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UNDERPINNING USING LONG MINIPILES IN COMPRESSIBLE SOILS

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UNDERPINNING USING LONG MINIPILES IN COMPRESSIBLE SOILS

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ABSTRACT. A minipile system was employed in underpinning a historical structure in Florida. The existing structure is founded on shallow foundations. Subsurface conditions at the project site consist of deep organic deposits interbedded with loose sand and silty sand. A minipile foundation system was deemed the most appropriate underpinning corrective measure for the already badly cracked structure. A total of 68 minipiles, 3.5 inch OD and 0.25 inch wall thickness closed-ended concrete-filled steel pipes, were installed to depths varying from 55 to 175 feet. Installation was accomplished with a pile driving hammer having a rated energy of 1 kip-ft. Highest required pile design load was 15 kips. Conventional static and wave equation analyses were performed to establish preliminary pile lengths, to assess pile drivability and establish driving resistance criteria. Dynamic pile testing using the Pile Driving Analyzer was conducted on six of the production piles. This paper describes geotechnical design considerations, installation procedures, and dynamic field monitoring. This case represents unique applications of pile dynamic testing and such slender, very long minipiles in compressible soils.

INTRODUCTION

In the early 1920's, a cypress bayhead was hydraulically filled with lake bed sediments over the existing surficial organic silts. A bandshell structure was constructed in 1929 on the hydraulic fill. Since that time, the main building has suffered noticeable total and differential settlements causing many cracks. Figure 1 presents a view of the building front.

In an effort to preserve this historic structure, underpinning was used to limit future additional settlements. Underpinning was accomplished using minipiles, consisting of closed-ended concrete-filled small diameter steel pipes. A schematic showing the locations of the piles and the connecting load-bearing steel beams is presented in Figure 2.

An engineering study was undertaken to evaluate the applicability of long minipiles in such compressible soil conditions for underpinning the structure. Conventional static analysis using information obtained from subsurface explorations and wave equation dynamic pile analysis were performed to establish preliminary pile lengths, to assess pile drivability and establish driving resistance criteria.

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Figure 1: View of the building front

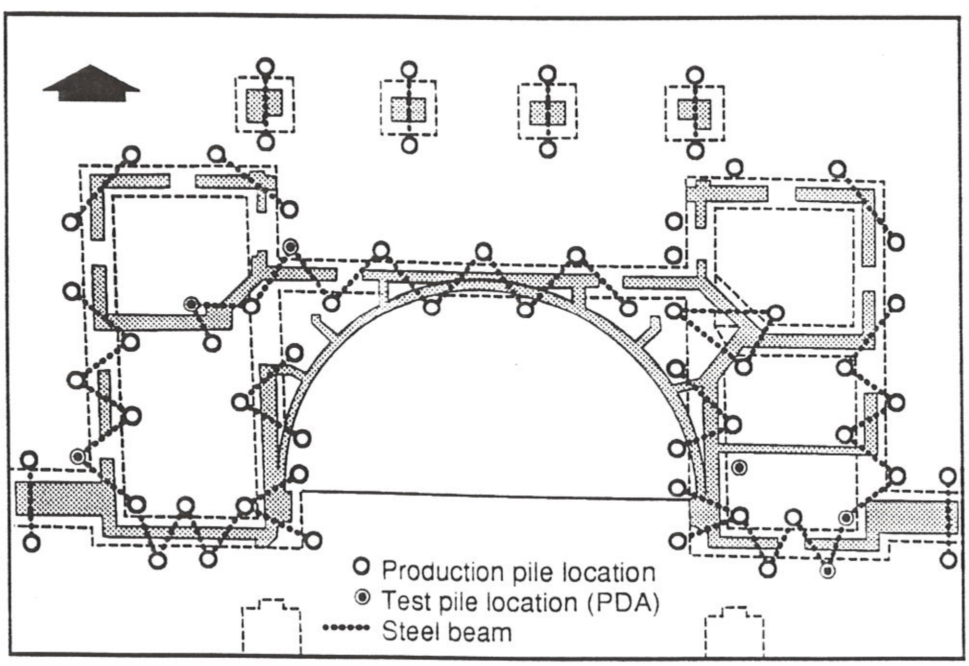


Figure 2: Pile and beam Locations

Dynamic pile testing using the Pile Driving Analyzer (PDA) and data analysis according to the CAPWAP method were performed on six of the production piles during installation in order to verify analysis assumptions and static pile capacities.

