft resistance. This paper presents discussions on pile installation and testing procedures and results, with particular emphasis on the two statically tested piles.

INTRODUCTION

A new structure planned to carry two lanes of vehicular traffic along State Route 79 over the Intracoastal Waterway near the Gulf of Mexico in northern Florida, U.S.A. will replace an existing antiquated steel vertical lift bridge. The 810 m (2657 ft) long structure consists of a concrete and steel bridge that includes a 76 m (220 ft) long main span with a maximum vertical clearance at mid-length of 20 m (65 ft) over water. Approach spans range in length between 35 and 53 m (116 and 175 ft). Structural loading, subsurface conditions and other factors dictated that bridge foundations consist of driven prestressed concrete piles.

Stratigraphic nomenclature for the upper geologic formations in the project area indicates that the soils within the depth of exploration were deposited during the Pliocene, Pleistocene and recent Epochs of the Neogene and Quaternary Periods in the Cenozoic Era. Subsurface soil conditions at the project site were investigated by Standard Penetration Test (SPT) borings and cone penetration test (CPT) soundings. Generally, the soil conditions along the bridge alignment were variable consisting of loose clean to silty clayey sands extending to depths between 9 and 27 m (30 and 90 ft) underlain by medium dense clayey sand interbedded with partially cemented zones and layers of shell overlying competent limestone at a depth of about 46 m (150 ft). Some borings indicated layers of soft to stiff clay, 1 to 3 m (3 to 9 ft) thick, between the two sand strata.

Each pile was designed for a compressive service load of 1560 kN (350 kips) and was required to reach a maximum resistance of 3340 kN (750 kips). Piers located within the waterway were designed to resist a lateral load due to ship impact of 8900 kN (2000 kips) by utilizing inclined piles. Piles driven in the water were required to reach a maximum uplift resistance of 1560 kN (350 kips) each.

A pre-construction pile driving and testing program was undertaken to evaluate installation equipment and procedures, assess pile drivability and capacity, and provide foundation design parameters and installation criteria. Fifteen test piles were driven. All test piles were dynamically monitored during initial installation and also during restrikes; two piles were statically tested to failure both in compression and uplift. This paper presents discussions on the pile installation and testing procedures and results with particular emphasis on the evaluation of compression and uplift capacities of the two statically load tested piles.

Since the original work for this project was done using the English System of units, soft conversions were used to express values in SI units (along with English values in brackets). A table for English to SI units conversion is appended at the end of the paper.

PILE INSTALLATIONS

A total of fifteen (15) test piles were driven in production pile locations spaced at an average interval of about 52 m (170 ft) along the full bridge length. Due to adverse weather conditions and other site related issues, installing the test piles occurred over a period of four months. This allowed for evaluation of time related pile capacity changes by testing some piles during restrike a few months after their initial installation.

Piles

The fifteen test piles were 762 mm (30 inch) square precast prestressed concrete sections with a 457 mm (18 inch) circular void throughout their lengths (effective concrete area = 4165 cm^2 (645.5 in^2)). Pile section lengths ranged between 14 and 41 m (45.5 and 135.0 ft). Six of the piles had to be spliced to extend their lengths due to insufficient resistance; spliced pile lengths ranged from 47 to 50 m (154 to 163 ft). A reinforced concrete collar type splice was employed. Pile concrete compression strengths, based on 28-day cylinder tests, were required to be 41 MPa (6 ksi). According to project specifications and considering pile specifics, maximum allowable pile dynamic compressive and tension stresses during driving were 24 and 9 MPa (3.5 and 1.3 ksi), respectively. Spliced piles had lower allowable tension limits. Five piles were driven at a 3:1 (vertical:horizontal) inclination, all remaining were vertical.

Driving Equipment

Pile driving and restriking were accomplished with a Conmaco 300-E5 single acting air hammer. This particular hammer model had a ram weight of 133.5 kN (30 kips) and was fitted with a slide bar allowing it to operate at either 0.45 or 0.91 m (1.5 or 3.0 ft) strokes (corresponding rated energies of 61 and 122 kJ (45 and 90 kip-ft), respectively). Pile top cushions consisted of sheets of plywood with a total thickness of 250 mm (9.75 inches).

