Dynamic and Static Tests on Driven and Cast-in-Place Piles

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Abstract

A pile installation and testing program was undertaken at the site of a proposed bridge replacement project in South Carolina, USA. Subsurface conditions at the test site consisted of 14 m of sand and clay, 0.6 m thick caprock layer over a deep underlaying stratum of calcareous sand. Three prestressed concrete piles, one steel H-pile and one drilled shaft were studied. All driven piles were dynamically monitored during installation with a Pile Driving Analyzer according to the Case Method. Some piles were also dynamically tested during restrike to evaluate time dependent pile capacity changes. All concrete piles, including the drilled shaft, were additionally tested using the P.I.T. dynamic low strain method for structural integrity assessment. Two of the driven piles were statically load tested to failure. The drilled shaft was tested up to the capacity of the loading system (8900 kN). One of the driven concrete piles was not subjected to a static loading test because of pile damage during installation. The steel pile was not statically tested due to misalignment between the pile head and the reaction system.

This paper presents discussions on the low strain and high strain dynamic testing and static loading test methods along with comparative evaluation of results. Ultimate pile static resistances determined from dynamic tests were within 8% of those measured by full scale static loading tests. Additionally, predicted and measured pile head load-movement and pile shaft forces at ultimate resistance relationships also agreed well. Structural damage in the shaft of one of the driven concrete piles was evident in records of both high and low strain dynamic tests.

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Introduction

Foundation settlements of more than 300 mm and corresponding structural distress required replacement of the swing span bridge carrying traffic along Highway 802 over Battery Creek in the eastern coastal region of South Carolina, USA. Settlements occurred when the supporting deep foundations (octagonal prestressed concrete piles with steel H-section extensions) punched through the bearing stratum which consisted of a relatively thin limestone caprock. The new replacement structure will be a segmental concrete bridge with a total length of approximately 915 m (including a 43 m long center span) and a width of 26.2 m. Navigational clearance dimensions will be 13.7 m vertical and 18.3 m horizontal near the middle of the bridge.

A pile installation and testing program was undertaken at the project location to determine relevant foundation design parameters and evaluate the performance of various pile types and sizes. The test site location was chosen for its accessibility and convenience in order to minimize the cost of the testing program. A total of five piles of various types and sizes were evaluated. Additionally, four shafts were installed as reaction piles. All test piles were dynamically tested and three were load tested statically. The specific objectives of this preconstruction testing program included: determination of pile installation characteristics, verifying minimum factors of safety against failure, determining pile load-movement behavior, analyzing pile-soil load interaction behavior, and to compare several different pile types to determine which would be the most suitable to satisfy project requirements. This paper presents discussions on pile installations and testing procedures and results.

The original work for this project was done in the English units, soft conversions were used to convert values to the SI units for this paper.

Subsurface Conditions

Twenty six soil test borings ranging in depth between 10 and 30 m were drilled to explore the subsurface conditions along the proposed bridge alignment. The pile driving and testing site was located on a small peninsula near an approach to the existing bridge. This location was chosen for its convenience allowing for economic accommodation of the test program and its required equipment.

One soil test boring was drilled to a depth of 27.4 m within the confines of the test site. The boring initially encountered a 1.8 m thick layer of firm sandy clay which was underlain by a 3.3 m thick layer of loose silty sand followed by a 1.8 m thick layer of soft plastic clay. At a depth of 7.0 m, a 2.4 m thick layer of loose sand was encountered. The Hawthorn

Formation (very loose to loose very silty sand) was found at a depth of 9.5 m. A thin (approximately 0.6 m) layer of hard, very sandy limestone ("caprock") was encountered at approximately 12.5 m depth. From a continuous coring run of the caprock only one 50 mm long sample was recovered. Due to the insufficient size of this sample, the compressive strength of the caprock could not be determined. A fine to coarse, calcareous sand displaying some cementation was encountered below the caprock and extended to the boring termination depth of 27.4 m. A simplified log of the boring showing soil strata including Standard Penetration Test N-values is presented in Figure 1.

Test Piles

Three prestressed concrete piles, one steel H-pile and one drilled shaft were studied. Additionally, four drilled shafts (890 mm in diameter) ranging in lengths between 27 and 32 m were installed as reaction piles for the static loading tests. The piles tested are referred to as Test Piles A, B, C, D and E. Test pile A was a steel HP 14x73 section (area = 138 cm^2) with a length of 24.4 m. Test Pile B was a 457 mm square (area = 2090 cm^2) prestressed concrete pile with a length of 19.5 m. Test Piles C and D were 610 mm octagonal (area = 3077 cm^2) prestressed concrete sections with lengths of 19.5 and 24.1 m, respectively. The 890 mm diameter, 17.7 m long drilled shaft was Test Pile E. Each of the precast concrete piles was cast with a steel "stinger" which extended 0.76 m beyond the tip of the pile. An HP 10x57 section was embedded 1.8 m into the center of the square concrete pile and HP 12x74 sections were embedded 2.3 m into the octagonal piles.