This paper presents discussions on the design, installation and testing of the minipile system used. Since all original project data is in English units, an SI conversion table is included in the Appendix.

SUBSURFACE INVESTIGATIONS

Subsoil conditions were investigated by four (4) Standard Penetration Test (SPT) borings, a continuous flight auger boring and three (3) electronic cone soundings (CPT). The borings were performed to depths of 45 to 65 ft, while the cone soundings were advanced to depths of 75 to 129 ft below the ground surface. A layout of the borings and soundings locations relative to the structure under consideration is shown in Figure 3. Results of the CPT soundings are presented in Figure 4 and SPT borings information are shown in Figure 5. A general soil profile is shown in Figure 6.

Information obtained from the various subsurface investigation techniques indicated a highly variable subsoil conditions consisting of mainly loose to very loose fine sands and silty fine sands in the upper 30 ft. In two of the borings and in the cone soundings, a thin peat layer was observed at a depth of approximately 12 ft. In two other borings a layer of soft peat 7 to 12 ft thick was observed at a depth of 30 feet below grade. Below these loose and organic soils, medium dense fine sands and silty fine sands were observed to a depth of approximately 65 ft. Below this depth, only loose fine sands and silty fine sands were observed. Generalized soil profiles extending to a depth of approximately 65 ft are presented in Figure 6. Pile penetrations ranged between 55 to 175 ft, and in many cases extending to below the depth of soil investigations.

PILE DESIGN

The highest loaded piles had design loads of 15 kips per pile. Preliminary pile design was based on conventional static analysis. The equations used for determining ultimate static load capacity Q_u are as follows:

$$Q_u = Q_s + Q_p \qquad (1)$$
$$Q_s = \Sigma[(\Delta A_s)(K\sigma'_v \tan\delta)] \quad \text{for cohesionless soils, and} \qquad (2a)$$
$$Q_s = \alpha c_u \quad \text{for cohesive soils} \qquad (2b)$$
$$Q_p = qN_qA_p \qquad (3)$$

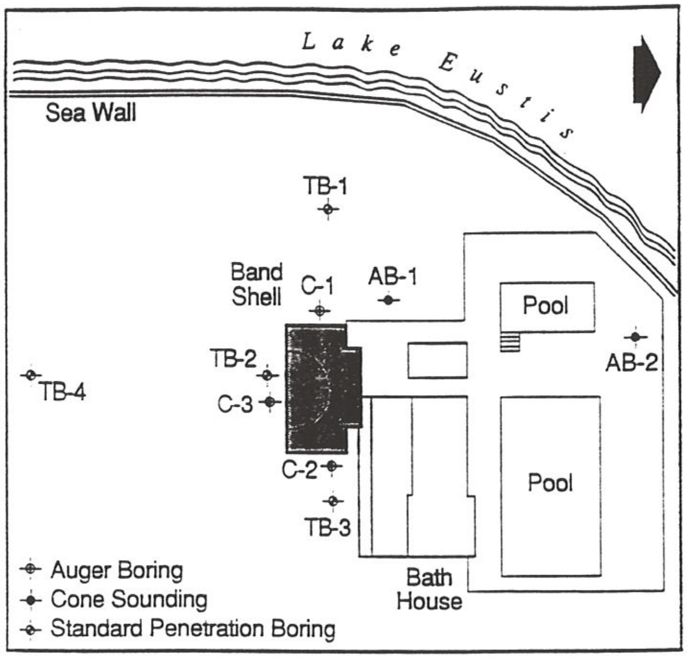
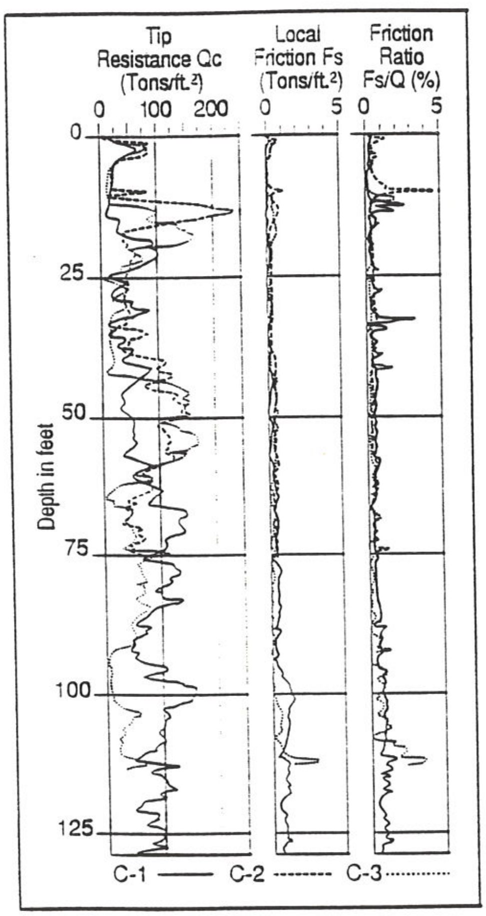


