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16. Abstract
Five continuous-flight-auger piles were installed in a stiff clay soil following standard practice. Four of these piles were later subjected to lateral load tests to large displacements. The length and diameter of the test piles were varied to investigate the effect of these parameters on pile performance. The fifth pile was used as a reaction for the four test piles. During installation, incremental grout take and upper and lower bound grout pressures were monitored continuously, and grout samples were acquired and tested. Cross-hole ultrasonic tests were performed on the piles following installation to ensure that the piles were structurally sound. A simple design model, the characteristic load model, was adapted from the literature, and soil parameters for that model were determined from geotechnical data available at the test site (the University of Houston National Geotechnical Experimentation Site, partially supported by the Federal Highway Administration) and by back-analysis of the loading test results. The final design model is intended for use in designing foundations for sound wall structures and is capable of predicting the deformation of the pile under a combined shear and moment load, as well as the loads that produce cracking in the pile. A provisional, detailed construction specification is presented that is based on specifications in effect in Texas and other states for continuous flight auger piles, current practice and observations made during the performance of the research.

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SPECIFICATION AND DESIGN CRITERIA FOR THE CONSTRUCTION OF CONTINUOUS FLIGHT AUGER PILES IN THE HOUSTON AREA

Final Report to the Texas Department of Transportation for

Project Number 7-3921

by

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August 1997

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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

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PREFACE

The research reported herein was motivated by the need of the Texas Department of Transportation (TxDOT) to design and construct foundations for sound walls in the Houston District reliably and economically. TxDOT had a favorable experience with the construction of continuous-flight-auger (CFA) piles, commonly called augercast piles, for a sound wall project in Harris County, and wished to explore further the possibility of using CFA piles for that purpose on future projects.

Sound wall design is controlled by lateral loading, from both wind and vehicle impact; hence, the research focused on the lateral-load behavior of CFA piles. During the course of this project, a simple design method for lateral loading was developed for use in the stiff clay soils of the Houston District.

CFA piles are constructed by augering a hole continuously into the earth and injecting cementitious grout into the augered borehole as the auger is withdrawn. Since grout is used as a structural material, it was considered necessary to study the mechanical properties of potential grout mixes, including one that is commonly used in the private sector, and to investigate chemical attack upon the grout. Such studies were performed in the laboratory.

Recommendations were also developed for the construction of CFA piles for sound walls in the form of a specification that includes construction processes and grout behavior. This specification was written in such a way that it can potentially be modified and adapted as more experience is accumulated by TxDOT for the construction of CFA bearing piles for structures.

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ABSTRACT

The technical literature was reviewed, and other state departments of transportation were surveyed concerning experience with the continuous-flight-auger (CFA) process of pile construction. While most state DOT's do not use CFA piles, a few (including Texas) were found to do so and have developed preliminary specifications for their construction. An industry standard has also been developed for the construction of CFA piles. Based on this information, the observation of the construction of test piles and discussions with authorities on CFA pile construction, a preliminary construction specification for the Texas Department of Transportation was prepared. This specification is rather detailed because the consequences of poor construction can be a foundation of compromised integrity.

A simple design method was also developed. The method makes use of the characteristic load method developed by others and analytical modeling of the applied ground-line loads that produce cracking in piles. The method permits the use of a wide range of pile diameters in clay soils of varying strength and assumes that the failure load is the load that produces cracking in the pile. In order to obtain benchmarks for calibration of the analytical model, as well as to observe quality control systems for construction, four test piles and one reaction pile were constructed at the National Geotechnical Experimentation Site at the University of Houston (NGES-UH). These piles were subjected to lateral loading tests to loads well beyond structural failure (cracking).

During construction, grout pressures and incremental pumped grout volumes were monitored with a prototype commercial CFA pile construction monitoring system, which proved to operate successfully. The use of such a monitoring system was incorporated into the preliminary construction specification. Various post-construction integrity testing systems were used to verify the as-built quality of the piles.

Simultaneously with the field loading tests, mechanical and chemical studies were performed in the laboratory on CFA pile grouts. It was found that a cement-rich grout mix with fly ash and a fluidizer produced very flowable material, that developed a compression strength in excess of 34.5 MPa (5,000 psi). Its tensile strength was relatively low compared to that of concrete with a comparable strength. That characteristic was taken into account in developing the design model. Studies were also performed on fiber-reinforced grout mixes with increased fluidizer and grout mixes subjected to chemical attack. While some small improvement in properties was found with some additives, the standard field mix that was used in the construction of the test piles was found generally to be the optimum grout mix design.

SUMMARY

Lateral loading tests were performed to structural failure on four continuous-flight-auger (CFA) piles with varying lengths and diameters at the National Geotechnical Experimentation Site at the University of Houston (NGES-UH). These tests were used as a benchmark for verifying and modifying a simple design model for laterally loaded CFA piles that is intended for use in the stiff clays typical of the soils in the Houston District. The occasion of the construction of these test piles, and one anchor pile, was also used to evaluate construction practices and the deployment of a simple quality control device that monitors incremental grout volume and pressures as the grout is being placed. Use of such a device should provide adequate assurance of structural quality of CFA piles in stiff clay soils.

Mechanical and chemical properties of the field grout and grouts made with variations of the field grout mix were studied in the laboratory. The results of this study, along with experiences gained in the field and a review of specifications and guidelines of other agencies, were used to arrive at a preliminary construction specification for CFA piles that may be used by the Texas Department of Transportation.

IMPLEMENTATION STATEMENT

The simplified design method presented in Chapter 4 is intended to be implemented directly by the Texas Department of Transportation (TxDOT) in the design of CFA piles for sound wall foundations. A preliminary construction specification is given in Chapter 6. That specification is intended to be a candidate for inclusion in the Department's standard specifications. However, the authors realize that any specification should be a consensus of those individuals within TxDOT who will need to enforce it. Therefore, it is fully intended that this specification undergo detailed internal review within TxDOT, and perhaps modification, to ensure that it does not conflict with TxDOT's policies and philosophies of specification presentation before it is employed. It is appropriate that this specification, or a modified version of this specification, be used first as a special provision on several projects before it is considered for adoption as a standard specification.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the assistance of Xibao Ni and Guntaka Reddy, graduate students in the Department of Civil and Environmental Engineering at the University of Houston, who conducted many of the computer analyses and performed most of the laboratory tests. The are also indebted to Mr. Joe Thomas, senior engineering technician at the University of Houston, for preparing the loading system for the field tests.

This research would have not been possible without the generous contributions of Berkel and Company, Inc., who installed the CFA piles and who reviewed the preliminary construction specifications; SMI, Inc., who furnished the steel to construct the piles; Pile Dynamics, Inc., who performed the automatic monitoring of the CFA pile construction; Fugro-McClelland Southwest, Inc., who performed the sonic integrity tests and provided the digitilt inclinometer; and Stress Engineering Services, Inc., who performed the fiber-optic logging of the reaction pile.

The authors also acknowledge the contribution of the University of Houston for providing the test site.

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CHAPTER 1: INTRODUCTION

General

Continuous flight auger (CFA) piles are constructed by excavating a continuous column of soil, typically about 0.3 to 0.9 m (12 in. to 36 in.) in diameter, with a continuous-flight, hollow-stem auger, and injecting grout into the space left by the auger as it is removed. Reinforcing steel cages can then be inserted into the grout after the auger is fully withdrawn and before the grout sets. CFA piles can be used as an economic alternative to other types of deep foundations in several applications. Historically, Department of Transportation engineers have used either driven piles or drilled shafts to support bridges, signs and walls. In comparison with these deep foundations, CFA piles can be installed more rapidly, resulting in major cost reduction. Despite this cost saving, however, the use of CFA piles is not widespread in transportation structures in the USA due to the uncertainties of the construction control.

CFA piles have been used to support buildings and industrial structures in the private sector for at least the past 25 years in the Houston-Galveston-Beaumont area. One notable problem with CFA piles over this period has been the high failure rate of piles subjected to axial load tests. Usually, these failures can be traced to structural defects associated with rapid extraction of the auger, in which suction pressures are exerted on the grout being discharged at the outlet orifice at the base of the auger, which then forms a neck (reduced cross-section). There has also been evidence in other tests, all in sands, that very low load transfer has been developed in the absence of structural defects because of improper construction controls. Despite these problems with load tests, however, there is no evidence of any superstructure failures in the Texas Gulf Coast area caused by structurally deficient CFA piles supporting structures that are actually in service.

Ordinarily, CFA piles are designed only to take axial load and only after load tests or appropriate nondestructive tests are performed to prove their structural integrity. Recently, the Houston District of TxDOT, for reasons of economics, has begun to specify the use of large-diameter [0.914 m (36 in.)] CFA piles for sound wall structures, most notably on a project on I-610, near Post Oak Boulevard in Houston, Texas. The loading on sound walls is predominately lateral, through wind pressure and vehicle impact. Little formal research has been done worldwide into the lateral-load performance of CFA piles; however, lateral loading is a major part of the sound wall application, so that continued use of CFA piles for sound walls and other future structures subjected to lateral loading will require development of a formal design procedure based on rational research.

The use of CFA piles by the Houston District of TxDOT has been limited by the lack of proven construction specifications and design methods. The objectives of this report are to provide reliable construction specifications and a design method for laterally loaded CFA piles so that they can be designed safely and efficiently in Texas coastal soils. This is accomplished through the following tasks:

1- Investigate the structural performance of CFA piles through construction monitoring and full-scale field lateral loading tests.

2- Use the results of the field lateral loading tests to develop p-y curves and a simple design method for CFA piles in stiff clay, which is typical of Texas coastal soils.

3- Survey the use of CFA piles in transportation practice in the USA and develop a preliminary construction specification.

Literature review

CFA pile systems came into use in the late 1940's, originally in the United States. Today, more than 300,000 meters of CFA piles are installed annually [Neate (1989)]. CFA piling has now gained rather wide acceptance worldwide. Ordinarily, CFA piles are of small diameter and moderate length [0.4 to 0.6 m (16 in. to 24 in.) in diameter and up to 30 m (100 ft) deep]; however, CFA piles as large as 0.9 - 1.0 m (36 to 40 in.) in diameter have been installed routinely in Japan and are beginning to be used in Texas (specifically, the Houston District of TxDOT).

Practice-oriented publications indicate that the popularity of CFA piles is high, and potentially attractive in highway construction, because of

- Fast construction and light, mobile construction equipment, resulting in reduced costs;
- Low noise and vibration, an advantage in urban environments;
- Little or no loss of ground or ground heave, an advantage at sites with adjacent structures; and
- No need for drilling slurries when sandy soils are encountered, as with drilled shafts.

Design methods

Axial loading

As with all piles, CFA piles resist applied axial load through a combination of shaft resistance (skin friction) and toe resistance (end bearing). In the United States, design rules have been worked out for axial loading of other types of foundations [e. g., Reese and O'Neill (1988)] through both theoretical considerations and back-analysis of large data bases. However, relatively little systematic study of the axial resistance of CFA piles has been performed in the United States, probably because private sector owners (the most frequent users of CFA piles) rely largely on the CFA pile contractor to provide a pile of required capacity through experience, coupled with site-specific load testing to verify the contractor's resistance estimates. As a result, design rules are relatively few and generally unproved in broad application.

By contrast, in Europe, rational design rules are well established for axially loaded CFA piles [Bustamante and Gianeselli (1993), DIN 4014 (1987), and O'Neill (1994)], and CFA piles are frequently used on public sector projects. One of the investigators is personally familiar with the use of many thousands of Starsol-type (trade name of the

Soletanche Company) CFA piles [Whitworth (1994)] for the Lyon Bypass Route of the French TGV. Other uses of these piles have occurred for over ten years in transportation projects in France.

In Germany, the national design code [DIN 4014 (1987) and Rizkallah (1988)] permits the same unit resistances to be used for CFA piles as for bored piles (drilled shafts), based on q_c values from cone penetrometer tests; however, a rigorous standard for construction is established. In France, design rules have been devised based on *in situ* tests, primarily the CPT, the PMT and the SPT [Bustamante and Gianeselli (1993)]. It should be pointed out, however, that the construction procedures in Europe and the United States tend to be different. In Europe, high-torque [136-272 kN-m (100,000 - 200,000 ft-lb)] rigs are used essentially to screw the continuous flight augers into the ground. Lighter rigs are used in the United States.

Van Impe et al. (1991) describe a philosophy of construction that ensures that the excavation method does not result in soil being "mined" from around the pile, thus reducing the lateral pressure that the soil exerts on the pile and thereby reducing the pile's resistance. In order to avoid soil mining, the downward rate of penetration of the auger, v, must be at least as large as a specified value. This philosophy of construction will discussed in Chapter 2. It is practical to attain such rates of penetration during excavation with high-torque rigs, and the European design criteria are based on this quality assurance criterion, which ensures that the ground will not be "depressurized." For example, the "LPC" (French) method, Bustamante and Gianeselli (1993), predicts ultimate toe resistance Q_B from

$$Q_B = K\, A_B\, N_{avg} \tag{1.1}$$

where K = 0.9 - 1.2 in clay and 1.8 - 2.1 in sand, A_B is the bearing area of the toe (base), and N_{avg} is the average SPT blow count in blows/0.3 m (blows/foot) from 1.5 diameters above to 1.5 diameters below the pile toe. Similar simple equations are given for the

CPT and PMT tests, which are common methods for subsurface exploration in France. Ultimate shaft resistance Q_S in the same method is given by the equation

$$Q_S = f_S A_S \ , \tag{1.2}$$

where f_s is a unit value of shearing resistance determined from tables and graphs for soils of various general physical and numerical descriptions. For example, in a clay or clayey silt, where N from an SPT test averaged over the length of the pile is 15, f_s is assigned a value of 0.035 MPa, or about 730 psf. The corresponding value in a sand would be 0.060 MPa or about 1250 psf.

Finally, Q_{TN} (nominal ultimate resistance of the CFA pile) is given by

$$Q_{TN} = Q_S + Q_B \ . \tag{1.3}$$

In European practice a factored resistance is used in an LRFD design context, i. e.,

$$Q_T = \phi \, Q_{TN} \quad \text{or} \quad \phi_1 \, Q_S + \phi_2 \, Q_B \ , \tag{1.4}$$

where ϕ is a global resistance factor and ϕ_1 and ϕ_2 are individual resistance factors for shaft and toe resistance, respectively.

European standards also require a high level of quality control of CFA pile installation on most public sector projects. A quality control innovation in the past 10 years has been the Enbesol method, developed by Soletanche, and similar methods by other large contractors, in which torque, rate of penetration, and rate of rotation of the auger are automatically and continuously monitored and recorded during excavation by a unit in the cab of the drill rig to assure that soil mining does not occur. During grouting, as the auger is being withdrawn, grout pressure, position of the tip of the auger and grout take are also monitored and recorded to assure that necking of the grout column is not occurring. If necking is indicated by the Enbesol instrumentation, the contractor can merely drill back into the wet grout in the defective section and re-initiate the grouting sequence, since the auger is still in the hole and the grout is still fluid. Frequently, cross-hole ultrasonic logging is also required in the finished pile as a secondary check of structural integrity. If such is required, tubes (usually PVC) are placed on the reinforcing cage before the cage is inserted into the grout (usually by vibration). Nondestructive tests are ordinarily conducted after the grout has hardened; however, recent research in Asia and in the United States [Brettman and Frank (1996) and Brettman et al. (1996)] has suggested that accurate detection of defects in the grout can be made while the grout is still unset using a single-tube ultrasonic device, giving the contractor a second chance to remove the grout and reinitiate the grouting sequence if a defect is detected.

In the United States, on the other hand, CFA piles, which are commonly referred to as "augercast piles," are viewed as inexpensive alternates to other types of deep foundations because contractors normally use small, low-torque rigs [typically around 27 kN-m (20,000 ft-lb)] and do not employ quality control measures equivalent to those used by European contractors. The use of low-torque rigs requires that the contractor excavate the soil by mining it (scraping the soil off the sides of the borehole and working it up the continuous flight auger to the surface, which can allow soil outside of the immediate area of the borehole to flow into the borehole, which in turn reduces the ground pressures and subsequently the ultimate axial, and possibly lateral, resistance of the CFA pile). The effect of this important detail has not clearly been quantified, which makes it imprudent to use European design criteria for design in the United States without careful analysis.

Some significant research into the performance of CFA piles constructed in sand by United States' contractors has been performed. Neely (1991) summarized the results of a moderate data base of load tests on CFA piles in sand by proposing that ultimate shaft resistance Q_s be computed using the following expression:

$$Q_s = \beta\, \sigma'_{v\,avg}\, A_s \quad , \tag{1.5}$$

in which A_s is the perimeter area of the CFA pile in contact with the soil, $\sigma'_{v\,avg}$ is the mean ambient vertical effective stress in the soil surrounding the pile (vertical effective stress at mid-depth of the pile in a uniform soil) and β is an earth pressure-wall friction coefficient, which is a function of the pile penetration (L). For example, for L = 5 m (16 ft), β = 2.70; for L = 10 m (33 ft), β = 0.85, for L = 20 m (66 ft), β = 0.25. Although β also decreases with depth in driven piles and in drilled shafts in sand [Reese and O'Neill (1988)] the more rapid decay of the factor β with increasing pile penetration for CFA piles in Neely's method suggests the effects of soil mining and depressuring.

McVay et al. (1994) reviewed the performance of 21 CFA piles constructed and load tested in Florida (primarily in sand) and concluded that the construction details were important factors in predicting the ultimate axial resistance. In particular, the selection of equipment, the rate of penetration, grout fluidity, the aggregate size in the grout, pumping pressures and rates, and the rate of extraction of the auger were cited as key variables. With the low-torque rigs commonly used in Florida, it was recommended that primary elements in QC procedures be (1) limiting the pitch on the auger to be one-half the auger's outer diameter, (2) monitoring the grout pressure and maintaining it so that it does not decrease as the auger is withdrawn (decreasing pressure indicating suction at the tip of the auger and corresponding possibility of necking), and (3) verifying that the overall grout take is 1.2 to 1.5 times the neat volume of the borehole.