Vibrating wire strain gages were installed at various points in each of the test piles for measurement of pile strains at different sections during static loading tests. Sixteen Geokon model VSM 4000 weldable gages were welded to the web of the H-pile. Gages and leads were protected by steel angles welded on both sides of the web. Geokon Model VCE 4200 gages were tied in the center of the concrete precast piles before casting, and tied to the reinforcing cage of the drilled shaft. Geokon VSM 4000 gages were used to instrument the H-pile extensions. Gages were located at five to seven locations along pile lengths corresponding to the different soil layers. Two gages were used at each location.

Installing the driven piles was accomplished with a Vulcan 520 single acting air hammer. This particular hammer had a ram weight of 89 kN and was fitted with a slide bar allowing for a 0.91 and 1.52 m strokes (corresponding rated energies of 81.6 and 136.0 kJ, respectively). Sheets of plywood with thicknesses ranging between 150 and 300 mm were used as pile top cushions when driving the concrete piles.

A 610 mm diameter auger was used to form a 9 m deep hole before inserting and driving Test Pile A (i.e., H-pile). The pile toe advanced to a depth of 13.4 m under the weight of the pile and hammer. The pile was driven with a 0.91 m hammer stroke for 30 cm before reaching refusal blow counts on top of the caprock. The pile was extracted and damage (slightly bent flanges) was evident both at the pile head and toe. Additionally, during extraction the "choker" collapsed the flanges over a one meter length of the pile within the upper seven meters. The top 0.5 m of the pile was removed and the collapsed flanges were reinforced with steel plates. A spud weighing 107 kN was dropped thirty times from a height of 12 m to break through the caprock. Test Pile A was again inserted in the hole and driving continued until pile top flanges were again bent. Another 0.75 m of the pile top was removed and pile driving continued (still with the short stroke) to a final depth of 21.6 m and driving resistance of 8 blows/ 0.25 m. During the installation process, the pile head rotated approximately 15 degrees and moved approximately 30 cm horizontally.

Prior to driving each of the concrete piles, a 610 mm diameter auger was used to form a hole 12 m deep and the spud was used to break through the caprock; a process similar to that of driving the steel pile. Test Pile B was driven to a depth of 19.2 m. End of driving resistance suddenly dropped from 33 blows/150 mm to 5 blows /250 mm. Dynamic tests showed that this pile was indeed broken.

Test Piles C and D were installed with no problems. Pile C was driven to a final penetration of 18.9 m and a resistance of 6 blows/25 mm using a 0.91 m hammer stroke. Towards the end of driving of Test Pile D, the hammer was operated with the short stroke, and the final pile penetration was 23.5 m with a resistance of 11 blows/102 mm.

The installation process for all drilled shafts (i.e., Test Pile E and reaction shafts) was similar. To retain the unstable overburden soils, a 914 mm outside diameter steel casing with a 12.5 mm wall thickness was first installed with a vibratory hammer to a depth of approximately 13.7 m which corresponds to the location of the caprock layer. An 890 mm diameter auger was then used to excavate each shaft to the desired depth. The excavation proceeded in the dry until the caprock was reached at which point the casing was filled with water. Two 50-lb bags of Flourigel were typically added shortly thereafter to maintain side-wall stability within the calcareous sand stratum. Following excavation, a clean-out bucket was used to remove the cuttings before the reinforcing cage was lowered into place and concrete placed by tremie method. As soon as the hole was filled with concrete, the casing was removed with the vibratory hammer; except for the test shaft where the casing was left in place.

Dynamic Pile Testing

All driven piles were dynamically tested during installation with a Pile Driving Analyzer™ (PDA) according to the Case Method. Some piles were also dynamically monitored during restrike. Subsequent dynamic data analysis was performed according to the CAPWAP® Method. All concrete piles, including the drilled shaft, were additionally tested using the Pile Integrity Tester™ (P.I.T.) dynamic method for structural integrity evaluation.

Since dynamic pile testing during installation is performed under the pile driving hammer blows, this type of test is commonly referred to as a "high strain" test. Testing is often performed for the purposes of evaluating hammer and driving system performance (Likins and Rausche 1988), assessment of pile driving stresses and structural integrity (Hussein and Rausche 1991), and pile driving resistance and static bearing capacity (Rausche et al. 1985). Dynamic measurements of strain and acceleration under hammer impacts are the basis for modern dynamic pile testing. The equipment consists of two each reusable strain transducers and accelerometers, bolted at opposite sides of the pile approximately one meter below its head, and a Pile driving Analyzer. The PDA is a state-of-the-art, user friendly, field digital computer. Basically, it applies Case Method equations to pile force and velocity data in real time between hammer blows after providing signal conditioning, amplification, filtering, calibration to measured signals and data quality assessment. High strain dynamic pile testing has been incorporated into many standards and specifications (ASTM 1989) and is now routine procedure in modern deep foundation practice worldwide (Goble and Hussein 1994).

The PDA computes some 40 different dynamic variables according to the case method after each hammer blow. The most interesting values are: maximum energy transferred to the pile, maximum dynamic pile compressive and tensile stresses, a structural integrity assessment factor, pile driving resistance and static bearing capacity. dynamic testing is also performed during pile restrikes to evaluate time dependent soil resistance changes and their effect on pile load carrying capacity.