Figure 3: Layout of SPT borings and CPT soundings

Figure 4: CPT soundings results



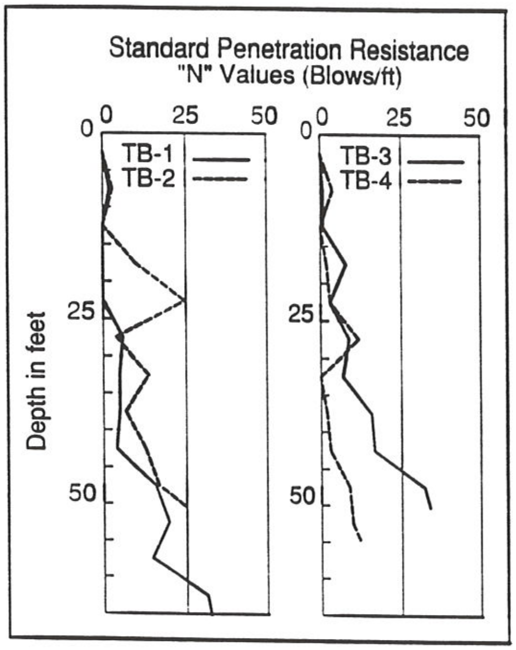


Figure 5: SPT borings results

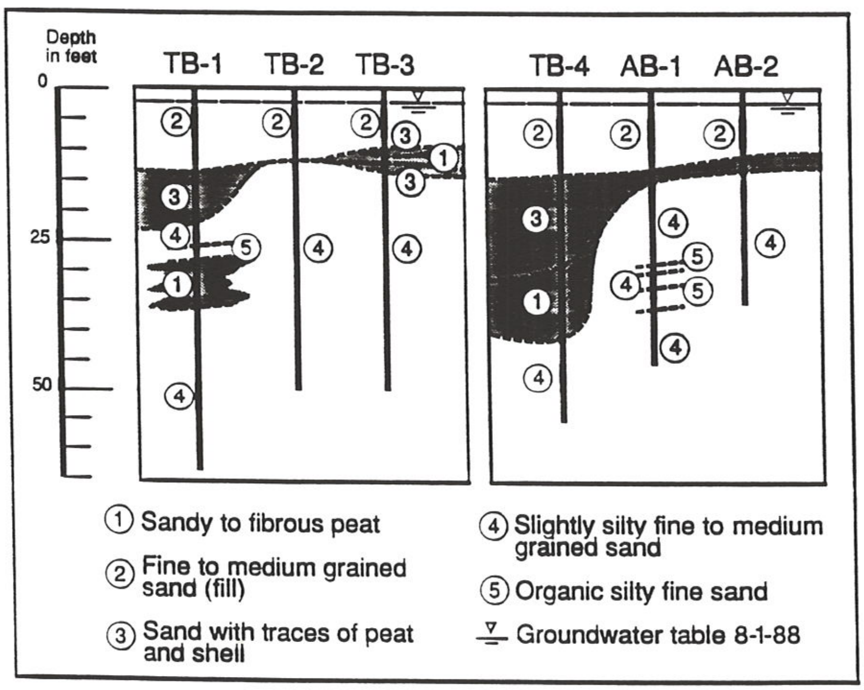


Figure 6: Generalized subsurface profiles

where Q_s and Q_p are the load carrying capacities due to friction and end bearing, respectively. K is the coefficient of lateral earth pressure, taken to be 1.8 times the coefficient of earth pressure at-rest, K_o; σ'_v is the effective vertical stress, limited to the vertical stress at a depth of 20 pile diameters; δ is the pile-soil interface friction angle, taken as 0.8ϕ; α is the adhesion factor, taken to be 1 for $c_u < 500$ psf where c_u is the undrained shear strength and ΔA_s is an incremental surface area of the pile shaft.