McVay et al. (1994) also compared computations of axial resistances predicted by several prominent methods for driven piles and drilled shafts with the measured resistances in load tests. Defining failure of a CFA pile as the applied load corresponding to a settlement of 5% of the nominal diameter of the pile, the FHWA method for drilled shafts [Reese and O'Neill (1988)] gave an acceptable ratio of computed to measured resistance of 1.04, but with a large standard deviation (0.28). Neely's method for CFA piles performed comparably, giving only a slightly higher prediction ratio. Design methods for driven piles considerably overpredicted the resistance of CFA piles (much higher prediction ratios) and had even larger standard deviations.

Soils in the Houston District of TxDOT, however, are predominantly stiff, overconsolidated clays and clayey silts [O'Neill and Yoon (1995)], although some fine, waterbearing sand layers are encountered. Very little information is available in the literature about the effect of using low-torque rigs to excavate stiff clays for CFA piles or on the possible correlation of design methods for drilled shafts, which are well-known in coastal Texas soils, with those for CFA piles. It is unlikely, based on principles of soil mechanics, that the effect of low-torque augering on load transfer in CFA piles in stiff clays will be vastly different from the effects of installing drilled shafts with short soil augers in stiff clays; however, that assumption remains to be shown to be correct.

Lateral loading

Little documentation of the results of research on laterally loaded CFA piles has been found in the literature. Dunnavant and O'Neill (1989) studied the effects of both foundation size and installation method (driven piles and drilled shafts) on the behavior of laterally loaded piles in the Beaumont clay formation (typical stiff clay found in the Houston District) at the National Geotechnical Experimentation Site at the University of Houston, located about 4 km southeast of downtown Houston. In that study, relatively little difference in the installation method was observed in large-diameter piles [1.2 m (48 in.) - 1.8 m (72 in.) in diameter]. However, no such information was obtained for piles of smaller diameters that are more typical of CFA piles.

A standard high-level method of analyzing the lateral load-deformation-moment behavior of drilled shafts or concrete piles is the use of numerical versions of the one-dimensional beam-column equation with coupled nonlinear soil resistance [e. g., Reese and Wang (1995)]. That method has the capability of handling the nonlinear bending behavior of the concrete and steel in the cross section, including yielding of the steel and cracking of the concrete [Wang and Reese (1987)] which makes it an ideal tool for analyzing the lateral load behavior of CFA piles and possibly for development of design charts for CFA piles in specific soils. Needed for input, however, are the stress-strain properties of the grout and the lateral soil resistance relations, or "p-y curves," which are presently unknown. In Texas coastal soils it may be appropriate to use formulations for p-y curves that have been developed from analysis of lateral loading tests on drilled shafts [Dunnavant and O'Neill (1989) and Welch and Reese (1972)]; however, such an assumption will need to be verified, and in all likelihood modified to account for differences in installation method and foundation diameter. Stress-strain behavior of grouts needed to define the structural behavior of the CFA pile will be addressed in Chapter 5.

In the recent past, design of CFA piles for sound walls has been executed by the Houston District of TxDOT by assuming that the TxDOT criteria for drilled shafts apply to axial resistance. For lateral loading the pile is assumed to behave as a cantilevered sheet pile for purposes of computing necessary penetrations and levels of safety against overturning based on methods in textbooks (S. Yin and S. Mebarkia, personal communication).

Structural integrity and other construction effects

As stated earlier, mining of soil during excavation and excessive rates of extraction of the auger while pumping grout are the main concerns in construction. These factors are well controlled with the quality controls developed in Europe. In the United States research into the provision of similar quality control features at low cost have only just begun in earnest, but it can be expected that some sort of quality control system, similar to the Enbesol system used in France, will be appearing in the USA soon (G. G. Goble, personal communication). In the meantime, it needs to be established whether the lower level of quality controls described by McVay et al. (1994), perhaps in combination with ultrasonic logging [Brettman and Frank (1996) and Brettman et al. (1996)] or other post-construction non-destructive evaluation [Rausche et al. (1994)] will be adequate for control and assurance of structural integrity of CFA piles in Texas coastal soils.

TxDOT has developed a preliminary specification for construction of CFA piles in coastal soils (S. Yin, personal communication) entitled TxDOT Special Specification -- Item 9000, Augered Pressure Grouted Piles, Feb. 1995. The point of view of TxDOT on the use of CFA piles is that construction specifications should be as open as possible, consistent with the assurance of a maximum degree of structural integrity, so as to foster competition among potential contractors and keep construction costs at a minimum. The literature review revealed a number of papers and manuals on recommended practice to maintain good integrity of CFA piles and adjacent structures with present United States practice, including case histories [e.g., DFI (1990), EBA, Inc. (1992), Esrig et al. (1994), Lacy et al. (1994), McVay et al. (1994) and Neate (1991)]. These documents provided guidance for updating the present preliminary TxDOT construction specification while maintaining the philosophical intent of openness.

Materials

Grout and steel are the materials used in the construction of CFA piles. The primary material over which the designer has control is the grout, and that is a major focus of this study.

Grouts used in the construction of CFA piles are usually rich in cement in order to improve pumpability (from the surface through the hollow stem of the auger into the borehole) and flowability once in the borehole. Cement content typically ranges from 8 to 11 sacks per 0.76 cubic meter (1 cubic yard). For maintaining good pumping and flow characteristics, the aggregate is generally limited to sand within the gradation of concrete sand (e.g., ASTM C 33). A grout fluidizer combining the functions of a retarder and a pumping aid is often added to the mix. Field control of grout consistency is maintained by the use of the grout flow cone (ASTM C 939). Since the grout is cement-rich, shrinkage is a potential problem that can be controlled by adding a pre-hardening expansive gassing agent to the mix at the job site.

Unless mistakes have been made in the installation process, the volume of grout injected to form a pile will always exceed the neat volume of the specified pile dimensions. Grout volume as installed will range from as little as 110 percent of the neat volume in stiff clays to 150 percent or more in low density silts. A frequent requirement in CFA pile construction is that adjacent piles closer than six diameters cannot be placed until after final set of the initial pile.

Performance testing of grouts is critical for the establishment of both construction specifications and design criteria, and such testing is a part of this study. The reasons for performance testing of grouts in this study are as follows [ASTM (1995), U. S. Grout Corporation (1981) and Gulyas et al. (1995)]:

1. Grouts are covered under the ASTM C 1107-91 specification, which establishes strength, consistency, and expansion criteria. This specification lists three general types of grouts, depending on their volume-change characteristics: (a) pre-hardening volume controlled types; (b) post-hardening volume controlled types, and (c) combined volume controlled types. Workability of these grouts is defined by their consistency classification using the ASTM C 939 Flow Cone. Despite ASTM C 1107-91's being called "Specification for packaged dry, hydraulic cement grout," there is no requirement in the specification for two very important properties of a high-quality grout that may apply to CFA piles: maximum allowable shrinkage and minimum strength. These must therefore be determined for field mixes.

2. Soil types and thickness of the soil layers in which the CFA piles are installed will affect the grout mix design. Grouts can lose water and harden prematurely in some soil formations. Hence, grout mix designs should correlate to soil factors.

3. ASTM C 1107-91 does not differentiate grout based on the type of aggregate. But, the type and grading of the aggregates will play an important role in the grout behavior and were investigated in this study.

4. Grouts that have a minimum strength exceeding 27.6 MPa (4000 psi) may be utilized for CFA pile grouting. Even though a 27.6 MPa grout is considered to be of good quality, the compressive strength of the grout is reported by 50.8 mm (2 in.) cube specimens, while pile designs are normally based on 150-mm (6-in.) cylindrical specimens (ASTM C 39). An adjustment must be made for converting cube strength to cylinder strength, and typically a strength reduction factor ranging from 0.75 to 0.80 is used to adjust compression test results on 50.8 mm cubes to equivalent cylinder strength for 26-mm (3-in.) cylindrical specimens. This factor will be investigated.

5. The presence of weak acids or sulfate solutions in the ground water may have negative effects on the long-term performance of the grout. Performance testing of grouts exposed to such chemicals is advisable and are reported.

6. In order to develop design charts and equations for the use of the Houston District of TxDOT, laterally loaded CFA piles must first be analyzed using a rigorous nonlinear method, such as the method described earlier. Such analysis will require knowledge of the stress-strain diagrams for the grouts used in construction of CFA piles. It is emphasized that because of the use exclusively of fine aggregates, a cement-rich paste, and low-shrink agents, stress-strain behavior of grouts can not be computed from familiar formulas for concrete based on compression strength. Instead, direct measurements must be made.

As a result of these concerns, limited laboratory investigations of the potential grout mixtures for CFA piles for TxDOT were included in this study. Some of the conditions that were investigated are listed below.

- Cementitious grouts with fluidizer additives. These additives to some extent reduce shrinkage and help pumpability. The particular fluidizer that was studied is a proprietary product of Berkel and Company.

- Cementitious grouts with added fly ash [Vipulanandan and Shenoy (1992)]. Fly ash can reduce the amount of cement in the grout mix and hence the cost of the mix. It can also aid in pumpability, reduce bleeding and reduce shrinkage.
- Susceptibility of grout mixes to chemical attack.
- Compression strength, tensile strength and stress-strain behavior of CFA pile grouts.
- Effects of fibers in the grout mix on the physical properties of the grout.

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CHAPTER 2: FULL-SCALE LOADING TEST PROGRAM AND TEST PILE INSTALLATION

General

Four CFA test piles were constructed at the University of Houston National Geotechnical Experimentation Site, NGES-UH. These piles were load tested laterally in order to obtain high-quality data that can be used to develop p-y curves and a simplified design procedure for CFA piles in stiff clay soil typical of the soil found in the Houston District.

The field lateral loading tests were conducted on full-sized CFA piles instrumented with inclinometer casings to measure the profiles of lateral deflection along the piles using a digitilt inclinometer probe. A Pile Installation Recorder™ (PIR) was used to monitor the pump grout pressures and the grout-volume ratio incrementally during the construction of two of the test piles. In addition, two types of integrity testing techniques were utilized to evaluate the integrity of the grout in the test piles after construction. One of these techniques, cross-hole ultrasonic logging, is a relatively common method for the quality control of drilled shafts and CFA piles. This type of integrity testing was performed on all four test piles. The second integrity testing method allows visible inspection of the grout by inserting a fiber-optic television camera through a transparent tube embedded in the pile grout. This technique was recently introduced to the deep foundation industry and was performed only on a fifth CFA pile that was used as a reaction for testing the other four piles.

Geotechnical data for the test site

The NGES-UH, is a well-known test site for foundations. It is a microdelta depositional site of Pleistocene age within the Beaumont formation. It represents the lower limit for theoretical preconsolidation in the region and possibly in the Beaumont formation. The Beaumont formation is underlain at the site by an older Pleistocene formation termed the Montgomery formation. The Beaumont-Montgomery contact, at the site, is at a depth of 8 m. A general profile for the site is shown in Fig. 2.1. Two of test piles were installed entirely in the Beaumont formation, to a depth of 6.1 m. The depth of the other two test piles was 10.67 m. However, the experimental evidence, presented later in this chapter and Appendix A, showed that the latter test piles attained their lateral capacity almost entirely in the Beaumont formation. At the test site, the Beaumont formation consists of 3.97 m (13.0 ft) of stiff gray and tan clay (CL-CH), underlain by 4.03 m (13.2 ft) of very stiff red and light gray clay (CH). The average liquid and plastic limits of the Beaumont formation are 61 and 19, respectively, and those of the Montgomery formation above 10.67 m (35 ft) are 29 and 15, respectively. According to O'Neill and Yoon (1995), the average overconsolidation ratio in the Beaumont formation is 7, while in the Montgomery formation above 10.67 m, it is 5. A detailed study of the engineering properties of the Beaumont formation is given by O'Neill and Yoon (1995). Their study suggested that the most consistent routine for profiling the undrained shear strength, s_u, at the NGES-UH appears to be the cone penetration test (CPT). Figure 2.2 shows the results of an electronic CPT test (10 cm^2 tip area) conducted at the CFA test pile location. Using these results, the undrained shear strength, s_u, can be computed according to the following equation:

$$s_u = (q_t - \sigma_{vo}) / N_k \quad , \tag{2.1}$$

where

q_t = cone tip resistance,

σ_{vo} = vertical total stress, and

N_k = coefficient of 19 for Beaumont formation and of 23 for Montgomery formation (O'Neill and Yoon, 1995).

Equation (2.1) leads to s_u of 103.43 kPa (15 psi) for the upper 6 m (19.7 ft) of the Beaumont formation and to s_u of 179.3 kPa (26 psi) for the Montgomery formation above 10.67 m, using a total unit weight of 19.9 kN/m^3 (127 pcf) in the Beaumont and 20.7

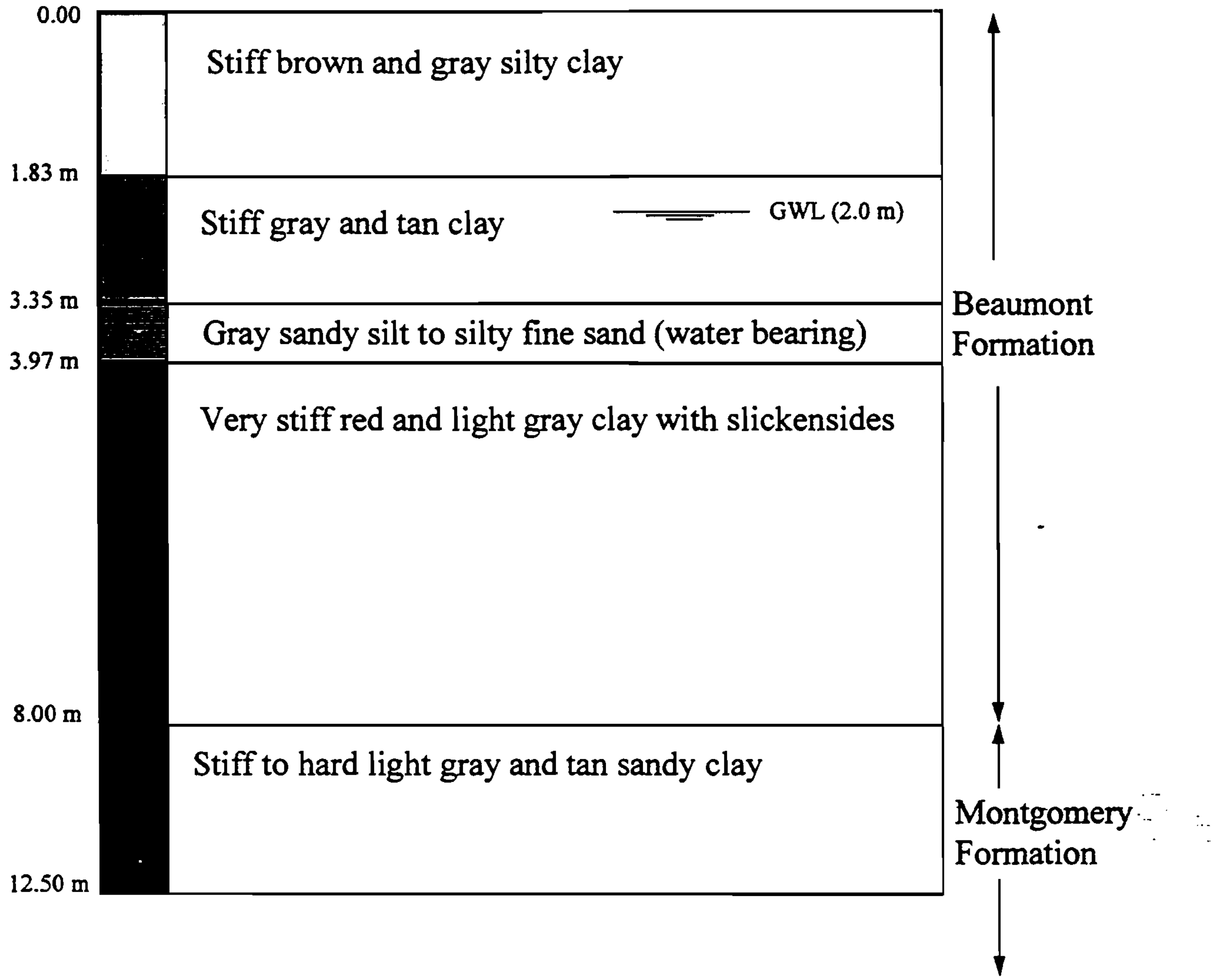

Fig. 2.1. General soil profile for the test site

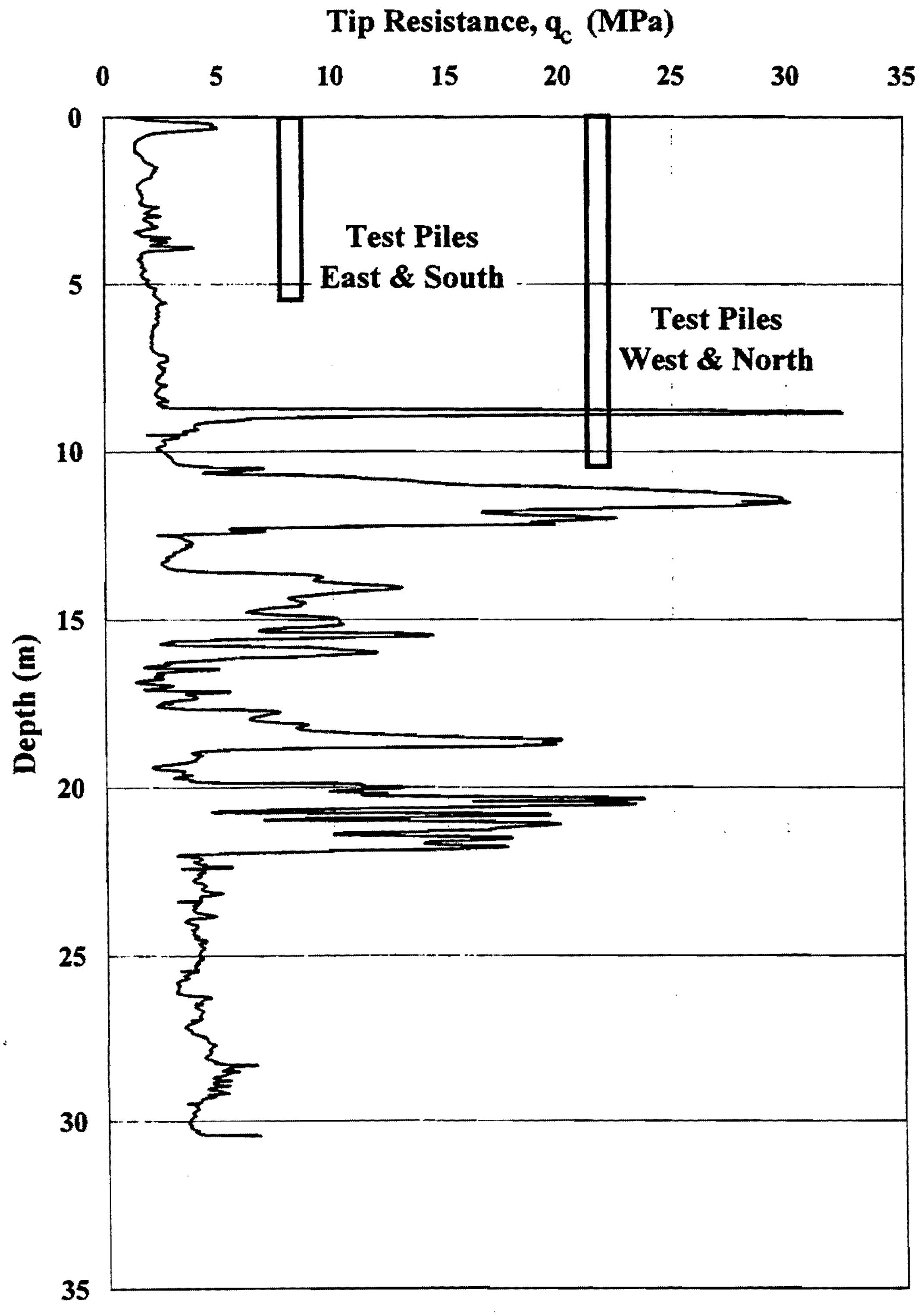

Fig. 2.2. Results of electronic CPT log

kN/m^3 (132 pcf) in the Montgomery to compute σ_{vo}. The Young's modulus, E_s, can be calculated according to O'Neill and Yoon (1995) by the following equation:

$$E_s / s_u = 206 + 1.4z \text{ (m)} \quad , \quad z < 20 \text{ m} \quad . \tag{2.2}$$