End of driving pile penetrations ranged between 11 and 48 m (36 and 158 ft), and driving resistances between 30 blows per 0.3 m (30 bl/ft) and absolute refusal. Most piles were restruck, in some cases several times, after initial installation.

DYNAMIC PILE TESTING

Preliminary pile drivability evaluations and dynamic analyses were performed using the GRLWEAP™ wave equation analysis program (1). All fifteen test piles were dynamically monitored during initial driving, and after splicing for those that were spliced. The majority of the piles were also dynamically tested during restrike, several times in some cases. Field dynamic testing was accomplished with a Pile Driving Analyzer™ (PDA) and subsequent data analyses were performed according to the CAPWAP™ Method (2). The PDA provided real time data analysis and results for field assessment of pile driving stresses and structural integrity, hammer\driving system performance, and pile driving resistance and mobilized static capacity. CAPWAP analysis results included mobilized static pile capacity, soil resistance distribution and dynamic behavior under hammer impacts, and pile head and toe static load versus movement relationships.

The Pile Driving Analyzer (PDA)

Dynamic measurements of strain and acceleration were taken approximately four feet below the head of each pile under hammer impacts during initial installation and restrikes. Pile strains were measured with reusable strain transducers and accelerations were measured with reusable piezoelectric accelerometers. Two each strain transducers and accelerometers were bolted at opposite sides of each pile to monitor and cancel (by averaging) effects of non-uniform hammer impacts. The Pile Driving Analyzer (PDA) is a state-of-the-art, user friendly data acquisition, processing and storage field digital computer. Figure 1 presents a photograph of a Model PAK Pile Driving Analyzer. It provides signal conditioning, amplification, filtering and calibration to measured signals. It converts measurements of pile strain into force and acceleration into velocity, checks data quality and applies Case Method equations to compute some 40 different dynamic variables in real time between hammer blows. In addition to dynamic pile records and gage calibration factors, required PDA inputs include pile length, area, elastic modulus (or longitudinal stress wave speed), density, and soil damping factor. After each hammer blow, the PDA computes many variables, the most interesting of which are: pile driving resistance and static capacity, pile driving compressive and tensile stresses which are compared to maximum allowable values so that pile damage is avoided, an integrity factor indicating pile structural integrity including assessment of extent and location of damage if present, and maximum energy transferred to pile head and corresponding energy transfer ratio (i.e., energy transfer divided by rated energy) for evaluation of hammer\driving system performance. In the field, dynamic records are displayed on the PDA's high resolution screen and are digitally stored in permanent memory for subsequent analyses. Testing results from each blow are also stored for later plotting as a function of pile penetration.

During pile installations, maximum pile compressive stresses (at gage locations) reached 24 kPa (3.5 ksi) and maximum shaft tension stresses reached 7 kPa (1 Ksi). Dynamic pile records indicated some damage near the toe of some of the inclined piles. Energy transfer ratios (maximum transferred energy divided by rated energy) ranged between 30 to 40 percent. End of driving Case Method computed static pile capacities ranged between 445 kN (100 kips) and 5030 kN (1130 kips). Testing during pile restrikes indicated noticeable increases in static capacities with time.

CAPWAP Method

The CAse Pile Wave Analysis Program (CAPWAP) is used to analyze field recorded dynamic pile records in a systems identification procedure using signal matching techniques (3). Performed interactively between the engineer and computer (although an automatic program solution is possible), soil resistance forces are computed as a result of a best fit match between measured and simulated pile top data in a wave equation analysis type procedure. In order to perform the analysis, the pile is modeled as a series of segments of equal stress wave travel time and soil reaction forces are applied to pile segments below ground level. The soil reaction forces are assumed to consist of a displacement dependent elastic-plastic static component and velocity dependent linear viscous damping dynamic component. To start the analysis, a complete set of soil resistance variables is assumed, a measured pile dynamic record (e.g., velocity) is imposed to the model and a resulting pile top dynamic record (e.g., force) is computed which is then compared to the corresponding measured data (force); if they do not agree, then the process is repeated with another set of soil resistance assumptions until a good match between computed and measured records is achieved. Alternatively, the force may be imposed as a boundary condition and the velocity computed. Results from a CAPWAP analysis include comparisons of measured with corresponding computed force\velocity curves, plots of simulated pile head and toe static loading tests showing applied loads and corresponding pile movements, a histogram of static soil resistance forces as a function of pile length and a plot of pile forces at ultimate soil resistance. Figure 2 presents a typical CAPWAP analysis plotted results. Numerically, for each pile segment, ultimate static soil resistance, soil quake and damping factors are tabulated.