Estimated soil parameters obtained from subsurface investigations were used in the above equations for determination of pile capacities for various pile penetration depths. Calculations based on assumed soil conditions and behavior indicated that pile penetrations of at least 80 ft were needed to develop pile capacity with allowance for downdrag loads. Due to the highly variable and unpredictable subsoils, however, it was not possible to determine with confidence final pile lengths required. The existence of a relatively hard and thick layer, for example, would stop a pile from penetrating further and supply sufficient support for the imposed loads. The use of driven steel minipiles was deemed most appropriate at this site due to the minimal effects that pile driving would have on the already badly cracked structure and the ease of extending pile length during installation.

Preconstruction wave equation analyses were performed using the GRLWEAP program for an assessment of pile drivability and for preliminary estimates of required driving resistance using the proposed MKT Model 5 hammer. Various combinations of pile lengths and soil conditions were evaluated. Analyses showed that for end of initial driving resistance between 100 and 200 blows per foot (BPF), pile capacities would be between 15 and 20 kips. For driving resistances between 200 and 300 BPF, static pile capacities were expected to vary from 20 to 28 kips, and for blow counts above 300 BPF, pile capacities would be above 30 kips. Maximum pile driving compressive stresses were computed to be between 16 and 17 ksi. Actual pile capacities driven to the above blow counts were expected to be somewhat higher than those computed and listed above due to: (a) neglect of pile residual stresses in the analyses, and (b) soil strength increase with time after pile installations.

PILE INSTALLATION

All piles used were "mill-reject," small diameter steel pipes with 3.5 inches OD and a wall thickness of 0.25 inch (nominal cross sectional area of 2.56 square inches). Pipes were delivered to the site in sections approximately 20 ft in length, field welding was therefore performed during construction for extending pile lengths as needed. The piles were driven closed-ended and filled with concrete after installation. Pile driving was accomplished with an MKT Model 5 double-acting air hammer. This particular hammer model has a ram weight of 0.2 kips, a rated energy of 1.0 kip-ft per blow and operated at 300 blows per minute. The driving system and pile were guided with a set of swinging leads. The close proximity of the pile locations to the existing structure dictated great care in