A TxDOT dynamic cone penetration test was also conducted within approximately 5 m (16 ft) of the quasi-static CPT at the CFA test pile location. The log of the TxDOT cone test, is given in Fig. 2.3. Based on a simple correlation between the TxDOT cone penetration resistance values in blows per 0.3 m (1 ft), N_{TxDOT}, and the undrained shear strength given by the quasi-static CPT test, s_u, the value of s_u indicated by the TxDOT cone test can be expressed as follows.

Beaumont formation:

$$s_u \text{ (kPa)} = 11.5 \, (N_{TxDOT}) \tag{2.3}$$

Montgomery formation:

$$s_u \text{ (kPa)} = 8.0 \, (N_{TxDOT}) \tag{2.4}$$

The piezometric surface was located at approximately 2.0 m (6.6 ft) below the ground surface at the time the test piles were installed .

Construction of the test piles

The test piles, as well as the reaction pile, were installed at no cost to the State by Berkel and Company Contractors, Inc., of Bonner Springs, Kansas. All the structural steel for the reinforcing cages described below was furnished courtesy of SMI, Inc., of Houston. The test piles were positioned around the reaction pile, as shown in Fig. 2.4. They were designated South, North, East and West. The geometries of the four test piles

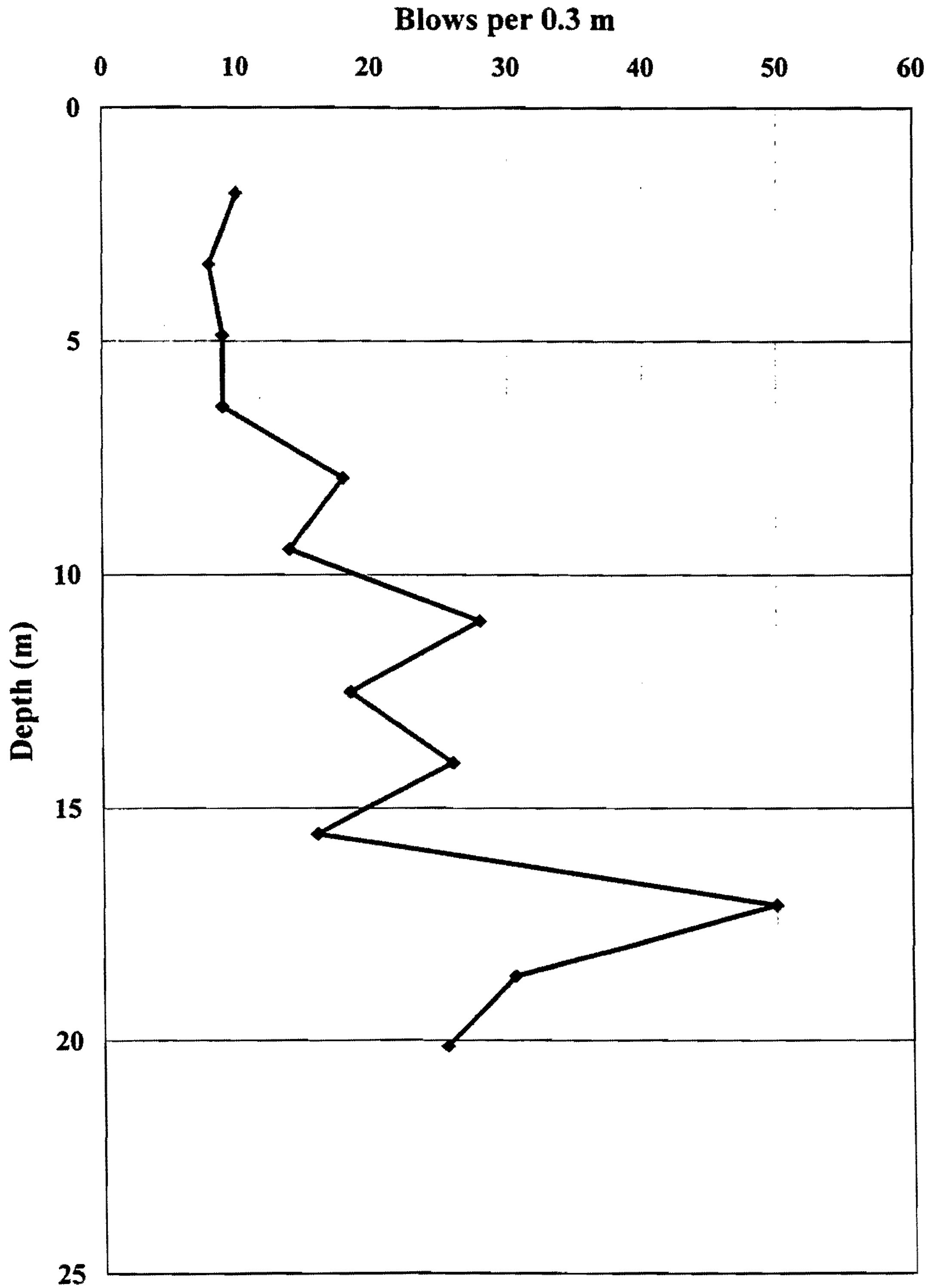

Fig. 2.3. Results of TxDOT cone log

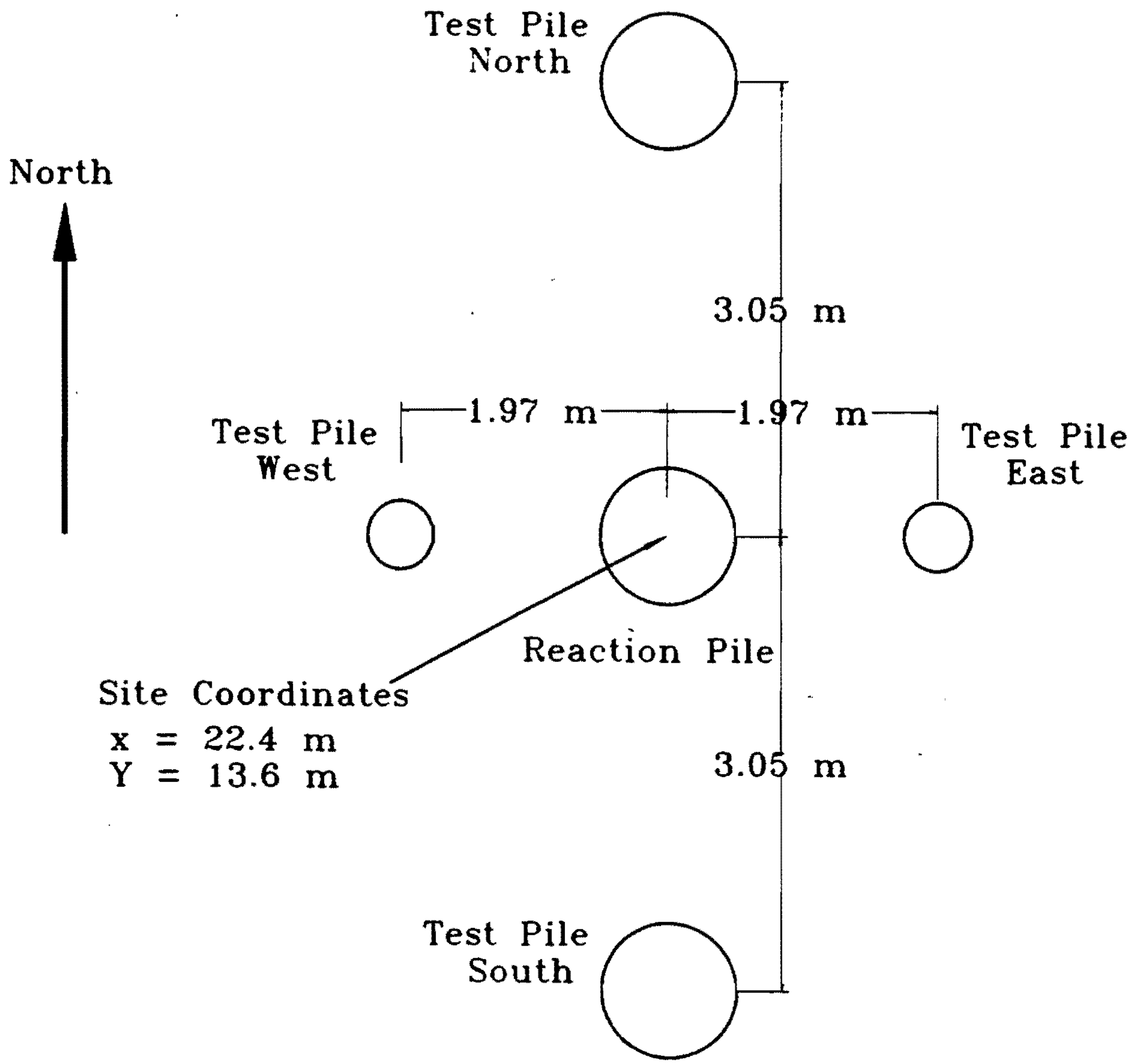

Fig. 2.4. Arrangement of the test and reaction piles

were selected to bracket the diameters and lengths of CFA piles that in the judgment of the authors would be required for sound wall foundations in the Houston District. The largest diameter selected was 0.914 m (36 in.), and the greatest length selected was 10.67 m (35 ft). The smallest diameter selected was 0.457 m (18 in.), and the shortest length was 6.10 m (20 ft). Each of the four possible pairings of these lengths and diameters was constructed and tested. Test Piles South and North were 0.914 m (36 in.) in nominal diameter, and 6.1 m and 10.67 m (20 ft and 35 ft) in depth, respectively. The others, East and West, were 0.457 m (18 in.) in nominal diameter, and 6.1 m and 10.67 m in depth, respectively. The reaction pile was 0.914 m (36 in.) m in diameter and 13.72 m (45 ft) in depth. The longitudinal reinforcing steel used in Test Piles South and North was 8 #10 grade 60 steel deformed bars, and in Test Piles East and West was 6 #6 grade 60 steel deformed bars. This represents about one percent steel for all test piles, which is the standard TxDOT minimum. In the reaction pile, three percent steel (14 #14 bars) was used. Figure 2.5 shows the schematic details of test pile reinforcement along with the idealized shear strength profile of the soil at the site used in later analyses of lateral load behavior.

The piles were installed by rotating a continuous hollow-stem flight auger into the ground until the required penetration was achieved and then pumping grout through the auger stem under pressure as the auger was slowly withdrawn to fill the drilled hole. The proportions of the grout constituents used in all piles are given in Table 2.1, and the mechanical and material properties of the grout are given in Table 2.2. Further details of the grout behavior are given in Chapter 5. Other construction data relative to pile installation and testing are given in Table 2.3.

The construction procedure following the normal good practice that is outlined in Chapter 6, included the preliminary construction specifications, with one exception. The reinforcing cage for Test Pile North was dropped into the grout, and it had to be fished out of the grout and supported slightly off the bottom by steel beams at the surface after

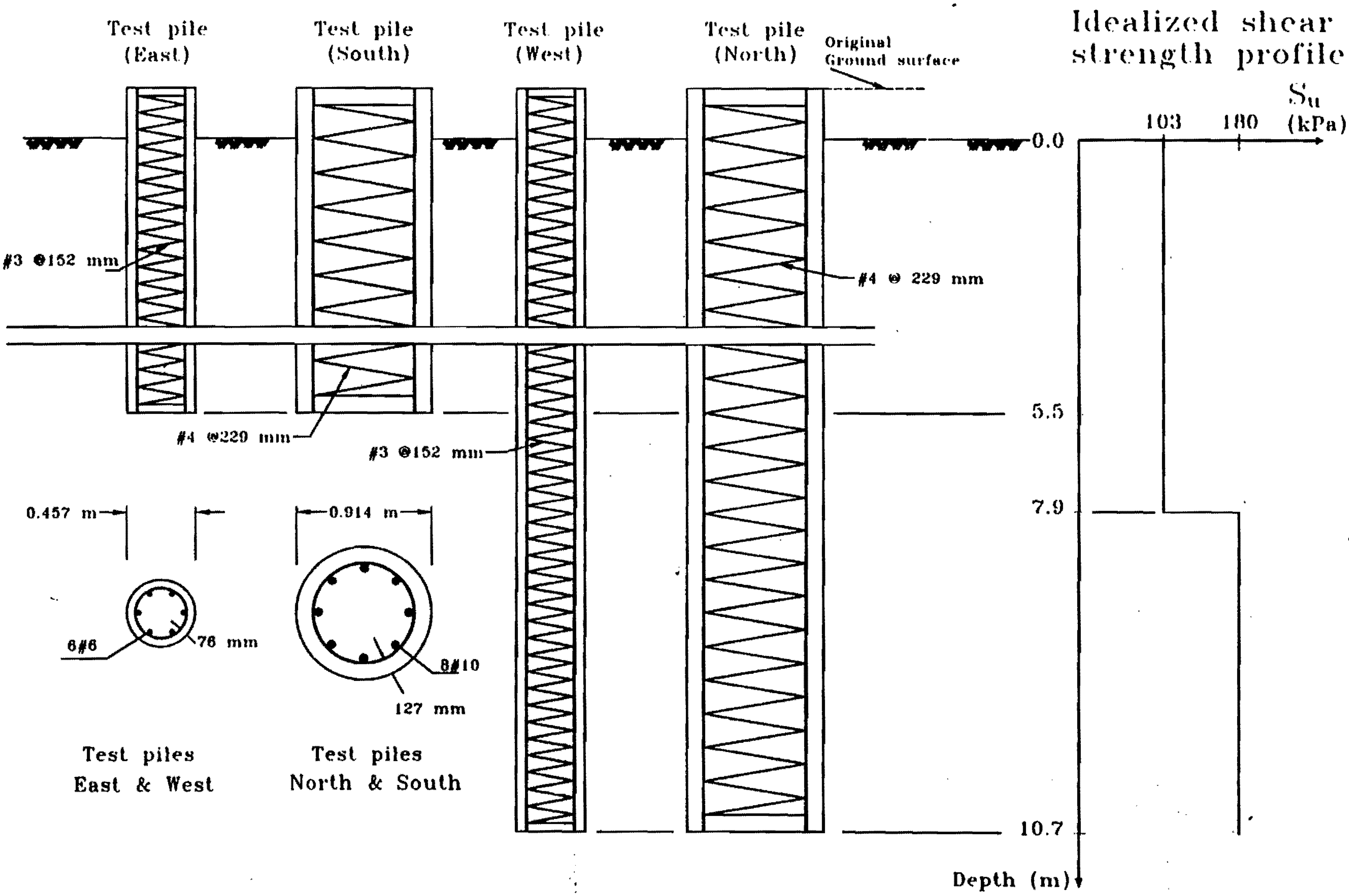

Fig. 2.5. Details of the pile reinforcement and the idealized shear strength profile

Table 2.1. Grout Mix Proportions for the Test Piles

Constituents	**Amount** kN/m^3
Cement	4.38
Sand	12.92
Fly ash	1.31
Water	2.43
Additive (Fluidizer)	0.022
Non-shrink additive	None

Table 2.2. Material Properties of the Grout for the Test Piles

Material property	**Test results**
Setting time	5.5 hours
Shrinkage (ASTM C1090)	0.015 %
Efflux time (ASTM C 939)	33 sec.
Compressive strength (after 28 days) -avg of 12 75-mm cylinders	36.8 MPa
Tensile strength (direct tensile test after 28 days) -avg of 6 75-mm cylinders	1.95 MPa

the auger was withdrawn. In some cases after the reinforcing cages were placed, a visible amount of spoil (clods of clay) fell into the grout columns. The grout columns were not protected by surface casings or sleeves during this process. The spoil appeared to float in the grout columns, and as much of this spoil as possible was removed with "screens" by the contractor's workers. Later integrity testing did not indicate that any defects were produced in the grout by accumulation of spoil. [It is important to specify that this material be completely removed from the grout using screens or other

Table 2.3. Installation of Test Piles

Test Pile South	Test Pile North	Test Pile West	Test Pile East
18 December 96	18 December 96	19 December 96	19 December 96
SD 1210	SD 1315	SD 1638	SD 1712
FD 1224	FD 1420	FD 1655	FD 1728
SG 1235	SG 1430	SG 1656	SG 1730
FG 1245	FG 1445	SG 1707	FG 1736
DT 5 March 97	DT 6 March 97	DT 7 March 97	DT 6 March 97

Notes:
SD Time drilling started
FD Time drilling completed
SG Time grouting started
FG Time grouting completed
DT Date pile tested

appropriate devices before the grout sets up. Some allowance should perhaps be made in the structural design of CFA piles to account for the presence of some small soil clods in the grout if post-installation integrity testing is not performed, since there is no way to assure that all soil clods have been completely removed by the workers using screens].