Dynamic data obtained in the field can further be analyzed according to the CAse Pile Wave Analysis Program (CAPWAP) for a more comprehensive understanding of the soil and pile behavior during pile driving and under static loading conditions (Rausche et al. 1994). The analysis is done in an interactive environment using measured pile data and wave equation type analysis as a system identification process employing signal matching techniques. Results from a CAPWAP analysis include static pile capacity, soil resistance distribution along pile shaft and under toe, soil damping and quake (maximum elastic deformation) values, forces along pile length at ultimate resistance and a simulated static loading test relating pile top and toe load-movement relationships.

Low strain dynamic tests are performed on concrete piles (driven or cast-in-place) for the main purpose of assessing shaft structural integrity. This type of test requires minimal pile preparation, and is simply done by affixing an accelerometer to the pile top and impacting the pile head with a small hand held hammer. The measured data of pile top motion is processed, analyzed and digitally stored by a dedicated system called the Pile Integrity Tester (P.I.T.). The premise of this method is that changes in pile impedance (elastic modulus times area divided by stress wave speed) and soil resistance produce predictable wave reflections that can be measured at the pile head. Since the pile impedance includes both material strength and geometric parameters, a reduction in impedance then represents a weakening in the pile shaft. Data evaluation may be done by visual inspection of the records in either time or frequency domain, or by more rigorous dynamic analysis (Rausche et al. 1994). Testing is usually performed shortly after pile installation so that deficient piles may be identified and corrective measures taken before construction of the superstructure. Application of this method was recently expanded to test piles under existing structures for determination of unknown pile lengths (Hussein et al. 1992).

Static Loading Tests

Two of the driven concrete piles (Test Piles C and D) were statically load tested to failure; the drilled shaft (Test Pile E) was tested up to the capacity of the loading system. Test Pile A was not load tested due to misalignment between the pile head and the reaction system and Test pile B also was not statically tested due to damage in the pile shaft. Figure 2 shows test and reaction piles relative locations.

All static loading tests were performed in general accordance with ASTM D-1193 quick load procedures. Load was applied in increments of approximately 15% of the anticipated ultimate value for each case. The load was held (typically for three minutes) to take two readings of four dial gages measuring vertical movement and two dial gages measuring lateral movement, one reading of all strain gages and one level/ruler reading. Loads were applied until the pile plunged or the capacity of the loading ram was reached. Loads were applied via an 8900 kN hydraulic ram and were measured with a 7120 kN electronic load cell. When the applied load exceeded 7120 kN, which only happened during the drilled shaft load test, the hydraulic pressure and a previous jack calibration were used to estimate the actual load. In each case, reaction for test loads was provided by means of a steel wide beam and two reaction drilled shafts.

Immediately before each test, all strain gages were read to obtain a reference "zero' value. All subsequent readings were subtracted from the initial readings to obtain pile strain values. Since there were usually two strain gages at each location in each pile, data from both gages were averaged. Readings from the top gages near the pile heads and load cell were used to compute pile elastic modulus. This computed value was then used at all strain gage locations. It was assumed that the gages at all locations were under plane strain conditions. Consequently, for concrete piles with steel stingers, the strain measured in the concrete was assumed equal to that in the steel (and the opposite when the weldable strain gages data was used). Therefore, at the pile toe, the total load was the sum of the values in the concrete and steel.

For the drilled shaft, loads were calculated for the concrete and reinforcing steel, neglecting the steel casing. If the steel casing was included, then the load calculated from the measured strain would be higher than the applied load. This finding indicates that the plane strain assumption is not valid for the drilled shaft/casing combination.

Discussion of Testing Results

During the first attempt to drive the H-pile refusal driving resistance was met at the top of the limestone at which point the PDA computed a pile capacity of 2670 kN. Maximum pile compressive stresses at the transducers locations was 193 MPa. As evident by the yielding of the flanges, stresses at both pile ends were higher probably due to non-uniform hammer impacts. Compression stresses were similar during the second driving attempt. At a penetration of 19 m, the pile encountered its maximum resistance and capacity (1600 kN according to CAPWAP). The pile capacity decreased with increasing pile penetration until an end of driving capacity of 445 kN was computed at a final pile penetration of 21.6 m. These capacities values were higher than anticipated for this type of pile. The pile head moved laterally during driving making it impossible to perform a useful static loading test under safe conditions.

Test Pile B (457 mm square concrete pile) was broken during installation. Dynamic data obtained during pile driving indicated pile damage 17.7 m below the pile head when the pile penetration was approximately 16.5 m. This damage location corresponds to the location of the embedded end of the steel "stinger' in the bottom of the pile. It is worth mentioning that prior to driving this pile, it was observed that the stinger was not straight or parallel with the pile's longitudinal axis. During driving, maximum pile compressive and tensile stresses reached 14 and 2 MPa, respectively. Due to site constraints, the PDA gages had to removed from the pile and the pile was driven an additional one meter without dynamic monitoring. Apparently, as evidenced by the P.I.T. test results performed after pile installation the pile was broken again at a location 11.6 m below its head during its last meter of penetration. P.I.T. test results are presented in Figure 3 in the form of pile top velocity record plotted as a function of length. Time to length conversion was done by using a material stress wave speed of 4000 m/s; an exponential amplification factor was applied to the measured data in order to compensate for pile and soil damping effects. This pile was not statically load tested.