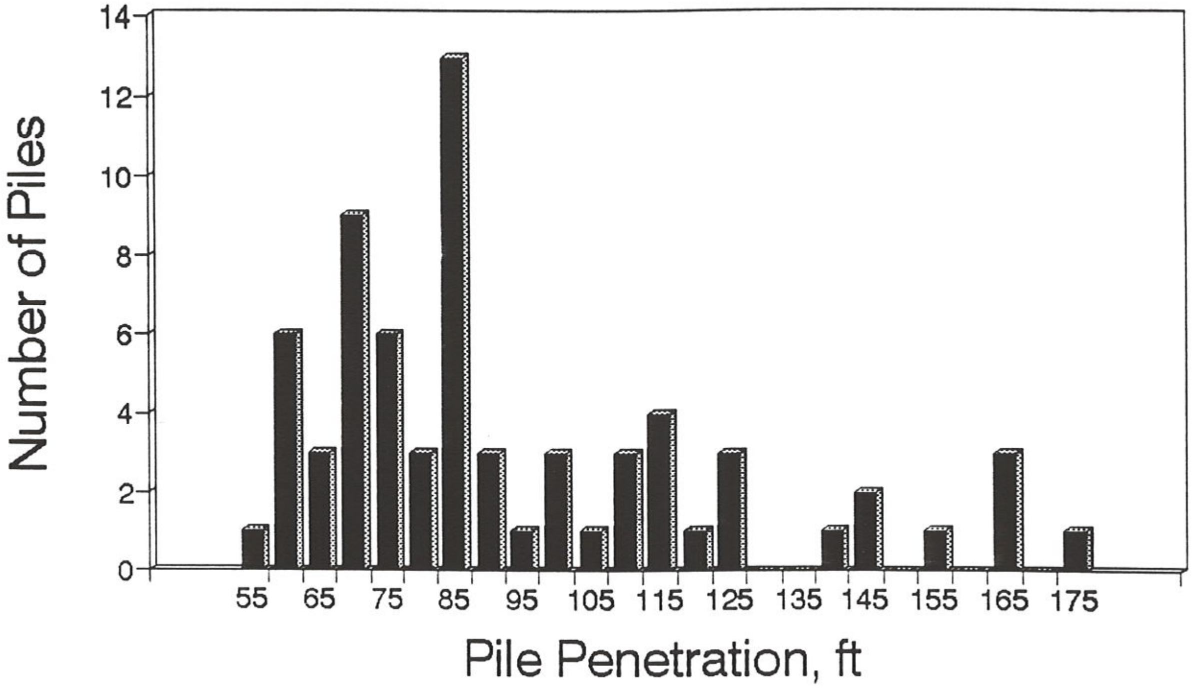


Figure 7: Distribution of pile penetrations

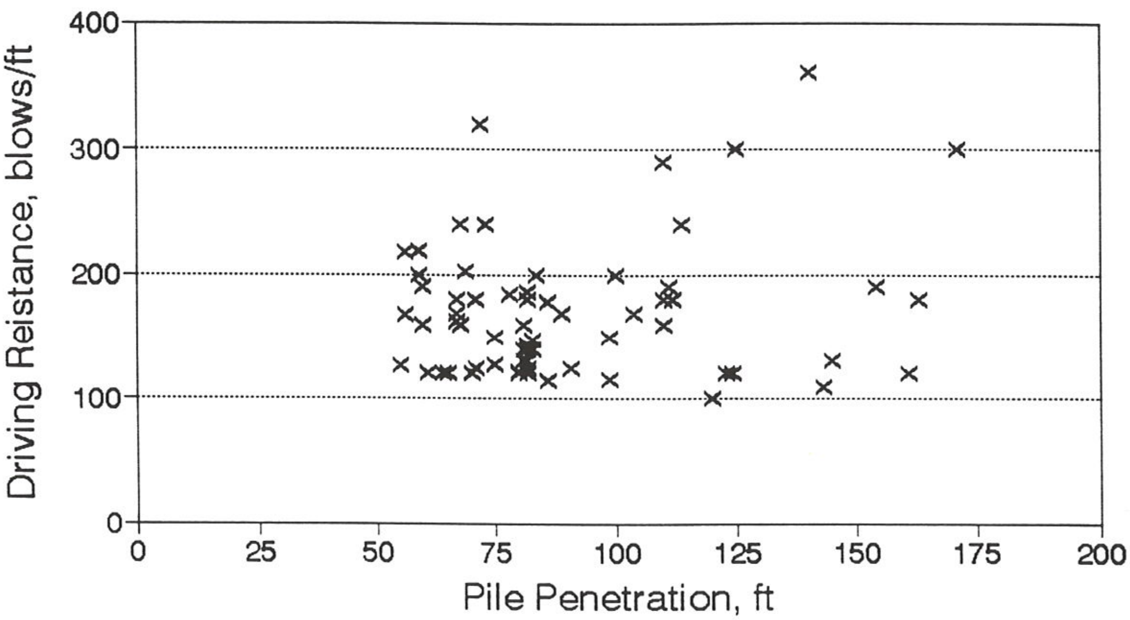


Figure 8: Driving blow counts versus pile penetrations

handling the driving equipment and in positioning each pile.

Pile penetrations ranged between 55 and 175 ft. About 70% of the piles had penetrations between 60 and 100 ft and 12% had penetrations of more than 125 ft. Figure 7 presents a histogram showing number of piles for each penetration depth using a 5 ft length increment.