A schematic arrangement for the CFA pile rig is shown in Fig. 2.6. This rig was a large rig capable of applying up to 119 kN-m (88,000 ft-lb) of torque to assist in penetrating the continuous flight auger into the ground. The torque applied by the rig is crucial to determine the possibility of depressurizing the soil and thereby reducing the pile's resistance. In order to avoid this possibility, Van Impe et al. (1991) recommended that the downward rate of penetration of the auger, v, must be at least as high as the value given by the following equation:

$$v \geq n\, p\, (1 - d_o^2/d^2), \qquad (2.5)$$

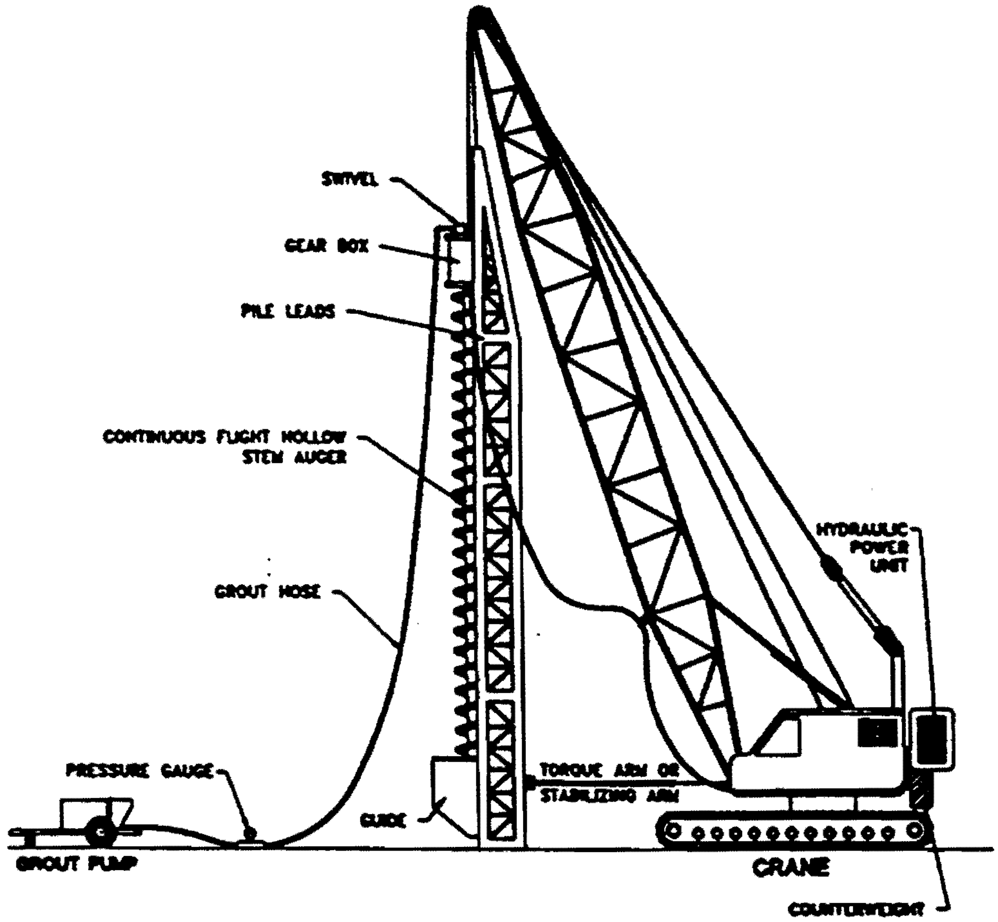

Fig. 2.6. Schematic arrangement for the CFA pile rig, after Deep Foundations Institute (1994)

where v is expressed of units of length per minute,

n = is rate of revolution of the auger (usually expressed in rpm),

p = pitch of the auger (length per turn),

d_o = is the diameter of the stem of the auger, and

d = the outside diameter of the auger from tip to tip of the auger flights.

During the construction of each test pile, the v-value was observed at a penetration approximately equal to half of the pile length. The observed values are listed with other constituents of Eq. (2.5) in Table 2.4.

Table 2.4. Calculated and Observed Rates of Penetration of the Auger

Pile designation	**d (mm)**	**d_o (mm)**	**Auger pitch (mm)**	**n (rpm)**	**v Eq. (2.5) mm/min**	**v Observed mm/min**
East	457.2	114.3	260.4	33	8056	1220
West	457.2	114.3	260.4	40	9765	610
South	914.4	152.4	355.6	16	5690	396
North	914.4	152.4	355.6	20	6914	457

It is clear, from Table 2.4, that the equipment used in the installation of the test piles did not satisfy the specifications of Van Impe's equation. However, the soils at the test site were cohesive and stiff and, based on the loading test results presented later, did not appear to exhibit any tendency to become depressurized or to be "mined" from the sides of the excavations during the test pile installation. This may not have been true had the soils at the site been "running sand."

Typically, in the installation of CFA piles, an analog pressure gauge is attached to either the pump or the grouting line to measure grout pressures. The minimum grout pressure at a given depth should be maintained higher than the total overburden pressure in the soil at that depth to minimize the possibility that the ground will flow into the pile beneath the auger. For that purpose, along with measuring the incremental grout flow vs. theoretical incremental volume, a Pile Installation Recorder^{TM} (PIR) was used by representatives of Pile Dynamics, Inc., during the construction of Test Piles North and West. This monitoring system consists of an auger position indicator attached to the boom line, a magnetic flowmeter and a pressure transducer mounted in the grout pump line adjacent to the grout pump. These components are connected to a control unit which allows real-time data recording and display. The system was easily installed and its use did not impede the construction progress. Photographs of this system are shown in Chapter 6. The operator of the control unit can tell immediately if insufficient grout has been placed at any position along the pile or if the grout pressure has been reduced below the overburden pressure. Permanent records of the minimum and maximum pump grout pressure vs. elevation of the auger tip, and grout volume ratio (grout placed/theoretical volume) vs. elevation of the auger tip can be retrieved and stored in a microcomputer. The PIR results for Test Pile North are shown in Figs. 2.7 and 2.8. Those for Test Pile West were not stored for later printing but were similar to those for Test Pile North. It can be inferred that the grout volume ratio and the pump grout pressure, during the installation of Test Pile North, were adequate except at few locations where the indicated grout volume ratio is slightly less than unity. In this particular study, these reduced values of grout volume ratio may not be significant because, instead of using the magnetic flowmeter, which was not operational on the dates that grout monitoring was performed, Pile Dynamics, Inc., used the pulses of the fluid pressure transducer to sense pump strokes on the grout pump and to convert the number of pump strokes sensed per increment of depth automatically to grout volume placed in each increment of depth by multiplying by the volume of the positive displaced pump. This procedure is less accurate than using a flowmeter, so that in fact there may have been no increments of depth in which the actual grout ratio was less than unity. Note in Figs. 2.7 and 2.8 that

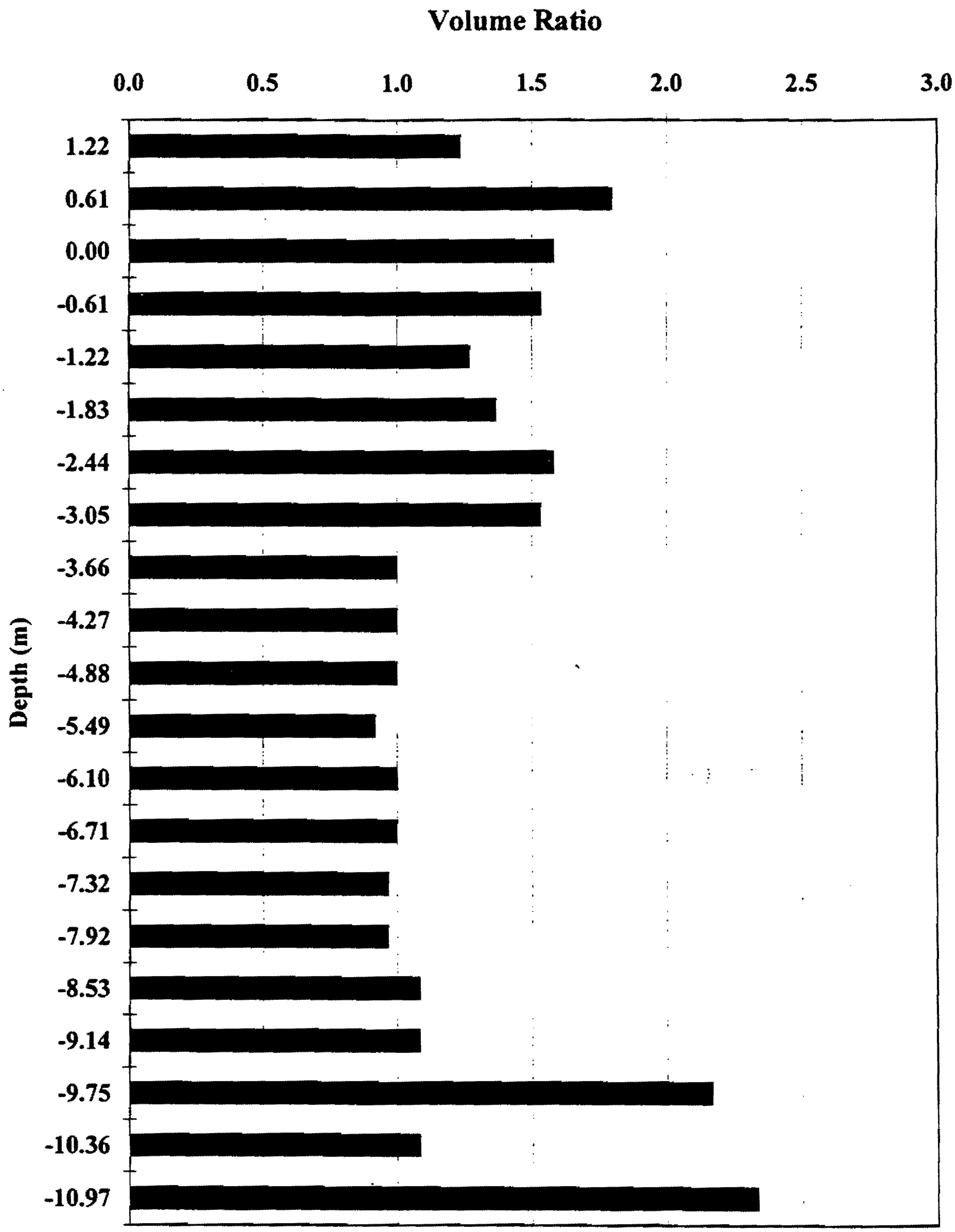

Fig. 2.7. Record of grout volume ratio vs. depth for Test Pile North

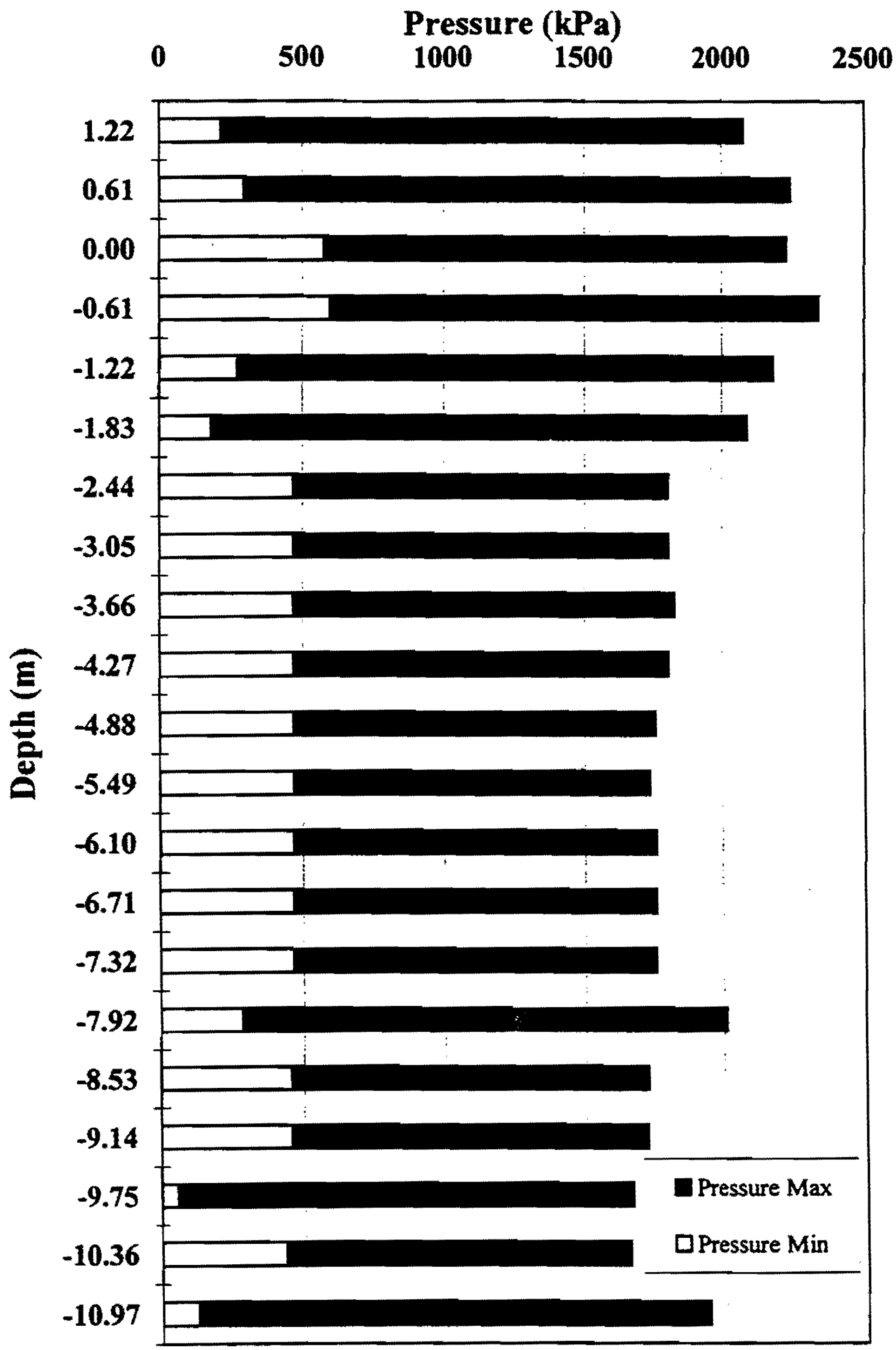

Fig. 2.8. Records of maximum and minimum pump grout pressure vs. depth for Pile Test North

the auger positions are indicated to be positive (1.22 m above the ground surface) at the end of the grout placement. Pumping ended with auger tip at the ground surface, so this represents a small error in position measurement.

The total vertical pressure in the ground is approximately 20 z kPa, where z is the depth in meters. That is, the minimum pump pressure should have been 20 times the depth of the auger tip (grout orifice). At the very bottom of the pile and a depth of 9.75 m (32 ft) in Fig. 2.8, the minimum grout pressures are below this value. Otherwise, all are above 20 z. The grout pressure at the bottom of the pile was low because the operator was reluctant to use higher pressure, which would have forced a considerable amount of grout up the auger at the beginning of pumping. This appears not to have been problematical. The reduced minimum pressure at the depth of 9.75 m is not explainable. There was no indication that this pile performed inadequately under lateral loading; however, the potential deficiency at a depth of 9.75 m was too deep to have had any measurable effect on the structural behavior of that pile under lateral loading. As explained in the next section, the cross-hole logging in the Test Pile North was not effective because the access tubes appears to have debonded from the grout, perhaps as a result of the cage having been dropped and fished out of the grout. This left a thin cake of grout on the insides of the tubes and may have resulted in some separation of water from the grout on the outsides of the tubes, which resulted in decoupling of the source from the receiver. As a result, the existence of a defect at a depth of 9.75 m in Test Pile North could not be confirmed or ruled out.

Integrity testing program

Two types of integrity testing were performed on the test piles. These tests are:

- crosshole and single hole ultrasonic logging, and
- fiber-optic television recording.

The ultrasonic logging was conducted under both wet and set grout conditions. The former was accomplished immediately after the construction of each test pile, and the latter was performed three days after construction. In both cases, cross-hole and single-hole tests were performed. Two tubes were cast in each test pile for the purpose of performing ultrasonic logging. One of these tubes, an ABS tube with a diameter of 48 mm, was also used during the loading tests as an inclinometer casing. The other tube was a standard PVC pipe with a diameter of 32 mm.