During the installation of Test Pile C (short octagonal concrete pile), maximum pile compression and tensile driving stresses were 17.9 and 2.6 MPa, respectively. This pile was driven to a final penetration of 18.6 m and an end of driving resistance of 240 bl/m and a PDA computed capacity of 4940 kN. Two days after installation, the pile was subjected to a static loading test. Testing results in the form of a pile head load-movement curve are included in Figure 4. The pile "failed" under a static load of 4895 kN (Davisson's criteria), but supported an ultimate load of 5340 kN. CAPWAP analysis performed with dynamic data obtained during restrike one day after the static loading test indicated a static pile capacity of 5010 kN, which is just 2.5% higher than "Davisson's" failure load. The low strain dynamic test result performed the same day of restrike is included in Figure 3. Pile-soil load transfer curves were computed from measured pile strains at various pile cross sections. These results are presented in Figure 6 and show that the pile developed very little friction above the caprock and that approximately 60% of total pile capacity was in end bearing. The total pile head movement at half the failure load (i.e., 2447 kN) was approximately 8 mm.

Test Pile D (long octagonal concrete pile) was driven to a penetration of approximately 23.5 m and was statically load tested two days after end of installation. During driving, maximum pile compressive stresses reached 20 MPa and tension stresses were generally below 6 MPa. The pile "failed" under a static load of 2270 kN (Davisson's criteria), but supported an ultimate of 2447 kN. The CAPWAP capacity computed from a restrike blow one day after the static test indicated a pile capacity of 2447 kN, only 8% higher than the Davisson's failure value. Figure 4 includes the pile top load movement curve from the static loading test. CAPWAP analysis plotted results are presented in Figure 5. Load transfer curves developed from pile strain measurements during the static loading test are presented in Figure 6. Testing indicates that the soils above the caprock provided virtually no resistance and that 65% of the pile capacity was due to end bearing. The "gross" pile head movement at half the failure load (i.e., 1135 kN) was approximately 5 mm. Results of the P.I.T. test performed after pile installation are included in Figure 3.

During the installation of Test Piles C and D, maximum transferred energies to the piles (at the transducers locations) averaged 45 kJ while the hammer operated at a 0.91 m stroke, which translates to a transfer ratio (transferred divided by potential) of 58 percent. Dynamic data from both PDA and P.I.T. did not indicate any damage in the shafts of these piles.

The drilled shaft (Test Pile E) was statically load tested 19 days after its installation. Prior to the load test, the pile was tested with the P.I.T. to verify its structural integrity. Low strain test results are included in Figure 3 and static load test result (load-movement curve) in Figure 4. The pile was loaded to the maximum capacity of the load cell (7120 kN) and then unloaded. The load cell was then removed and the shaft was loaded to 8900 kN (as determined by the hydraulic pressure gage on the jack). The pile head movement under this load was only 8 mm, which is approximately equal to the theoretical pile elastic shortening. The pile did not appear to be close to failure, although it is not known if it was on the verge of plunging. As mentioned earlier, analyzing data from the embedded strain gages was a complex process. Three sets of calculated load-transfer curves are possible depending on the assumptions employed. The gages located in the pile within the length of the steel casing usually gave erratic results while those located below the casing and caprock produced more reasonable and stable results. Regardless of how the load was being carried above and within the caprock, less than 15% of the applied load was transferred to below the caprock. Pile-soil load transfer curves obtained from computed forces in the concrete, and ignoring the steel casing, are included in Figure 6.

Summary

A pile driving and testing program was undertaken at a proposed bridge replacement project site. Three prestressed concrete piles, one steel H-pile and one drilled shaft were studied. Evaluations included observations during pile installation, dynamic high- and low-strain testing and static loading tests. One of the concrete piles was damaged during driving and was not load tested. The steel pile also was not statically tested due to pile head lateral movement during driving. The other two driven concrete piles were load tested to failure. The drilled shaft was loaded to the capacity of the loading system and did not fail. Data analysis indicated that dynamic predictions of static pile capacities, pile head load-movement and pile-soil load transfer relationships agreed well with those obtained from static loading tests. Figure 7 presents plots of actual (i.e., static loading test) and predicted (i.e., CAPWAP analysis) pile head load-movement relationship for Test Pile D. Static loading test determined and CAPWAP analysis simulated pile-soil load transfer curves for Test Pile C are presented in Figure 8. Results of the program were instrumental in designing a proper and economical foundation system.