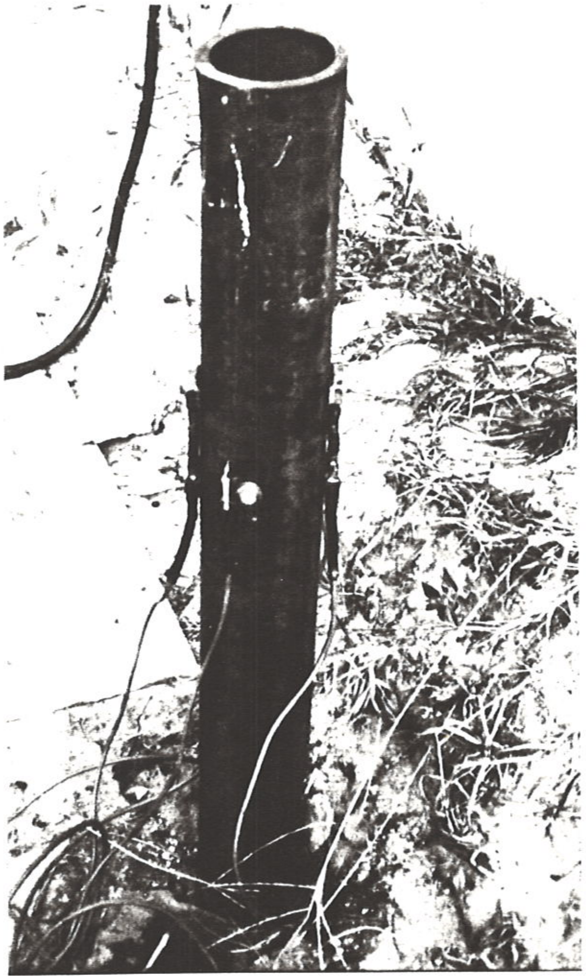
In all cases, pile driving resistances were above 100 blows per foot (BPF). Approximately 85% of the piles had blow counts between 100 and 200 BPF and 13% had driving resistance between 200 and 300 BPF. Two piles were driven to more than 300 BPF. Due to the highly variable subsurface conditions, driving resistance was independent of pile penetration depths. Figure 8 presents pile driving resistance and penetration for each pile. Of the two piles driven to more than 300 BPF, one was stopped at a penetration of approximately 72 ft in the vicinity of C-1 and AB-1 and the other (in the vicinity of TB-2 and C-1) penetrated twice as deep. As indicated by the figure, it is clearly seen that the same driving resistance was encountered at different depths by different piles, and at the same toe penetration, different piles encountered varying driving resistance.

DYNAMIC PILE TESTING

Dynamic pile testing and analysis are routine procedures in modern deep foundation practice [1]. Testing is often performed for the purposes of evaluating hammer and driving system performance [2], assessment of pile driving stresses and structural integrity [3], and pile driving resistance and static bearing capacity [4]. The method is employed on thousands of project sites around the world annually for testing piles of many sizes and various materials. In addition to testing foundation piles of "common" sizes (i.e., 12 to 36 inch diameter steel pipe piles, 12 to 30 inch square precast driven concrete piles, steel H-piles, and timber piles), testing have been performed on steel pipe piles of up to 80 inches in diameter and 4 inches wall thickness and over 1000 ft in length [5]. Drilled shafts up to 5 ft in diameter and over 100 ft in length have also been dynamically tested [6]. This case history involving small diameter, long and flexible minipiles presents an innovative application of dynamic pile testing.

Dynamic measurements of strain and acceleration under hammer impacts are the basis for modern dynamic pile testing. Strain is measured using reusable transducers and acceleration is measured with piezoelectric accelerometers. Gages are mounted near the pile top, approximately two diameters below its head. Figure 9 shows pile instrumentation. 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 measured signals. A photograph of the Model GCPC Pile Driving Analyzer is presented in Figure 10. Pile strains are converted to forces and accelerations to velocities as a function of time for each hammer blow. Figure 11 presents plots of pile top force and velocity histories for an 86 (in the