Figure 1: The Pile Driving Analyzer

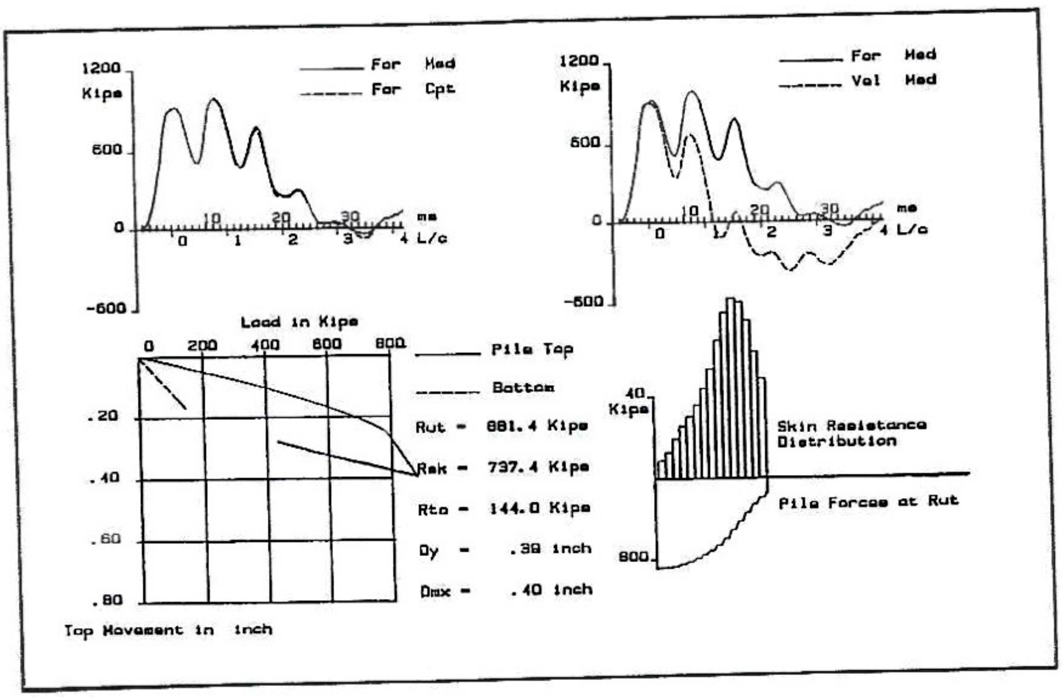


Figure 2: CAPWAP Analysis plotted results

CAPWAP analyses were performed on data representing end of driving and\or restrike situations of all test piles. Generally, CAPWAP computed static pile mobilized capacities agreed well (within 10% difference) with values computed by the PDA in the field. Analyses performed with data from end of driving indicated an average skin quake (maximum elastic soil deformation) of 2.5 mm (0.1 inch) and average toe quakes of 17 mm (0.7 inch) and 4 mm (0.15 inch) for situations when pile toes were in the overburden and bearing layer, respectively. Soil damping factors along pile shafts and under toes averaged 2 s/m (0.6 s/ft) and 0.6 s/m (0.2 s/ft), respectively. Analysis with data from restrikes indicated somewhat higher skin damping factors and lower toe quakes than end of driving situations. The ratio of shaft friction to total pile capacities varied depending on whether the piles reached the limestone or not.