Acknowledgments

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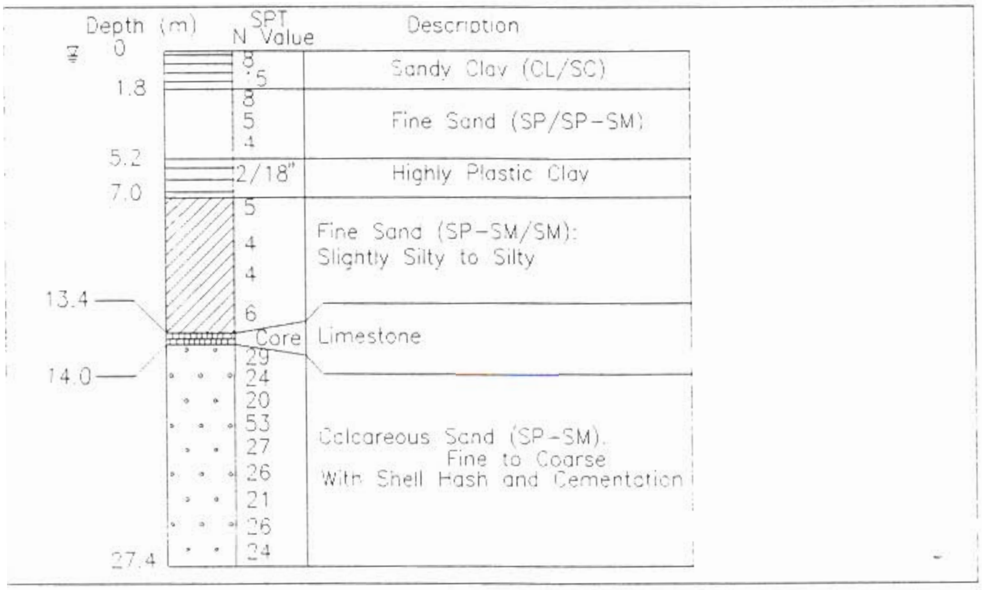