Figure 9: Pile dynamic instrumentation



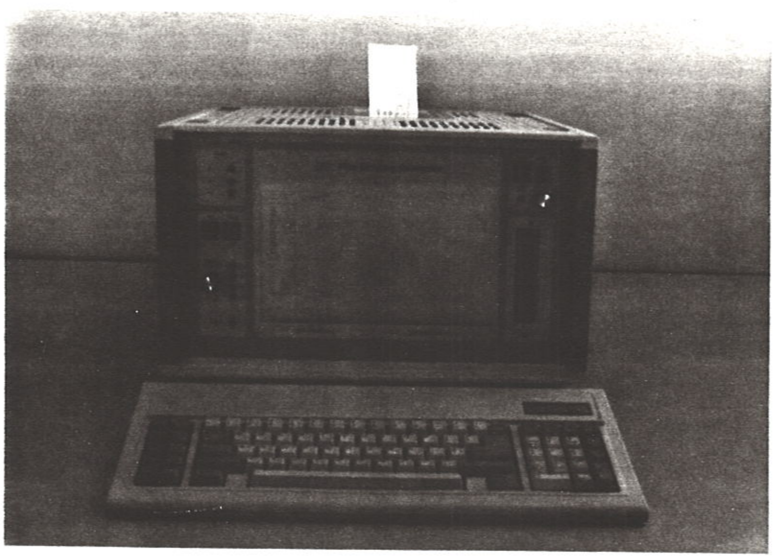


Figure 10: The Pile Driving Analyzer

vicinity of C-2 and C-3) and 175 ft (in the vicinity of C-1 and AB-1) long piles. Force and velocity records are assessed for data quality and are evaluated according to the Case Method [7]. Computations yield information pertaining to pile driving stresses and structural integrity, hammer and driving system performance, and soil resistance and static pile capacity.

Dynamic pile data obtained in the field can further be analyzed in the office according to the CAPWAP method for a more comprehensive understanding of the soil behavior [8]. The analysis is done in an interactive environment using measured data and wave equation type analysis in a signal matching technique. Results from a CAPWAP analysis include comparisons of measured with corresponding computed force/velocity records. Numerically for each segment (typically 5 ft long) of the pile, ultimate static resistance, soil quake and damping factors are tabulated. Also included in the results is a pile load-movement curve from static test simulation. Figure 12 contains plots of CAPWAP results on the 86 ft long piles..

Six of the production piles were dynamically tested. The piles tested ranged in length from 55 to 175 ft covering the whole range of pile penetrations. During driving, pile top compressive stresses averaged 20 ksi, and maximum transferred energies averaged 0.36 kip-ft. This transferred energy translates to an average transfer efficiency of 36%, a value commonly observed on many projects involving double-acting air hammers driving steel piles [9]. Dynamic pile top records obtained during the testing of some of the longer piles did not contain clear toe reflections and had characteristics that would suggest pile bending. Plots of pile top records contained in Figure 11 show the difference in characteristics between the data for the 86 and the 175 long piles. It was not possible to independently verify pile straightness due to the small pile diameter and exceptional lengths.

Dynamically computed static pile capacities ranged between 20 and 30 kips and were generally somewhat higher than originally estimated based on static analysis and preliminary wave equation results. This may be attributed to the conservative approach adopted in interpreting subsurface investigations data during the preliminary design phase. Dynamic data analysis with the CAPWAP program indicated that almost all of the pile capacities were derived from skin friction, particularly for the longer piles. The average computed unit skin friction was approximately 0.5 ksf. Soil damping factors were within generally accepted values with an average of 0.22 s/ft. Soil quake values, however, were relatively low, i.e., 0.075 inch along skin and 0.03 under toe, compared to conventional values. This may be expected given the small pile diameter size. There were no sufficient dynamic data to evaluate soil strength changes with time. Increases in pile driving resistance on several piles between end of initial installation and restrikes some time later, however, suggests the presence of soil set-up and an increase of pile capacity with time.