The crosshole ultrasonic logging was performed at no cost to the State by Fugro-McClelland Southwest, Inc. The test was conducted by lowering two probes into the tubes, which were water-filled. One of the probes contained a transmitter of acoustic energy at 62 kHz, and the other probe contained a receiver. Ultrasonic signals radiate from the transmitter in all directions. Some signals arrive at the receiver through the pile medium (the grout). If the grout is sound, the delay time from the transmission unit to the receiving unit is small. If there is a defect such as a soil inclusion, the delay time is increased. This appears as a gap in the display of the received signals. The single tube test is similar to the cross-hole test except that both the transmitter and the receiver are placed at the ends of one probe in one tube, with an acoustic isolator in between.

The wet grout tests were not effective in this study. The results of the tests in the set grout appear to be quite effective except for the tests conducted on Test Pile North. For Test Pile North, the results of both the cross-hole and single-hole tests were erratic along the entire length of the pile. This usually occurs when the grout shrinks causing a debonding along the tube wall. The detailed results of ultrasonic logging along all test piles are given in Appendix A.2. However, visual analysis of the logs does not indicate any defects in the other test piles.

In addition to the ultrasonic logging, a recently developed fiber-optic integrity testing technique was used in the reaction pile. The test allows visual inspection of the pile medium along the pile. It was performed by Stress Engineering, Inc., by lowering a fiber-optic television camera in a transparent polycarbonate tube which is attached to the rebar cage before pile construction, similar to the way an endoscope is used in medical applications. The fiber-optic record, which was stored on a videotape, showed no defects along the pile except a micro-crack, about 0.25 mm (0.01 in.) wide, at the bottom edge of the steel collar that was placed at the pile head as a loading reaction. This micro-crack most likely occurred due to shrinkage of the grout in the steel collar and did not affect the performance of the reaction pile. Figure 2.9 shows a photograph of this micro-crack.

Loading test arrangement

The test piles were positioned around the reaction pile, as shown in Fig. 2.3. After construction, the test area was excavated to a depth of 0.30 m. The excavation was performed to remove fill at the ground surface. This assures that the test piles were supported only by the natural soil. Lateral load tests were performed by jacking a test pile and the reaction pile apart using a manual jack system. The system consisted of a hydraulic jack and a portable 70 MPa pump. Loads were measured with an electronic load cell. Both the load cell and the hydraulic jack were placed inside a reaction strut consisting of a 203-mm (8 in.) steel pipe, as shown in Fig. 2.10. The steel pipe was connected to the reaction pile using a pin joint, as illustrated in Fig. 2.10, to rule out the possibility of developing bending moment in the reaction system or at the pile head. Two dial gages were used to monitor the lateral deflection of the test pile and the reaction pile. The dial gages were attached to wood reference frames which were supported in the soil at 3.0 m (10 ft) from the center of the test pile.

Loading procedure

The loading tests were designed to produce bending moments in the piles under zero axial load by applying shear loads 150 mm (6 in.) above the ground surface. Lateral ground-line shear loads were applied in increments monotonically. When the applied ground-line shear load during the test reached a value that would produce a ground-line deflection in the pile that was equivalent to that which was calculated to occur under the design wind load acting at a large distance above the ground on a sound wall panel, y_{dg},

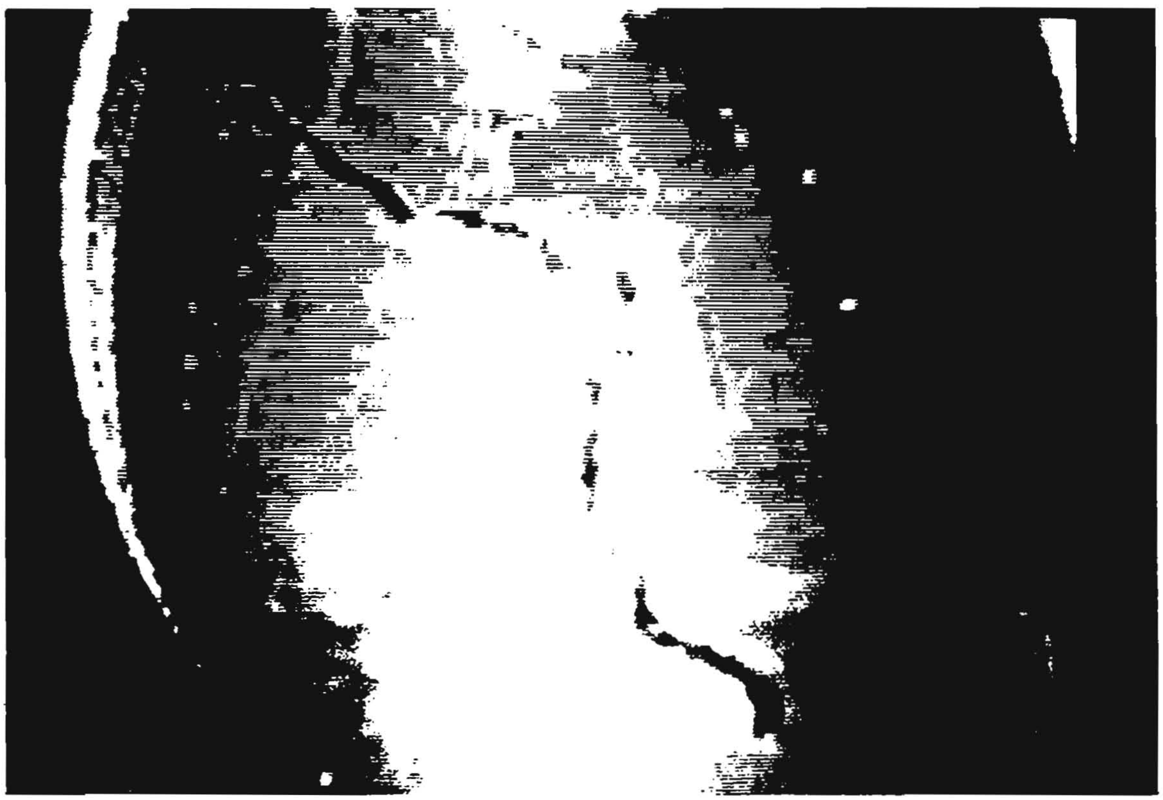

Fig. 2.9. A photograph of a micro crack as recorded by a concreteoscope for the reaction pile

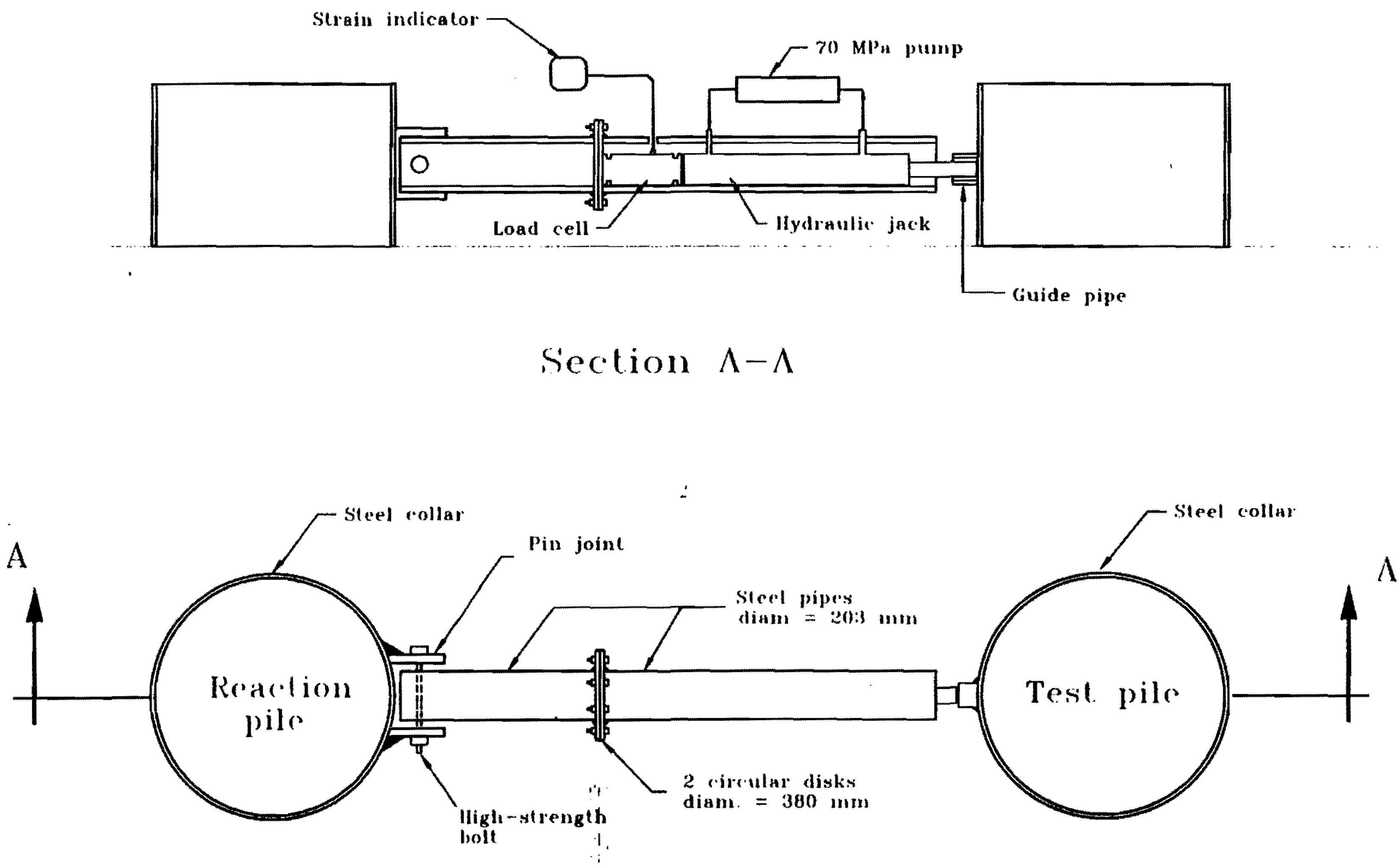

Fig. 2.10. Detail of the loading test reaction system

the load was released and cycled five times to the same value to simulate the effects of buffeting from wind loading. The shear load was then increased in increments until a load equal to about 1.67 times the load that produced y_{dg} was achieved, and five cycles of that load were then applied. After load cycling at that value, the loads were increased monotonically until either the capacity of the jack was reached or a lateral movement of 20 per cent of the pile diameter occurred. Cyclic loading involved reducing the load of the pile to zero and then reapplying the load that had been reached on the previous step. That is, the cyclic loading was one-way. Loading increments can be seen on the graphs that will be referenced later. The time-lengths of the increments were approximately five minutes.

The primary working load for a sound wall is a uniform horizontal wind pressure, which can be computed, according to AASHTO (1989), by the following equation:

$$w = 0.00256(1.3V)^2 C_d C_c \quad , \tag{2.6}$$

where: w = wind pressure in pounds per square foot,

V = wind speed (mph) based upon 50-year mean recurrence interval,

C_d = drag coefficient (1.2 for all sound walls), and

C_c = combined height, exposure and location coefficient.

The load w can be expressed in terms of a ground-line shear, P_t, and a ground-line moment, M_t, acting on a CFA pile, as shown in Fig. 2.11. The ground-line deflection, y_t, associated with P_t and M_t was then calculated by a commercial computer code LPILEplus^{TM} (Reese and Wang, 1995) for a pile of specific dimensions in the soil at the test site. An equivalent ground-line shear, P_{teq}, that induces the same value of y_t (y_{dg}) was considered the design load for the purpose of performing the loading test. The design loads were predicted for the following sound-wall dimensions:

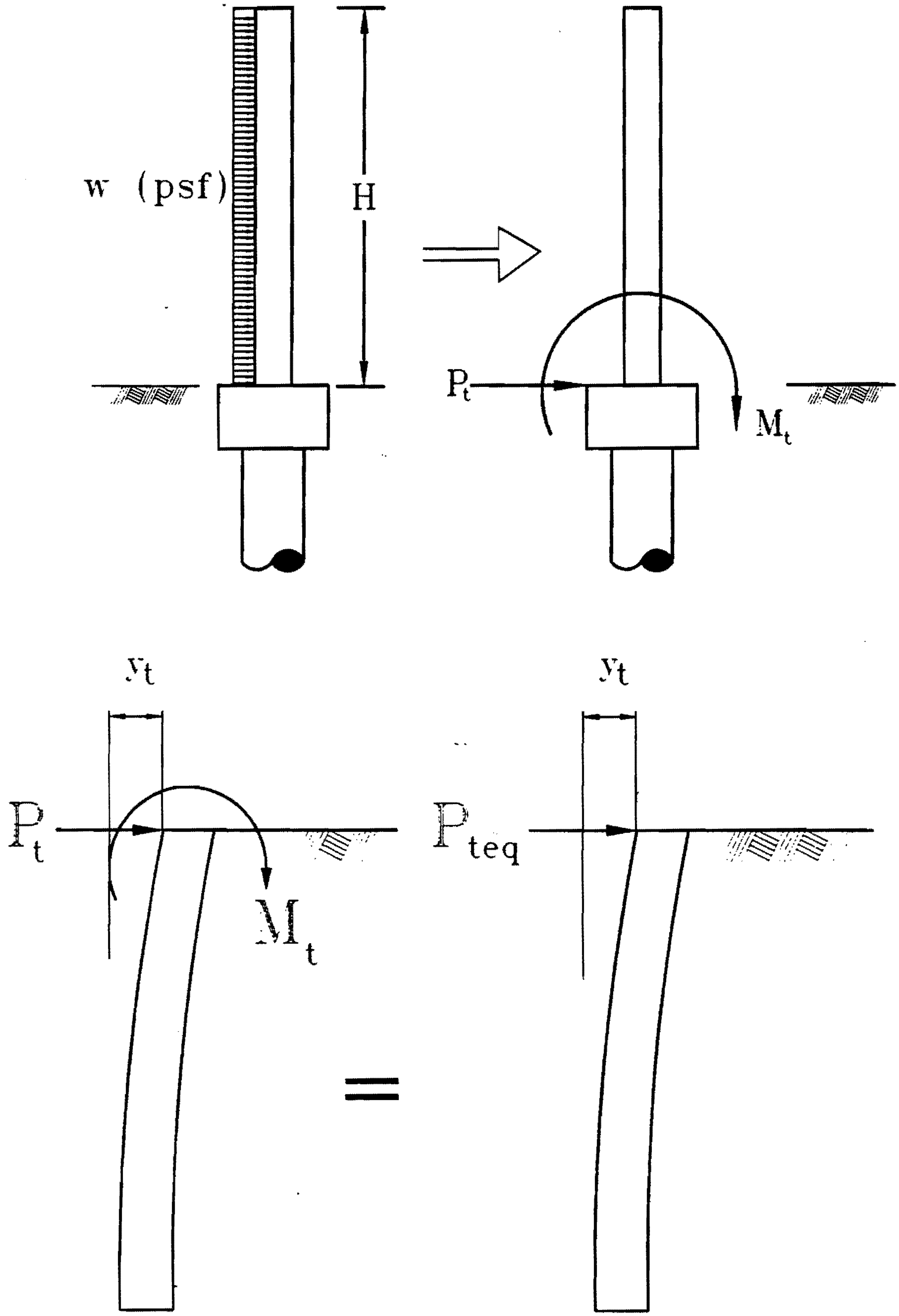

Fig. 2.11. A schematic diagram of the concept of design load determination

- Wall A: span between piles, L, = 6.1 m (20 ft) and height of wall, H, = 6.1 m (20 ft),
- Wall B: span between piles, L, = 6.1 m (20 ft) and height of wall, H, = 6.1 m (30 ft),

For Test Piles East and West, with Wall A, the design load was 62.3 kN (14 kips), while for Test Piles South and North, with Wall B, the design load was 192 kN (43.2 kips). Figure 2.12 illustrates the loading test procedure.

Loading test results

The measured ground-line shear-deflection, P_t-y_t, curves for Test Piles East, West, South and North are shown in Figs. 2.13, 2.14, 2.15, and 2.16, respectively. The profiles of the lateral deflections along the test piles are given in Appendix A. The following are observations regarding the measured P_t-y_t relations:

1- Plastic hinges were formed in both Test Piles East and West. For Test Piles South and North, the mobilized lateral defections were adequate to determine the piles' performance.

2- Increasing the length of the 457-mm (18-in.) pile diameter (Test Pile East) did not influence behavior. The ultimate lateral capacity of the longer pile (Test Pile West) was slightly less than that of Test Pile East, as shown in Fig. 2.17. This is probably due to the variation in the grout strength since the elastic portions of the P_t-y_t curves for both piles were identical. The variation is appreciable only near the ultimate strength of the piles.

3- The pile length had a significant effect on the cracking load and cracking deflection of the 914-m (36 in.) piles (North and South). This can be clearly seen in Fig. 2.18. It should be noted, however, that the P_t-y_t response of both piles, after the second set of loading cycles, was essentially identical.