FIGURE 1: SOIL BORING

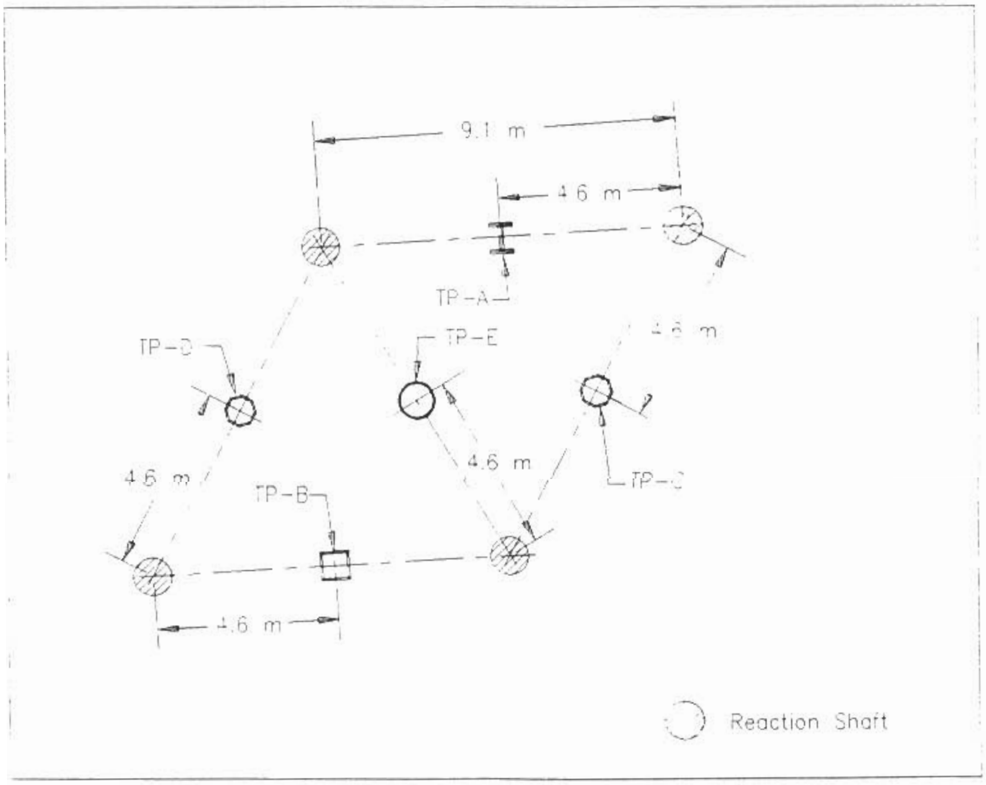

FIGURE 2: TEST AND REACTION PILE LOCATIONS

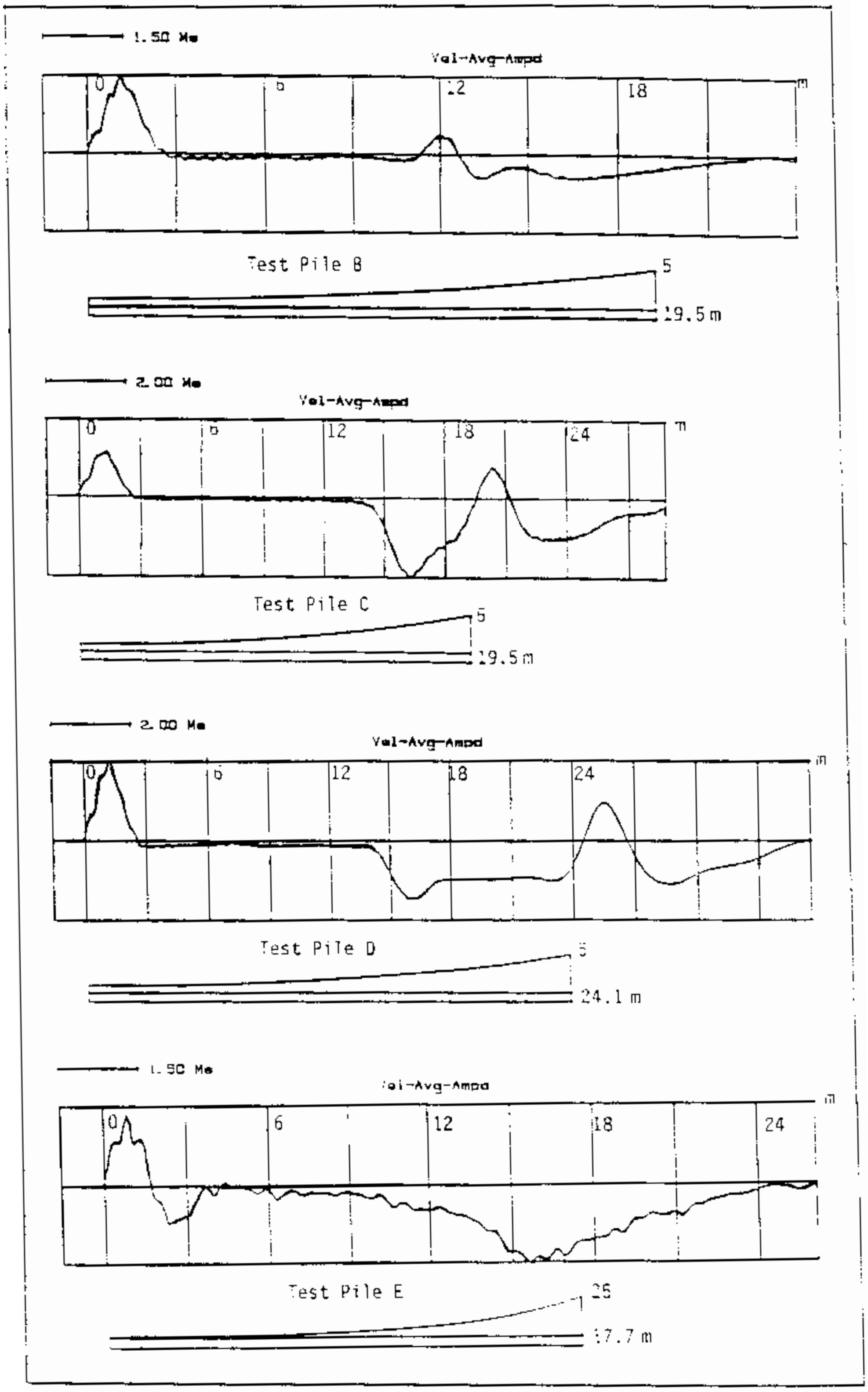

Figure 3: Low strain dynamic testing (P.I.T.) results.