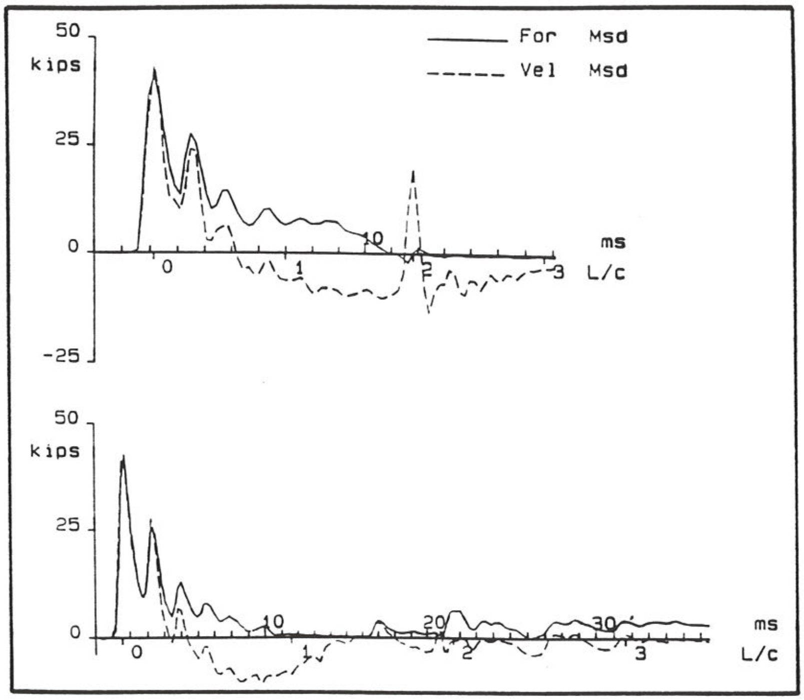


Figure 11: Records of pile top dynamic data for two piles

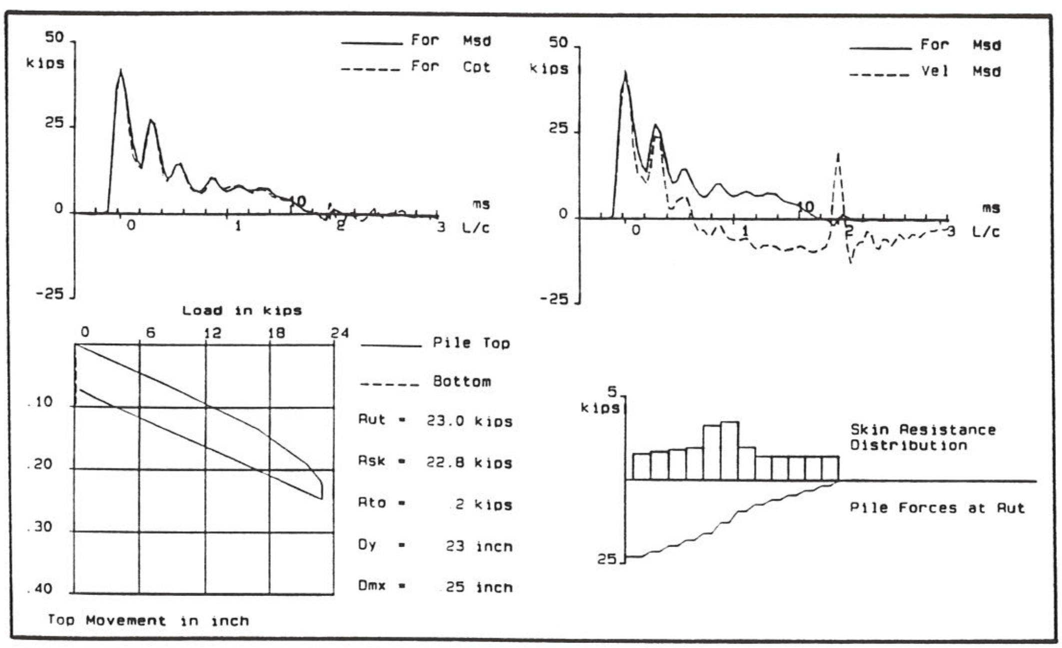


Figure 12: Typical CAPWAP analysis plotted results

SUMMARY

Long closed-ended, small diameter, concrete-filled steel pipe minipiles were employed as an underpinning system of foundations under an old historic structure. The subsurface conditions were highly variable and generally consisted of a deep layer of organic compressible soils interbedded with loose and dense sand layers. The piles had length to diameter (L/d) ratios between 188 and 600. Dynamic pile testing and analysis were innovatively applied to evaluate behavior and bearing capacities of such slender piles in compressible soils.

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APPENDIX - Unit Conversion Factors

To Convert	To	Multiply By
ft	m	0.03048
in	cm	2.54
lb	N	4.45
kip	kN	4.45
Ton	kN	8.9
ksi	MPa	6.89
Kip-ft	kJ	1.36