STATIC LOAD TESTS

Two of the test piles were tested to failure using static loading in compression and tension. The two piles will be referred to here as Test Pile 1 and Test Pile 2. The compressive and tensile testing loads were applied in general accordance with the Florida Department of Transportation (FDOT) modified quick test procedure, which is similar to the ASTM quick test method (ASTM D1143 section 5.6). Using this procedure, the load is applied to the pile in increments equal to approximately 5% of the specified maximum test load (i.e., 8000 kN compression and 2700 kN in tension) until "failure" load is reached. Each load increment is applied immediately after a complete set of gages and instrument readings is taken, usually within five to fifteen minutes per increment. The load is removed in decrements of 10% of the maximum test load obtained following the same procedure as that during the loading process.

Compression load was applied to each pile using an 8900 kN (2000 kips) hydraulic jack reacting against two steel girders. A load transfer assembly was utilized to transfer load from the reaction beam to four groups of five steel HP 14X89 reaction piles driven to depths of approximately 21.5 m (70 ft). The H-piles resisted the compressive load applied to the test pile through uplift side friction. A 10700 kN (2400 kip) load cell and calibrated pressure gage were used to verify the imposed loading magnitudes. Tension was applied using a tension load transfer assembly and a 4450 kN (1000 kip) hydraulic jack. The same reaction beam and H-pile system that was used for the compression tests was used for the tension tests. The H-piles resisted the tensile loads on the test piles through compressive shaft resistance and end bearing.

The primary system used to monitor pile movements consisted of two dial gages that measure in increments of 0.025 mm (0.001 inch) and have 50 mm (2 inch) of travel. The dial gages were mounted on independent wood reference beams located along opposite sides of the test pile. The dial gage stems were aligned parallel to the longitudinal axis of the pile. A secondary system employed to record pile movements included a wire, mirror and a scale marked with 0.25 mm (0.01 inch) increments. The mirror and scale were attached to the pile with the wire aligned such that it passed across the face of the scale. Movements were measured by aligning the wire with its reflection in the mirror and recording the corresponding scale reading. As a further check on pile movements, a scale was attached to the pile head and movement was monitored using a surveyor's level. In addition, four reaction piles (one in each group) were monitored for movement by reading attached scales with a surveyor's level.

The project's special provisions required that a modified Davisson Method be used to establish the pile ultimate load. For piles greater than 60 cm (24 inch) in diameter (width), the ultimate load is defined as the load that causes a pile head movement equal to the calculated elastic pile compression plus one thirtieth of the pile width. For interpretation of the static loading test results of the two test piles, the material modulus of elasticity used to calculate the elastic pile compression was estimated based on results of the CAPWAP analyses.

To evaluate the contribution of skin friction and end bearing to the total pile capacity, the relative movement between the pile head and toe was measured using two tell-tales located at opposite corners of the pile. Each tell-tale consisted of an unstressed steel rod placed in a constant diameter PVC pipe that rests on a steel plate located approximately 30 cm (12 inch) above the pile toe. Two dial gages that measure in 0.025 mm (0.001 inch) increments and have one inch of travel were mounted on the top of the pile. The dial gage stems rested on flat steel plates fixed to the top of the tell-tale rods.

Test Pile 1

The SPT boring closest to this test pile indicated that the soil conditions consist of approximately 27 m (90 ft) of alternating loose and medium dense sand layers underlain with stiff clay to a depth of 34 m (111 ft). SPT N-values generally ranged between 4 and 15 down to a depth of 34 m (111 ft). The test pile was 32.2 m (109 ft) long and was initially driven to a penetration of 31.7 m (104 ft). Dynamic testing indicated an end of driving static pile capacity of 2580 kN (580 kips) and a restrike capacity one hour after driving of 3578 kN (804 kips). Driving resistance increased from 40 to 60 blows per 300 mm (1 foot) between end of driving and beginning of restrike. Two weeks after initial installation, the pile was subjected to a compression static loading test. The result was a plunging failure at 3780 kN (850 kips). Maximum pile head movement was 29 mm (1.15 inch) after rebound and the movement at twice the design load of 3100 kN (700 kips) was 5.3 mm (0.21 inch). Dynamic testing performed during restrike nineteen days after end of initial driving indicated a static pile