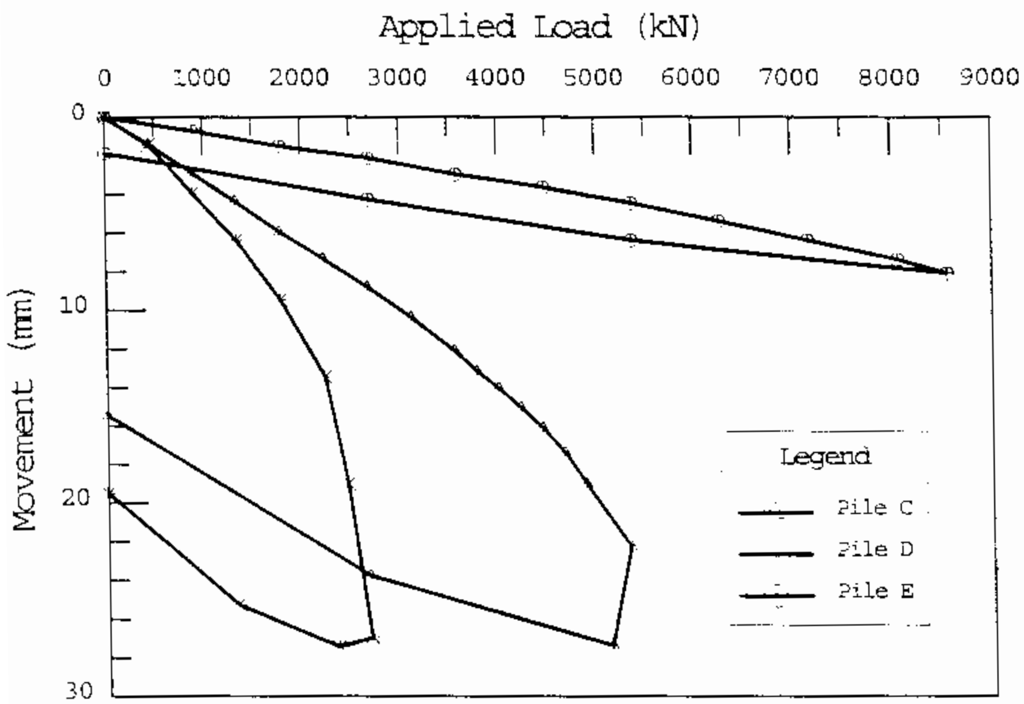

Figure 4: Pile head load-movement relationships from static loading tests

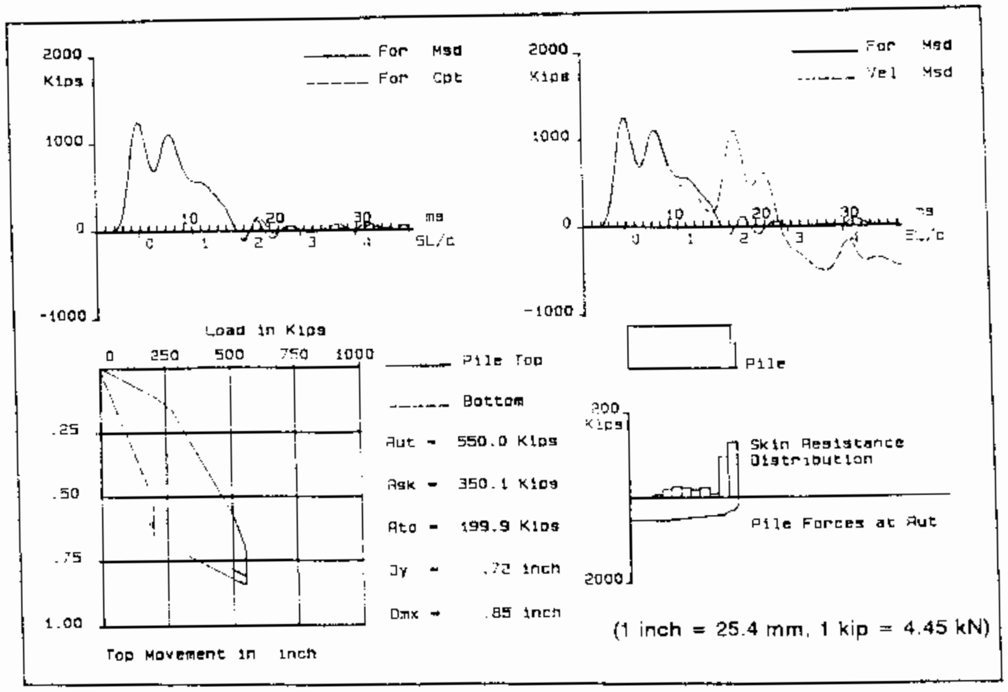

Figure 5: CAPWAP analysis results - Test Pile D.