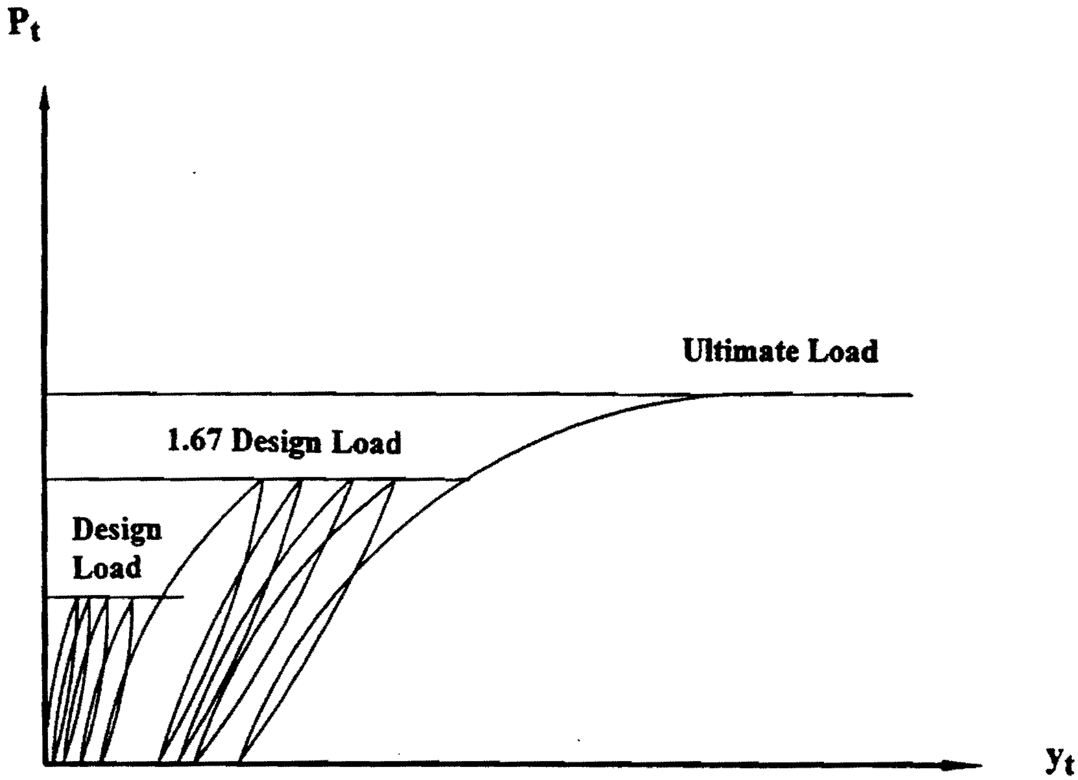

Fig. 2.12. Loading sequence

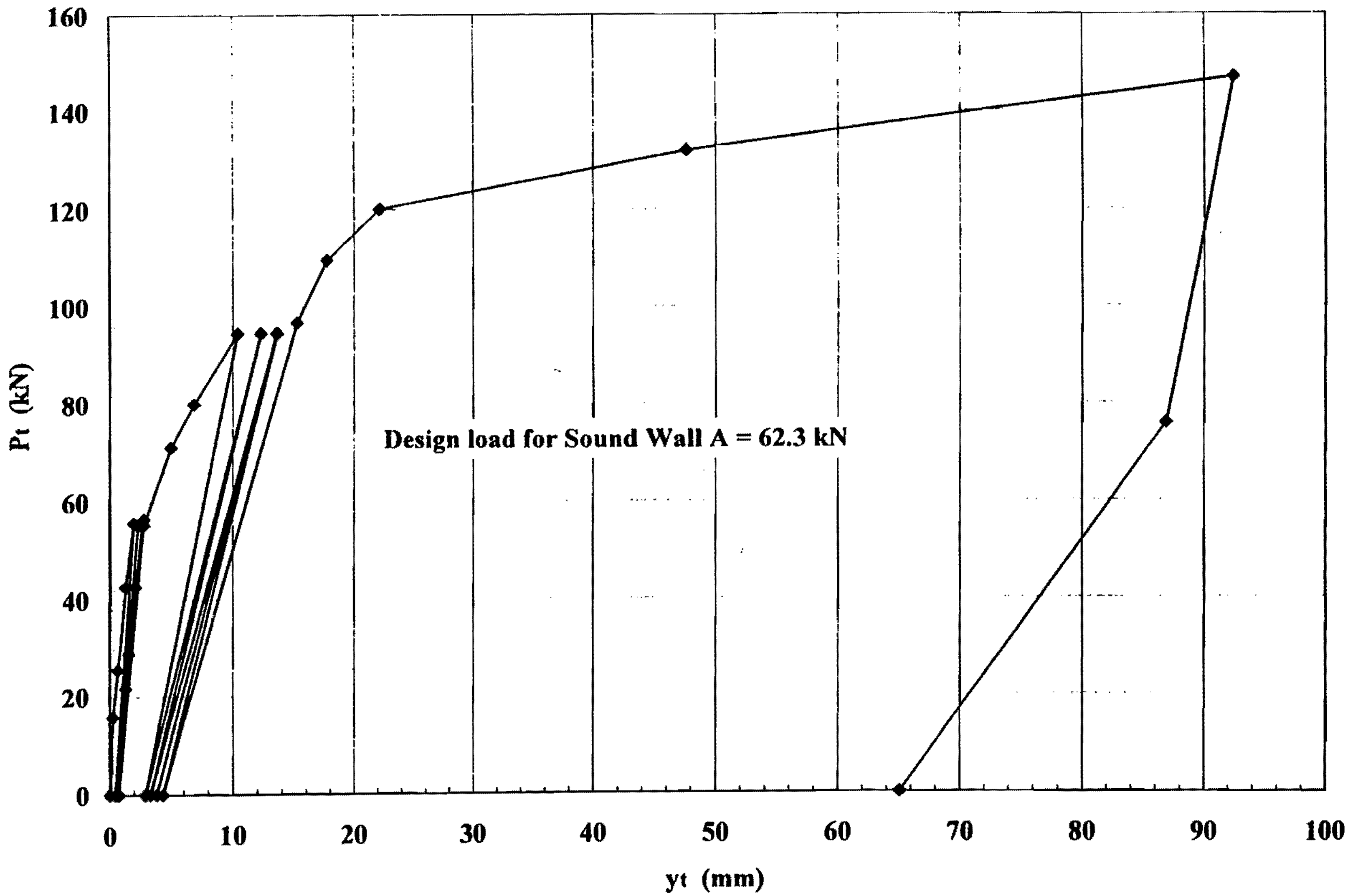

Fig. 2.13. Lateral load-defelection curve for Test Pile East (Diameter = 457 mm and Length = 6.1 m)

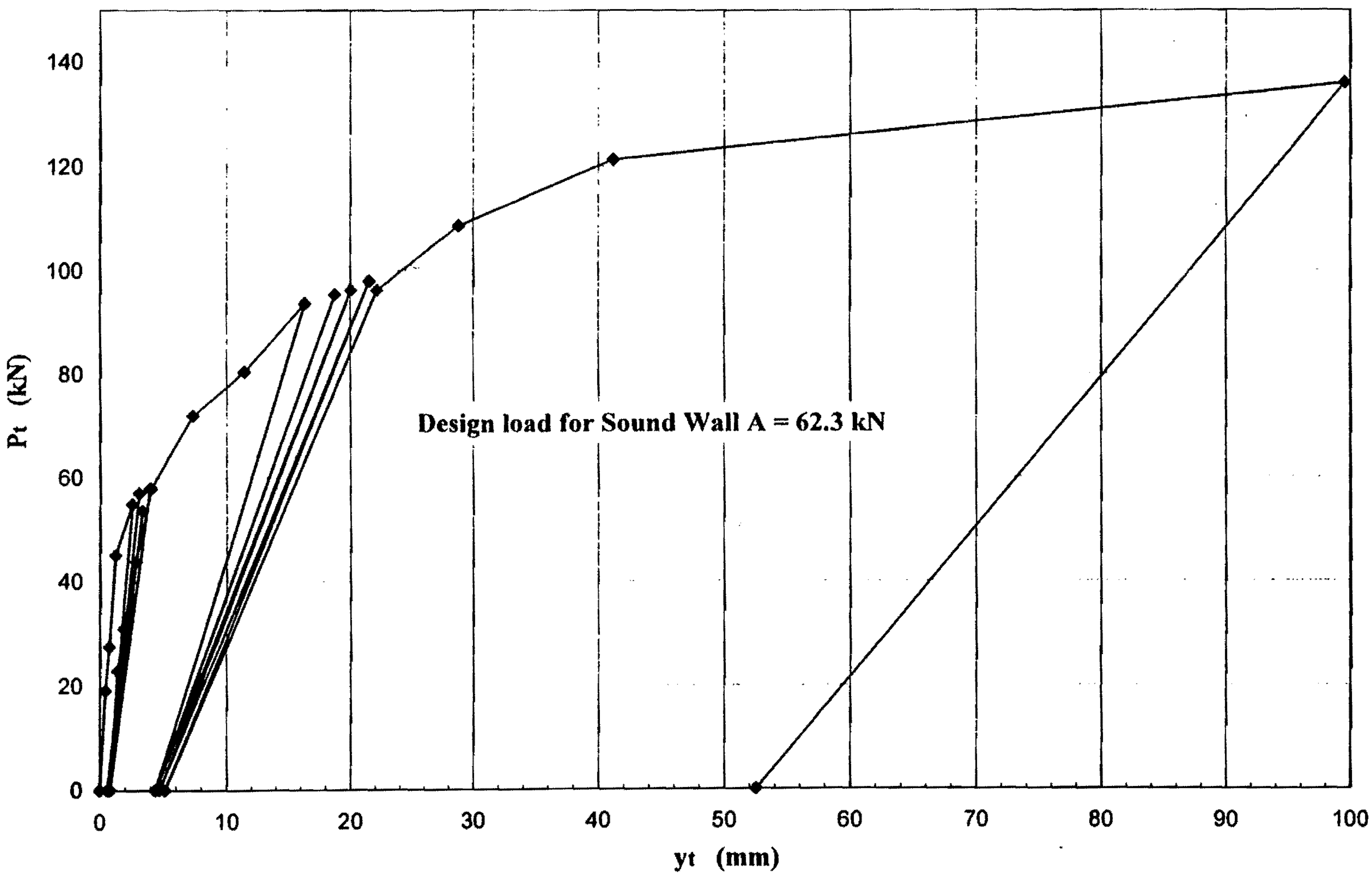

Fig. 2.14. Lateral load-defelection curve for Test Pile West (Diameter =457 mm and Length= 10.67 m)

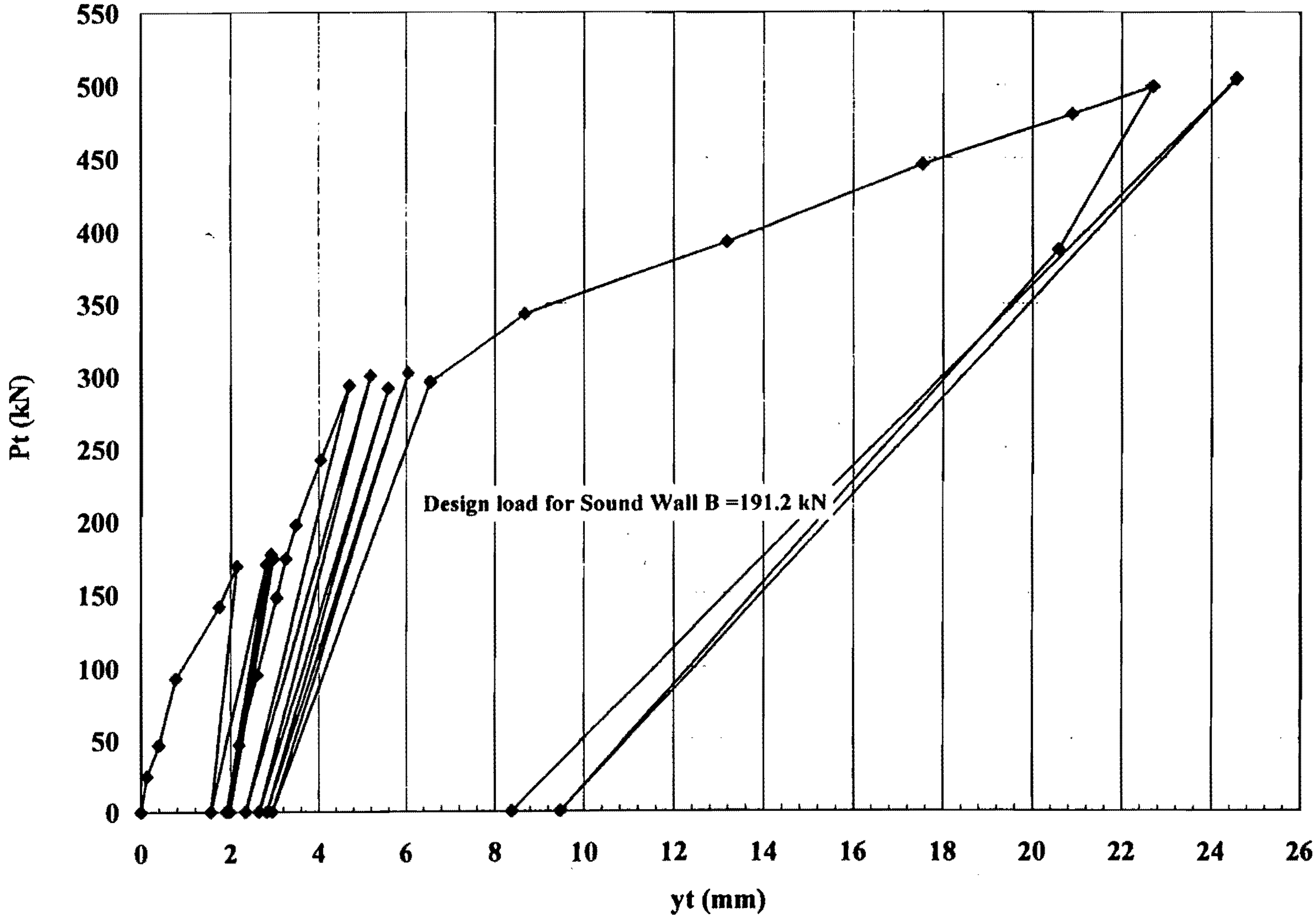

Fig. 2.15. Lateral load-deflection curve for Test Pile South (Diameter = 914 mm and Length = 6.1 m)

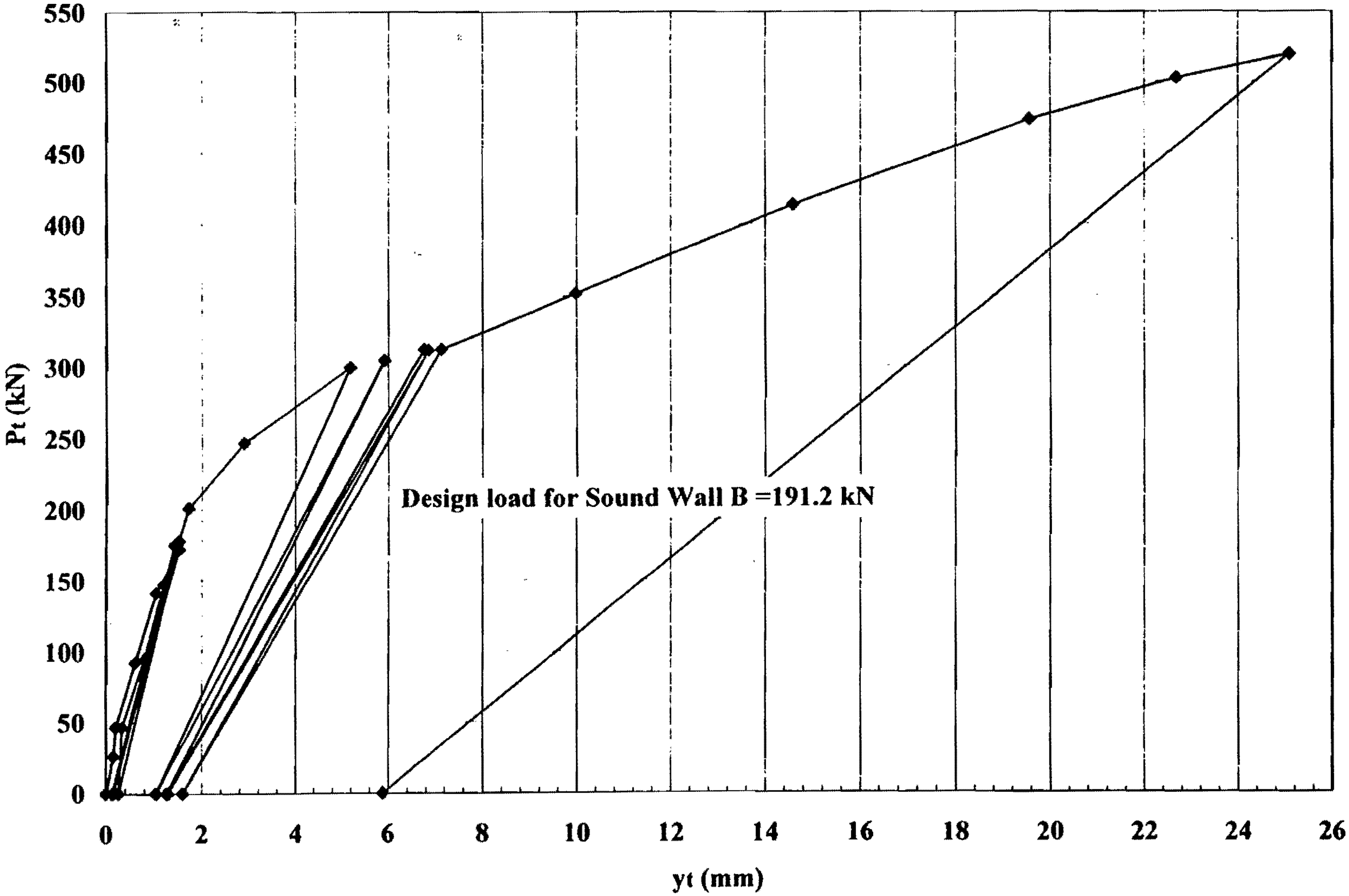

Fig. 2.16. Lateral load-deflection curve for Test Pile North
(Diameter = 914 mm and Length = 10.67 m)