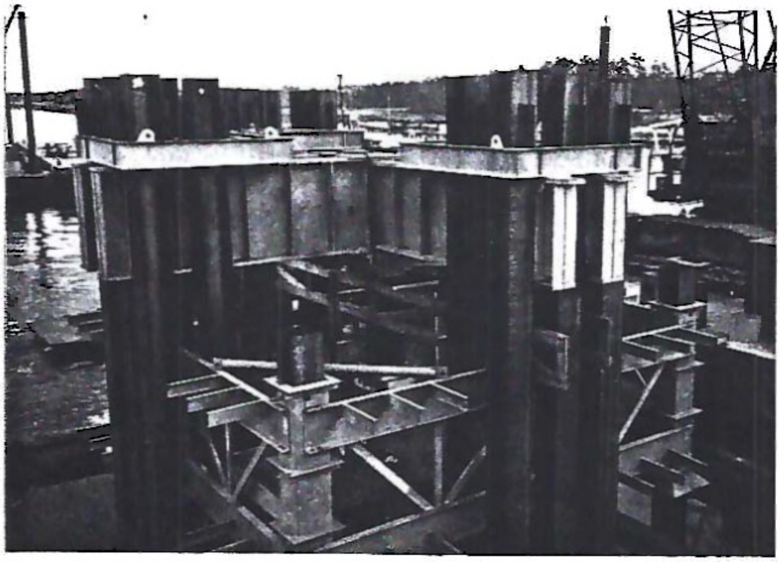
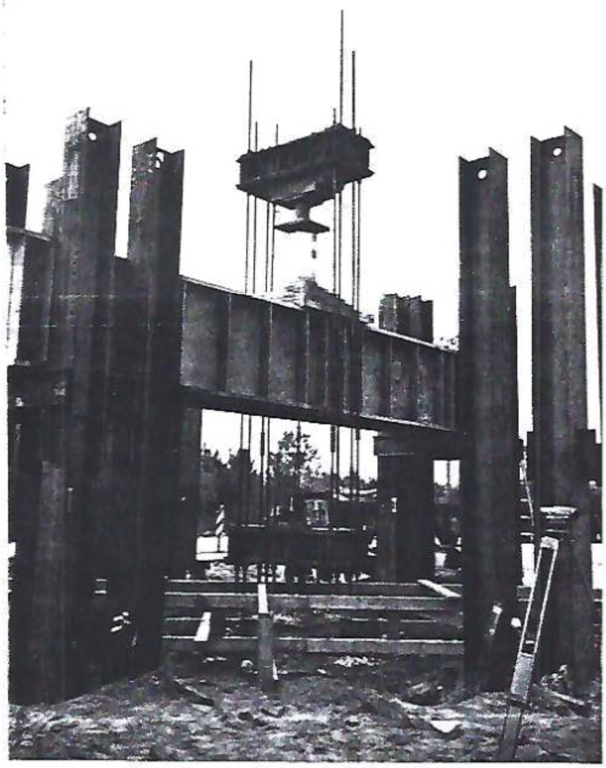


Figure 3: Compression static loading test

Figure 4: Tension static loading test



capacity of 3930 kN (881 kips). Six days after the compression loading test, the uplift loading test was performed. During the test, the pile experienced a pullout failure at 2225 kN (500 kips), with 6.4 mm (0.25 inch) of movement at the required uplift resistance of 1560 kN (350 kips). Figure 5 presents pile head load-movement plots from both static loading tests on this pile. Based on tell-tale data and CAPWAP analysis of restrike dynamic data, the ultimate compression shaft resistance of this test pile was determined to be 3000 kN (675 kips). Considering all testing results, it was concluded that the ratio of uplift to compression static pile capacities was 59% and that the uplift static pile capacity was 74% of compression shaft resistance.