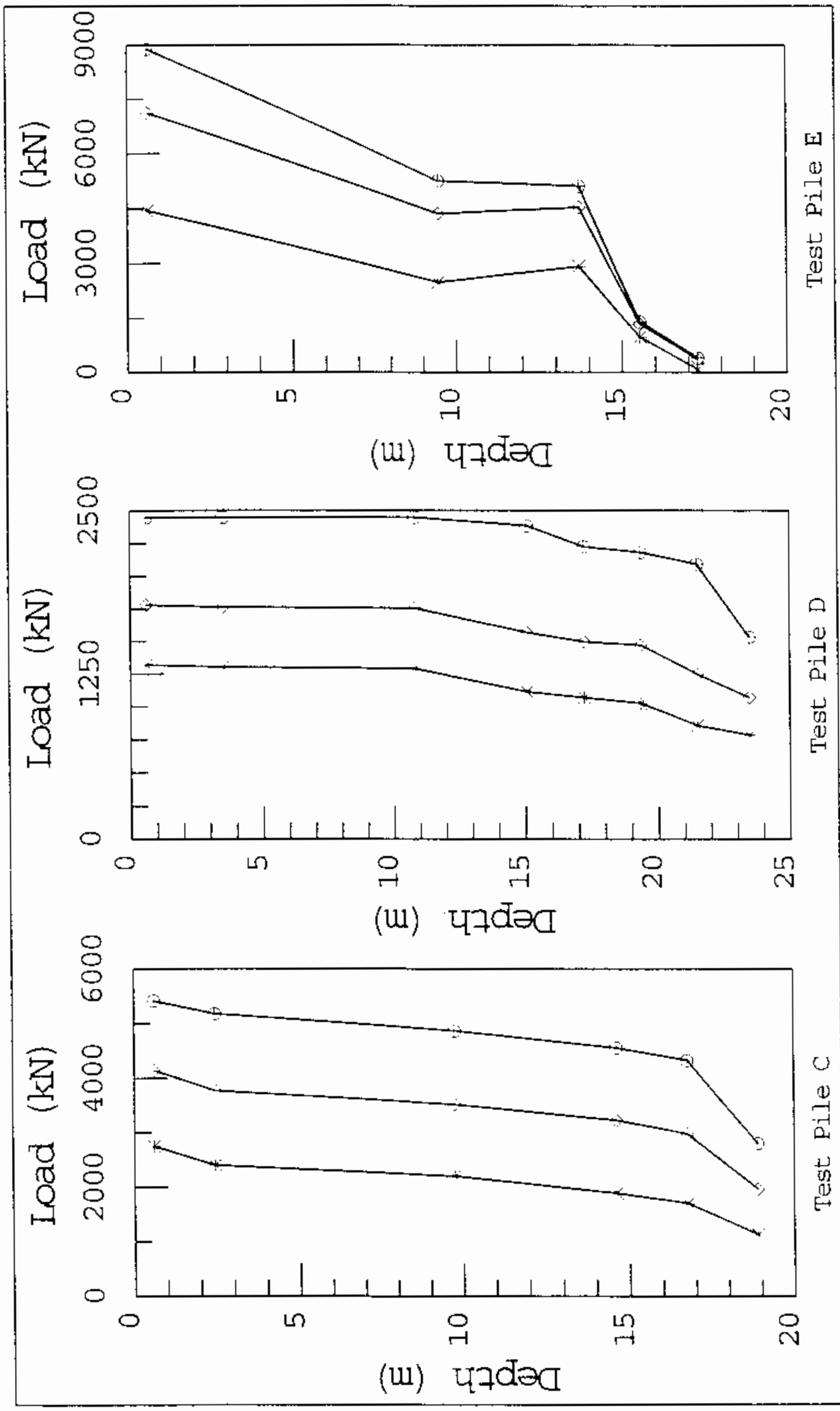

Figure 6: Load transfer curves

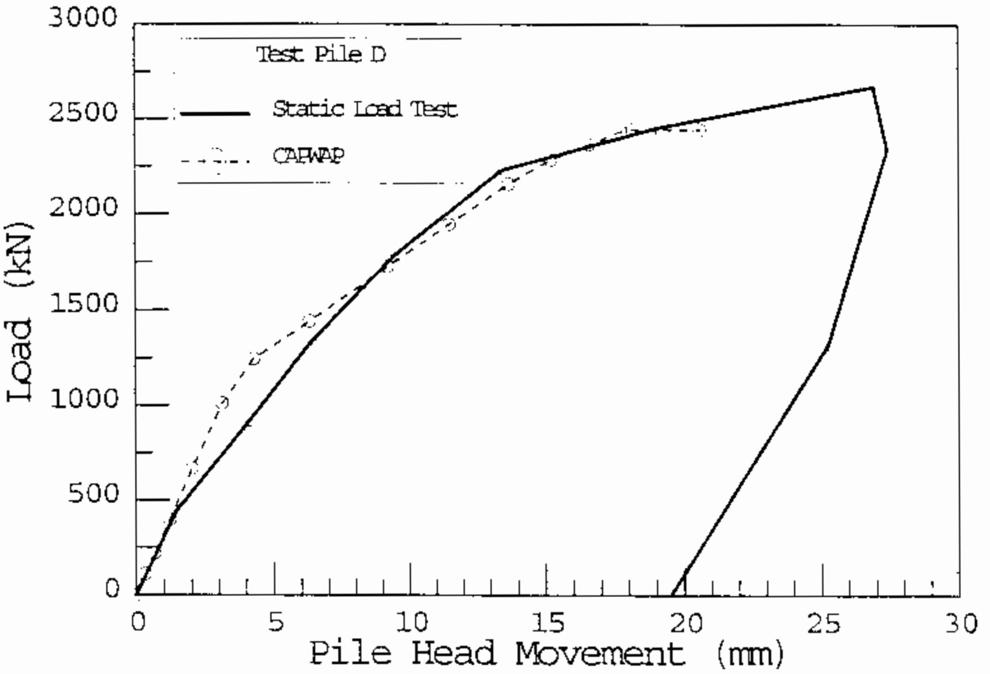

Figure 7: Measured and simulated pile head load - movement curves

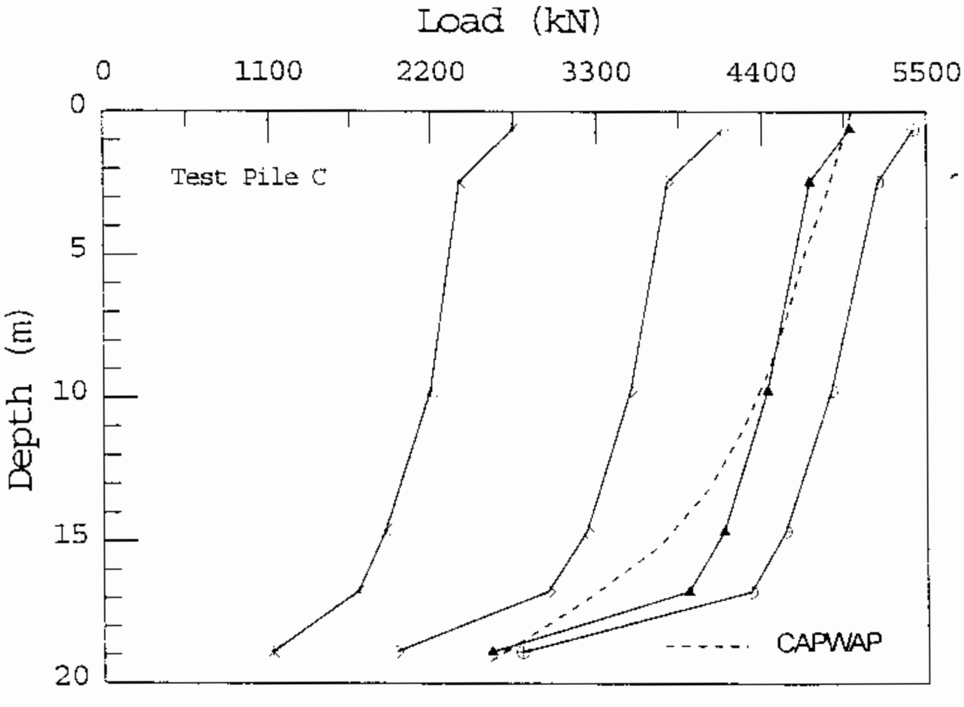

Figure 8: Load transfer curves including CAPWAP simulated curve