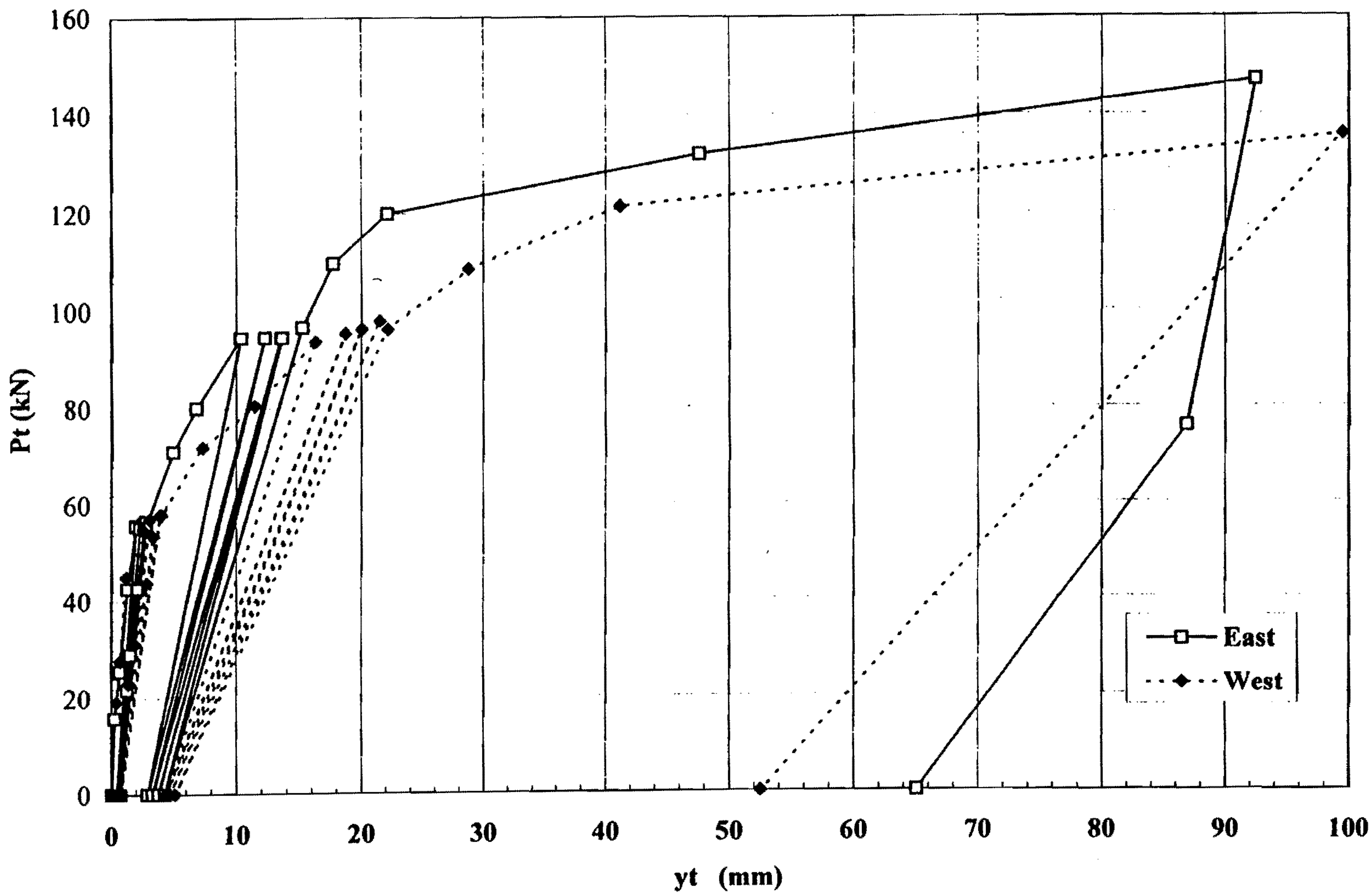

Fig. 2.17. Comparison of the lateral load-defelection curves for Test Piles East and West

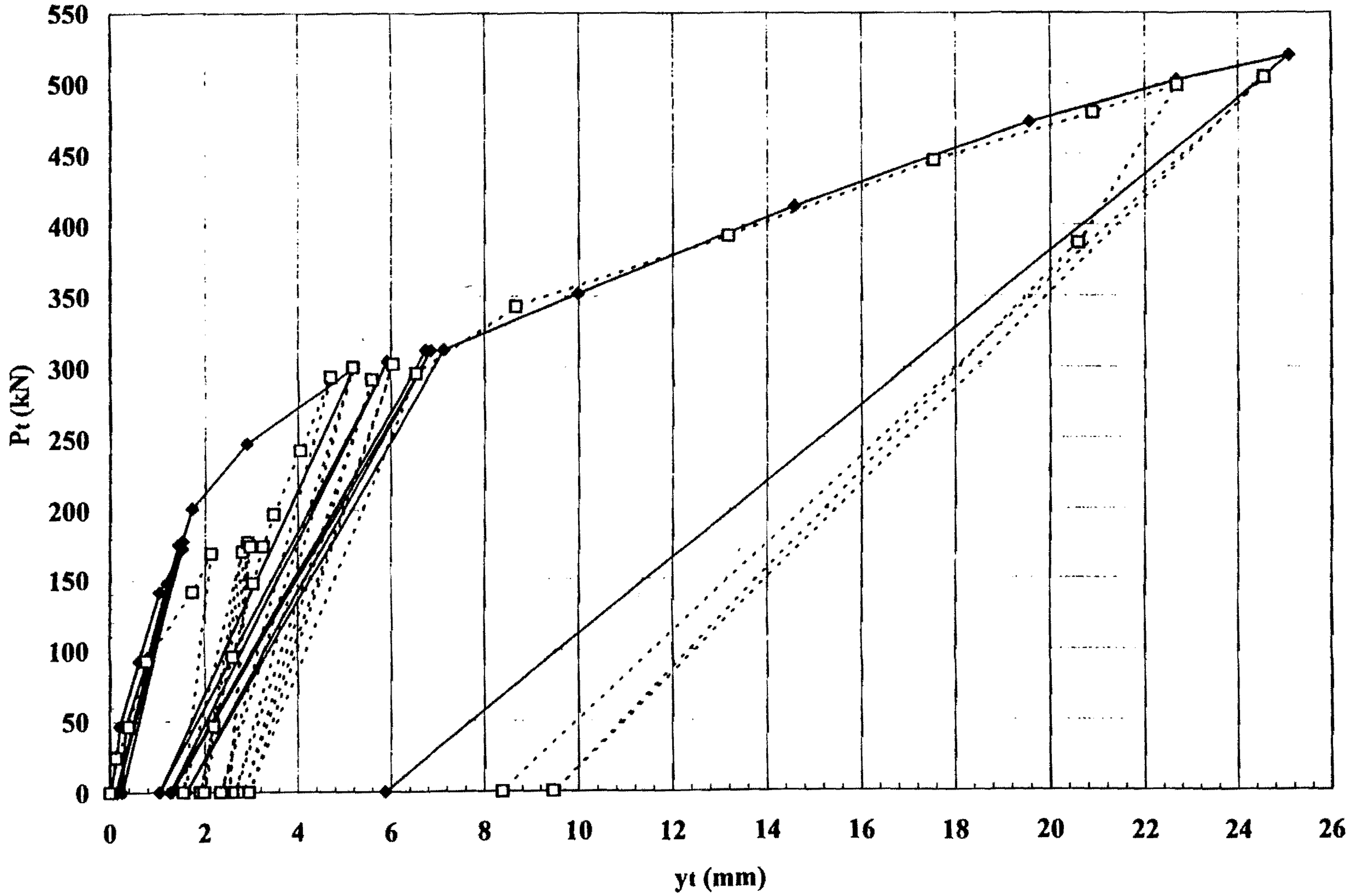

Fig. 2.18. Comparison of lateral load-deflection curves for Test Piles South and North

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CHAPTER 3: ANALYSIS OF THE LOADING TEST RESULTS

Rational analyses of laterally loaded piles are based on the concept of subgrade reaction, in which the pile behavior is determined by solving the following differential equation:

$$EI\ d^4y/dx^4 + P_x\ d^2y/dx^2 - p - w = 0\ , \tag{3.1}$$

where

P_x = axial load on the pile,

y = lateral deflection of the pile at depth x along the length of the pile,

p = soil reaction per unit length,

EI = pile flexural rigidity, and

w = distributed load along the length of the pile, if any.

The resistance of the soil is a nonlinear function of displacement and can be expressed by relations between p and y called p-y curves. The introduction of nonlinear p-y curves into Eq. (3.1) requires a numerical solution, which in turn require a computer program, such as the finite-difference-based code LPILE [Reese and Wang (1995)]. This software can be used to analyze all types of laterally deep foundations in multilayer soil conditions provided the p-y curve for each layer is known. In this study, LPILE was used to reproduce the observed behavior of test piles by varying the p-y curve inputs. This deconvolution process began with the set of p-y curves developed by Welch and Reese (1972) for drilled shafts in stiff clay, which are given by the following relationship:

$$p\ /\ p_u = 0.5\ (y/y_{50})^{0.25}\ , \tag{3.2}$$

where

p_u = the ultimate soil resistance

$$= (3 + \gamma\ x / s_u + x/2D)\ s_u D \qquad \leq 9\ s_u D\ , \tag{3.3}$$

γ = average effective unit weight from the ground surface to the depth of the p-y curve,

s_u = average undrained shear strength from the ground surface to depth x,

D = pile diameter

$$y_{50} = 2.5\ \varepsilon_{50}\ D\ \text{, where} \tag{3.4}$$

ε_{50} = the strain corresponding to one-half the maximum principal stress difference in UU-triaxial compression test.

The effect of cyclic loads is considered by associating the p values calculated by Eq. (3.2) with a revised value of y for cyclic loading (y_c) determined by the following equation:

$$y_c = y_s + y_{50}\ C\ \log N \tag{3.5}$$

where

y_c = soil deflection under N cycles of load at a given value of p,

y_s = soil deflection under static load at a given value of p,

C = empirical parameter = $9.6\ (p/p_u)^4$, and

N = number of cycles of load application.

The Reese-Welch p-y criterion is particularly appropriate as a reference for the CFA pile tests at the NGES-UH since it is based upon the back-analysis of cyclic lateral loading tests upon a 0.76-m- (30-in.-) diameter drilled shaft in the Beaumont formation at a former TxDOT test site in the interchange of SH 225 and I-610 near the University of

Houston. The Reese-Welch p-y curves were used, through LPILE runs, to reproduce load-deflection response at the test-piles heads for N=1 up to the first cyclic loading point and N=5 thereafter. The computed and observed curves are shown in Figs. 3.1 through 3.4 for Test Piles East, West, South and North, respectively. The observed curves are envelopes to the measured curves shown in Fig. 2.13 through 2.16. It can be seen that the computed curves provide reasonable predictions of the lateral deflections observed for the 457-mm- (18-in.-) diameter test piles (Piles East and West). However, they overpredict observed deflection for the 914-mm- (36-in.-) diameter piles (Piles South and North). The reason for the good match for the smaller piles and the poor match for the larger piles is unclear, but it may be associated with the difference in construction methods for the CFA piles and the drilled shaft tested by Reese and Welch. For example, the CFA piles took less time to construct, and the maintenance of high ground pressures near the surface while constructing the larger CFA piles might have been more effective than in the construction of the drilled shaft, in which the soil underwent relaxation prior to concreting. The differences may also be due to the fact that Reese and Welch based their diameter effects upon earlier tests on driven piles in clay formations other than the Beaumont formation, since they tested only one drilled shaft, which may not be strictly valid, even for drilled shafts, in the Beaumont formation.

Whatever the reason for the differences, it is necessary to introduce a different mechanism for describing the effect of pile diameter in the Reese-Welch p-y criterion than appears in the original criterion. This was accomplished by conducting a parameter study using LPILE, in which approximations of the stress-strain properties of the grout in the test piles were used to model the nonlinear bending in the CFA piles, and in which the Reese-Welch p-y curves were modified by modifying p_u, the ultimate value of p. The modification was made in p_u, rather than in y_{50}, or some other parameter, because it appeared that the greatest errors in predicted pile-head movements occurred at the highest loads. The most appropriate formulation for p_u was selected by matching both y_t and the shape of the y vs. x relation for several selected values of P_t for each pile, but giving more