Test Pile 2

Based on the closest SPT boring, soil conditions at this test pile location consisted of very loose to loose sands to 21 m (70 ft) underlain by 9 m (30 ft) of soft to firm clay. A layer of medium dense clayey sand extended from the bottom of the clay to the termination of the boring at a depth of 38 m (124 ft). SPT N-values ranged between 3 and 18 from ground surface to a depth of 38 m (124 ft). The pile length was 41 m (135 ft) and it was driven to a depth of 35 m (115 ft). Dynamic pile testing and data analysis indicated an end of driving static pile capacity of 3880 kN (760 kips) at a driving resistance of 5 blows per 25 mm (1 inch). Towards the end of driving, maximum pile top transferred energy averaged 50.3 kJ (37 kip-ft) with a maximum compression force of 6980 kN (1570 kips). Data from a one hour restrike test indicated a static pile capacity of 4187 kN (941 kips). The pile movement during the restrike was 38 mm (1.5 inch) under 10 hammer blows and energy transferred and compression force were slightly higher than those observed at the end of driving. The static compression loading test was performed one month after initial pile installation. The result was a plunging failure at 4230 kN (950 kips). The maximum pile head movement was 33 mm (1.3 inch) after rebound and the movement at twice design load was 6.8 mm (0.27 inch). Twelve days after the compression static loading test, the uplift test was performed. The pile experienced a pullout failure at 2450 kN (550 kips), with 8.1 mm (0.32 inch) of movement at the required uplift capacity of 1560 kN (350 kips). Plots of pile head load versus movement from both static loading tests of this pile are presented in Figure 6. Based on tell-tale data and CAPWAP analysis results, it was concluded that compression shaft pile capacity was approximately 3340 kN (750 kips). Results of static loading tests and analysis of dynamic pile records indicated that uplift static pile capacity was approximately 58% of compression pile capacity and that uplift pile capacity was approximately 73% of compression shaft friction.

In general, both piles drove and behaved as expected according to the soil conditions. Dynamic measurements during initial pile installations and static loading tests indicated that pile capacities increased with time. This capacity increase occurred shortly after the end of driving. This

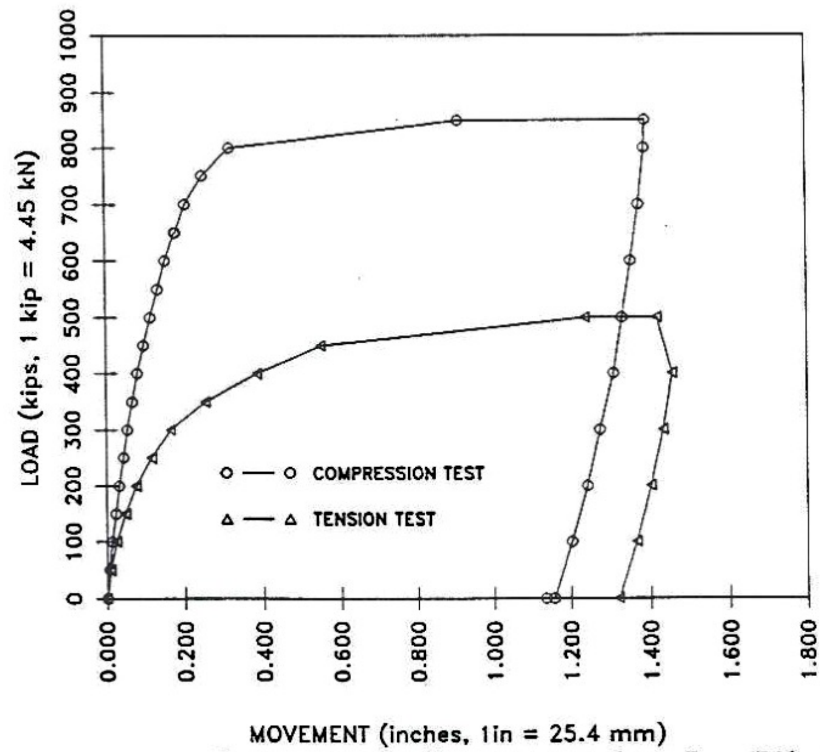


Figure 5: Static loading test results, Test Pile 1

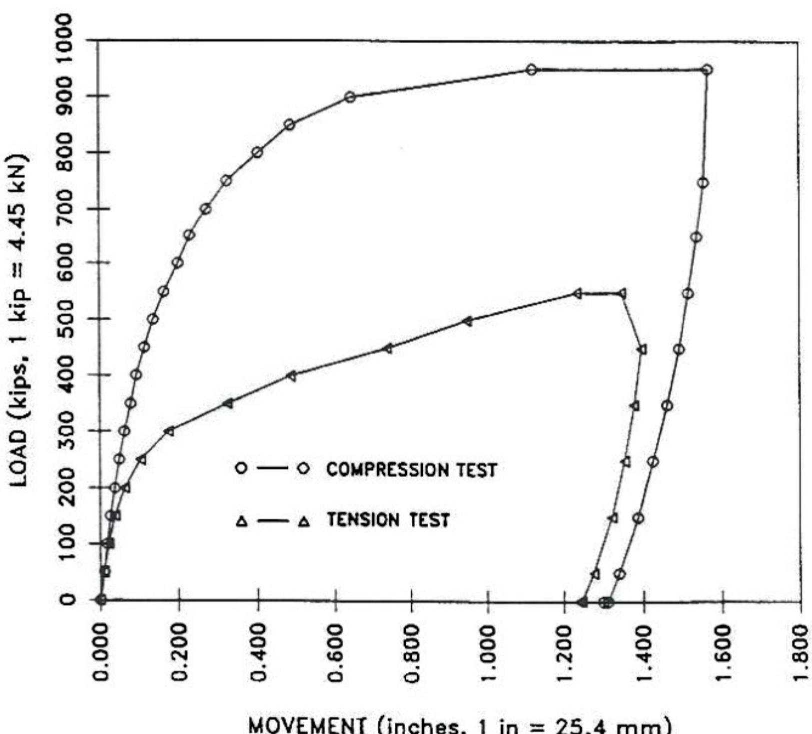


Figure 6: Static loading test results, Test Pile 2

information was useful for the timing of restrikes during production pile driving. The ratios between uplift and compression pile resistances was used to estimate the uplift capacities of the remaining test piles. Tell-tale data was useful as it allowed for a confirmation of the CAPWAP estimates of shaft resistance.

CONCLUSIONS

Fifteen prestressed concrete piles were driven as probe test piles at a proposed bridge construction site. Dynamic pile testing was performed, using a Pile Driving Analyzer, during initial pile installations and also during restrikes. Dynamic records were further analyzed according to the CAPWAP Method which allowed for computation of static pile capacity and separation of shaft and end bearing resistances. Two of the test piles were statically load tested to failure both in compression and tension. Correlations between dynamically predicted and statically measured pile capacities agreed very well. Similarly, analysis of dynamic pile records yielded simulated pile static load versus movement relationships and soil resistance distributions that agreed well with those determined from static loading tests. Static pile capacities increased with time with the majority of resistance gain occurring shortly after end of pile installation. Testing of two piles showed that uplift pile capacities were approximately sixty percent of compression capacities. It was concluded that uplift pile capacities were approximately seventy five percent of compression shaft resistance values. This conclusion agrees with findings reported from tests performed on prestressed concrete pile at another site more than 750 km away (4).

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REFERENCES

1. HUSSEIN, M., RAUSCHE, F., and LIKINS, G. Wave equation analysis of pile driving: methodology and performance. Proceedings of the Sixth National Conference on Microcomputers in Civil Engineering, University of Central Florida and American Society of Civil Engineers, Orlando, Florida, November 1988, 80-85.

2. LIKINS, G., HUSSEIN, M., and RAUSCHE, F. Design and testing of pile foundations. Proceedings

of the Third International Conference on the Application of Stress Wave Theory to Piles, Bengt Fellenius ed., Ottawa, Canada, May 1988, 644-658.

3. RAUSCHE, F. Soil response from dynamic analysis and measurements on piles. Ph.D. Dissertation, Case Western Reserve University, Cleveland, Ohio, 1970, 168 p.

4. HUSSEIN, M. and SHEAHAN, J. Uplift capacity of driven piles from static loading tests. Proceedings of the Third International Conference on case Histories in Geotechnical Engineering, University of Missouri-Rolla, St. Louis, June 1993, 1-12.

APPENDIX

1 ft = 0.305 m, 1 inch = 2.54 cm, 1 kip = 4.45 kN, 1 psi = 6.89 kPa, 1 kip-ft = 1.36 kJ.