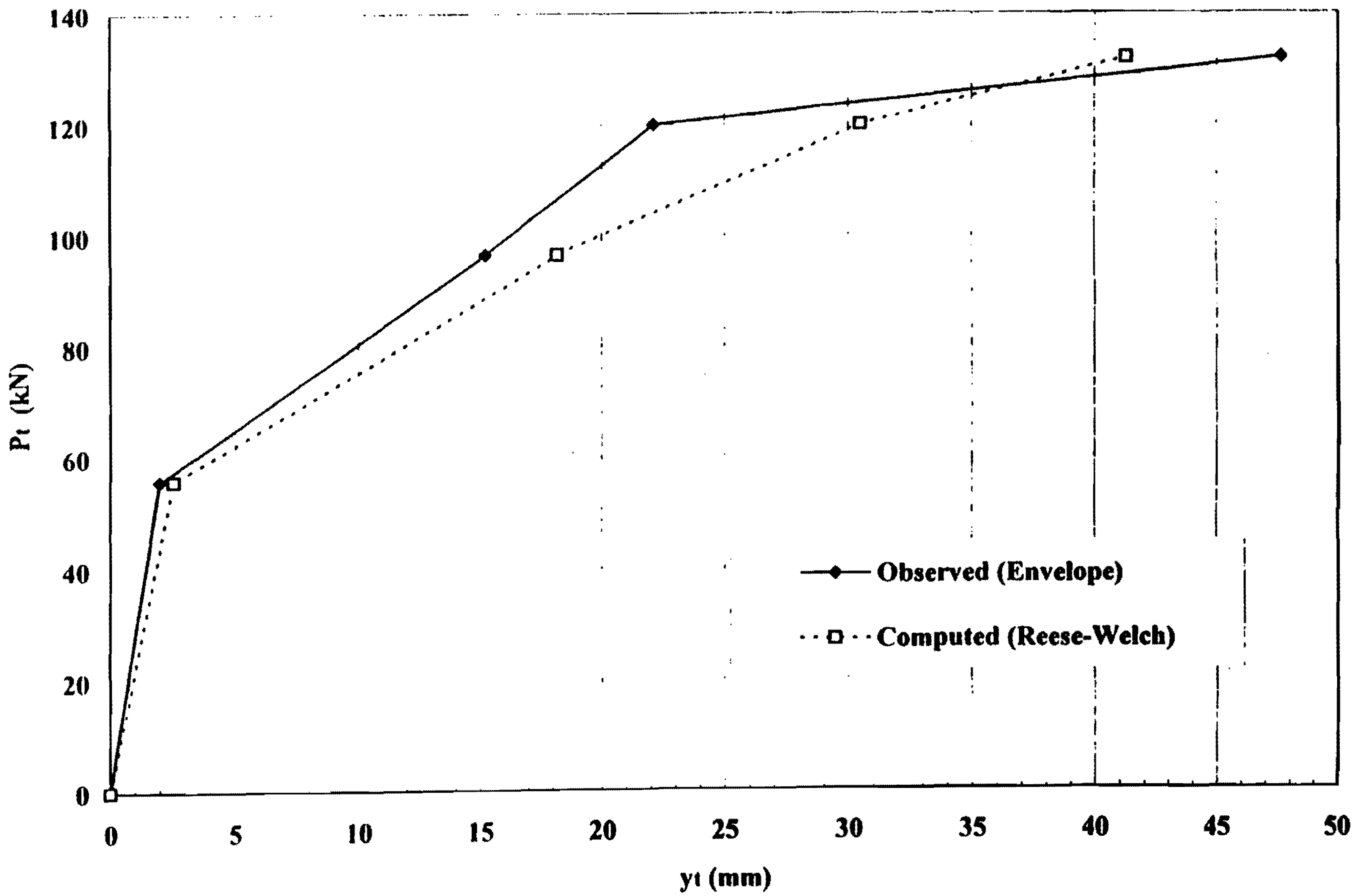

Fig. 3.1. Observed and computed load-deflection curves for Pile East

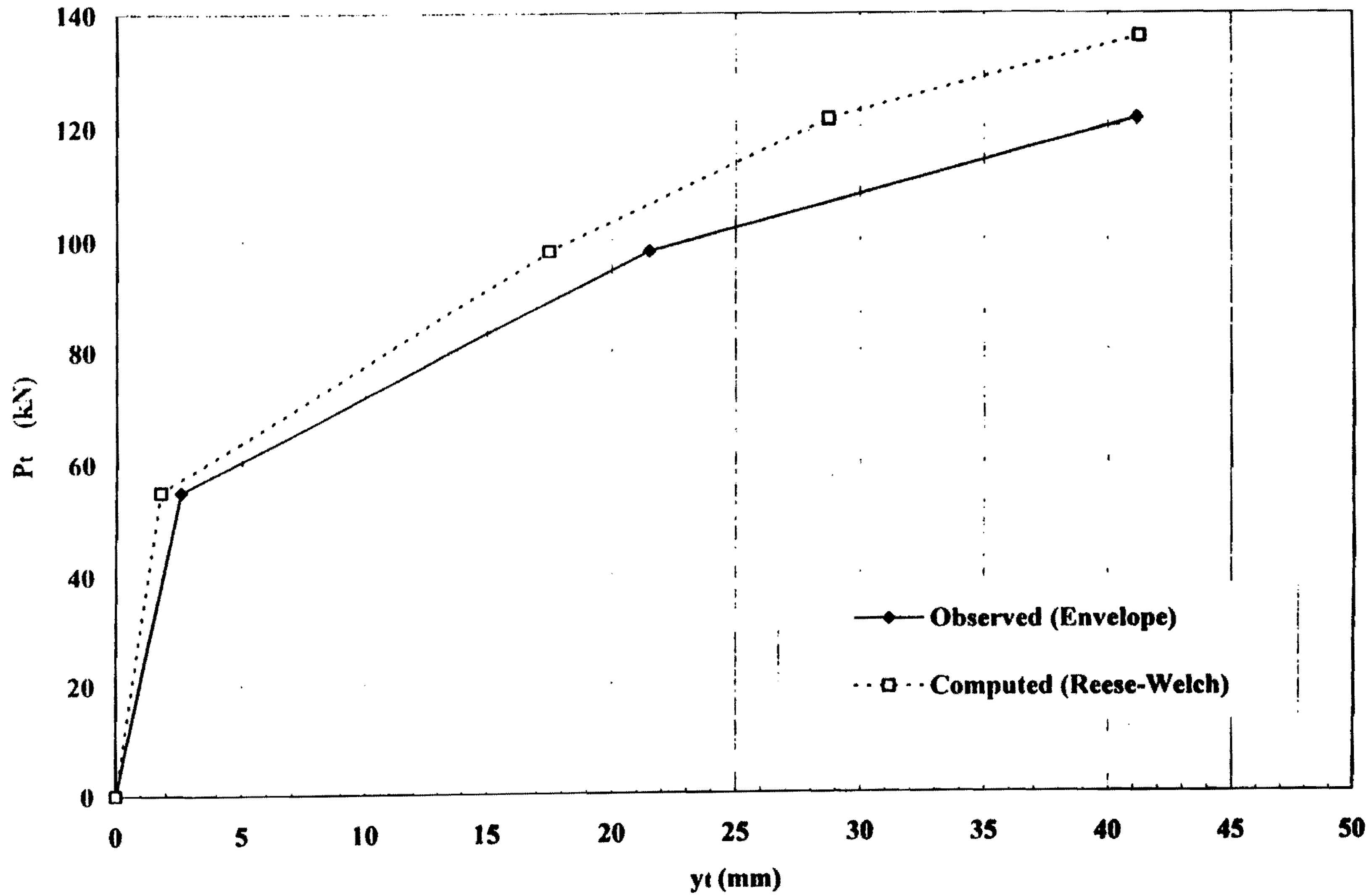

Fig. 3.2. Observed and computed load-defelction curves for Pile West

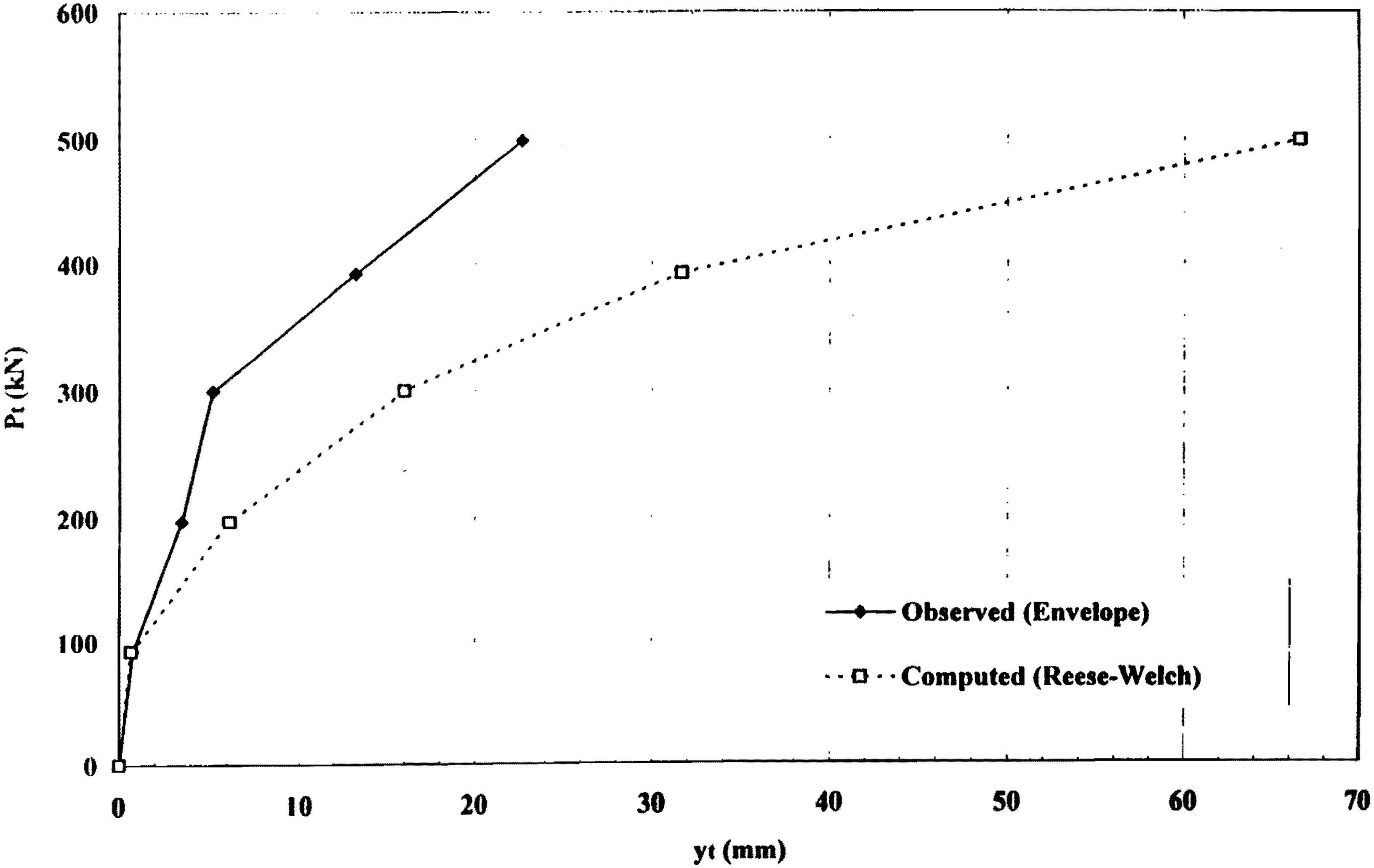

Fig. 3.3. Observed and computed load-deflection curves for Pile South

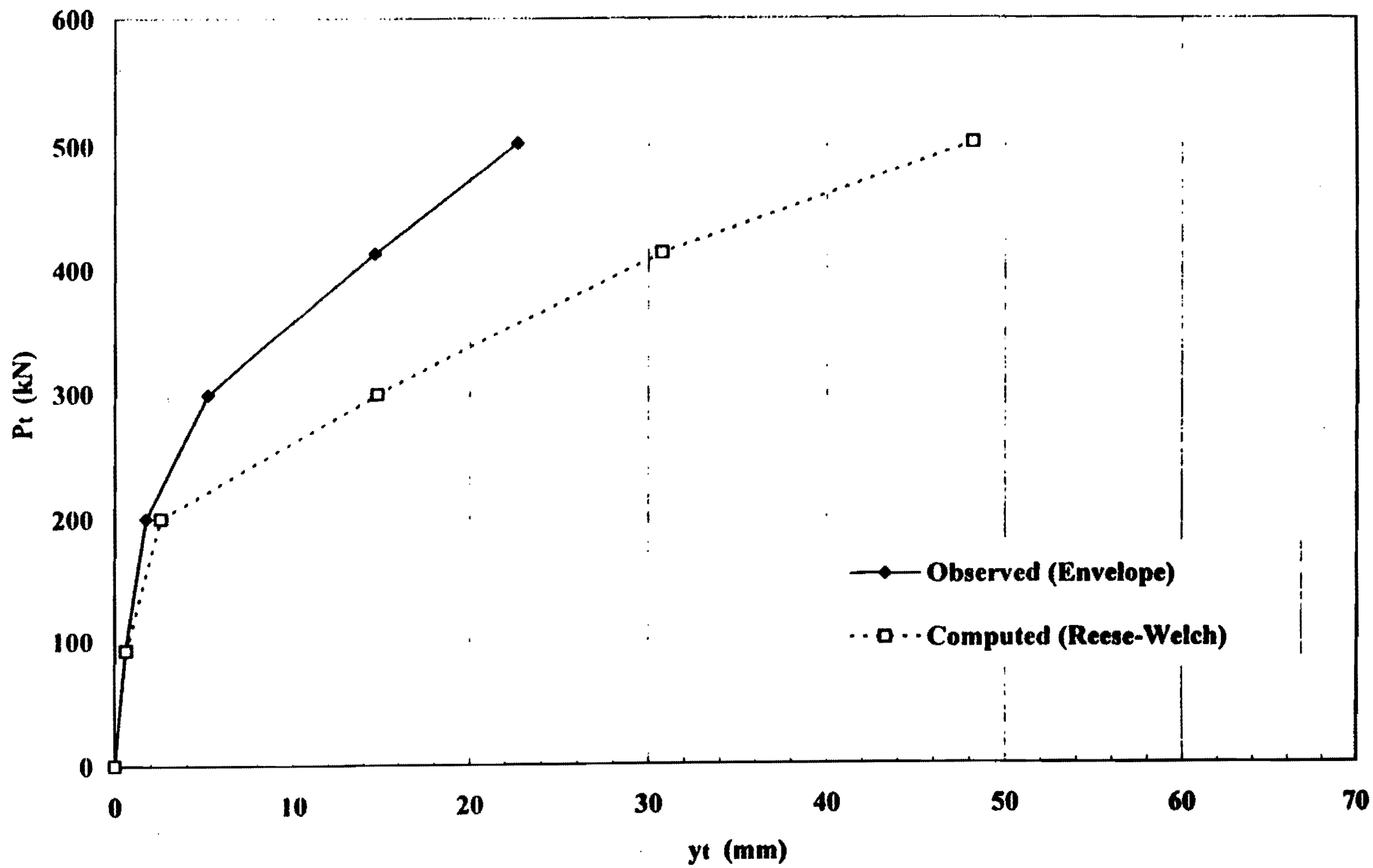

Fig. 3.4. Observed and computed load-deflection curves for Pile North

weight to matches in Test Piles South and East, which were the shortest piles and whose behavior was most strongly dependent upon the exact formulation of the p-y curves.

The result of this parameter study can be expressed by means of a multiplier for p_u in the Reese-Welch criterion. That multiplier, termed ζ_r, is expressed in the following:

$$p_u = [(3 + \gamma' x / s_u + x/2D) s_u D] \varsigma_r \quad \leq 9 s_u D \quad , \tag{3.5}$$

where

$$\varsigma_r = [\, 1 + 0.1\, s_u\, x \,/\, (3\, s_u D + \gamma D\, x + 0.5\, s_u\, x)\,]\,[\, 1.5 - 1.1\, (D_r - D)/D_r\,] \quad , \tag{3.6}$$

where D_r = reference diameter = 0.914 m (36 in.).

The modified p-y relations were then used to synthesize the observed P_t-y_t relations and profile of deflection along the test piles using LPILE. The computed and observed P_t-y_t responses are shown in Figs. 3.5 through 3.8 for Test Piles East, West, South and North, respectively. It can be seen that the modified p-y curves provide improved agreement with observed the P_t-y_t relations for the test piles. These modified p-y curves were utilized in the development of the simplified design method, described in Chapter 4.

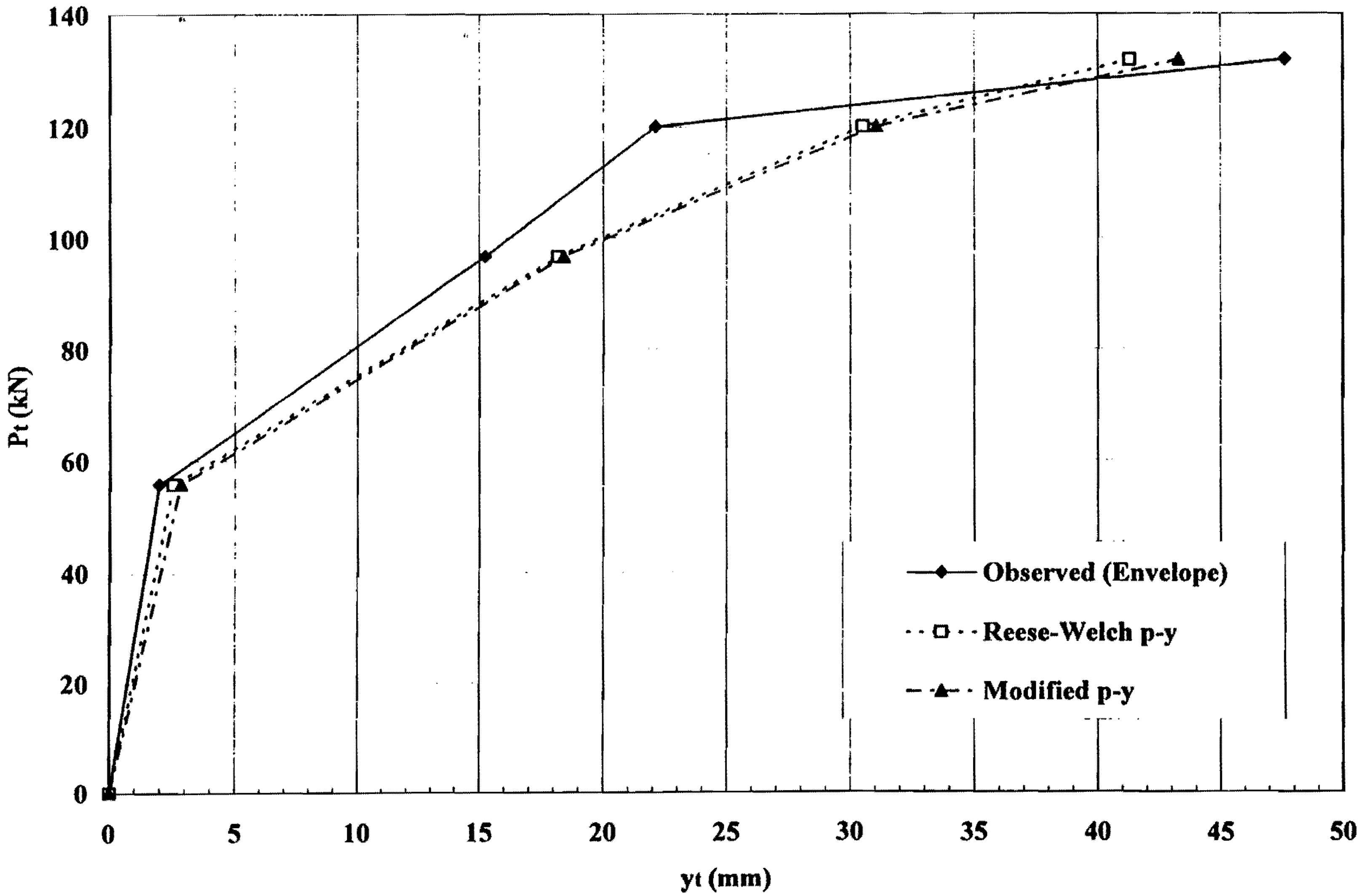

Fig. 3.5. Comparison of computed load-deflection curves for Pile East

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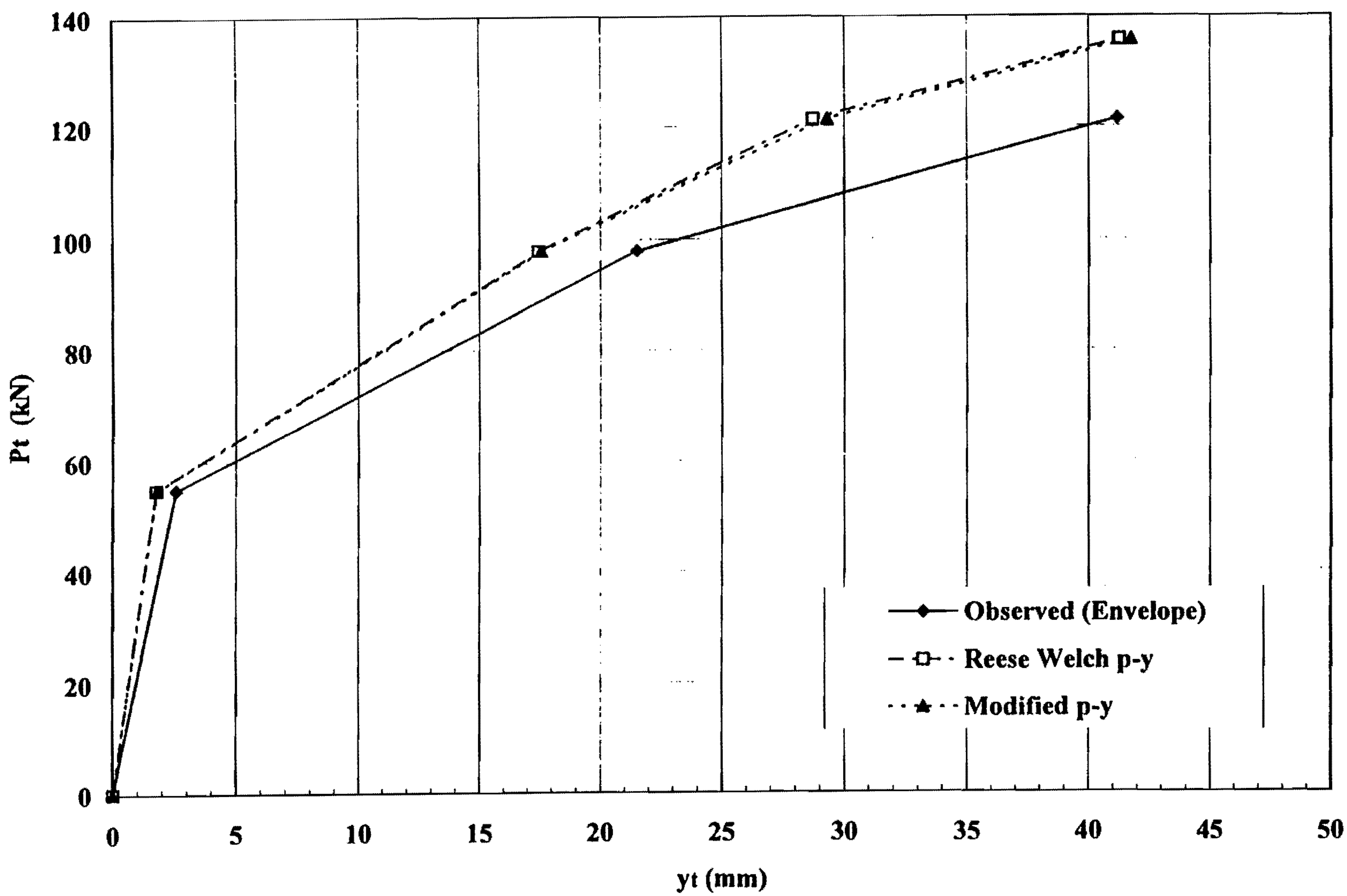

Fig. 3.6. Comparison of computed load-defelction curves for Pile West

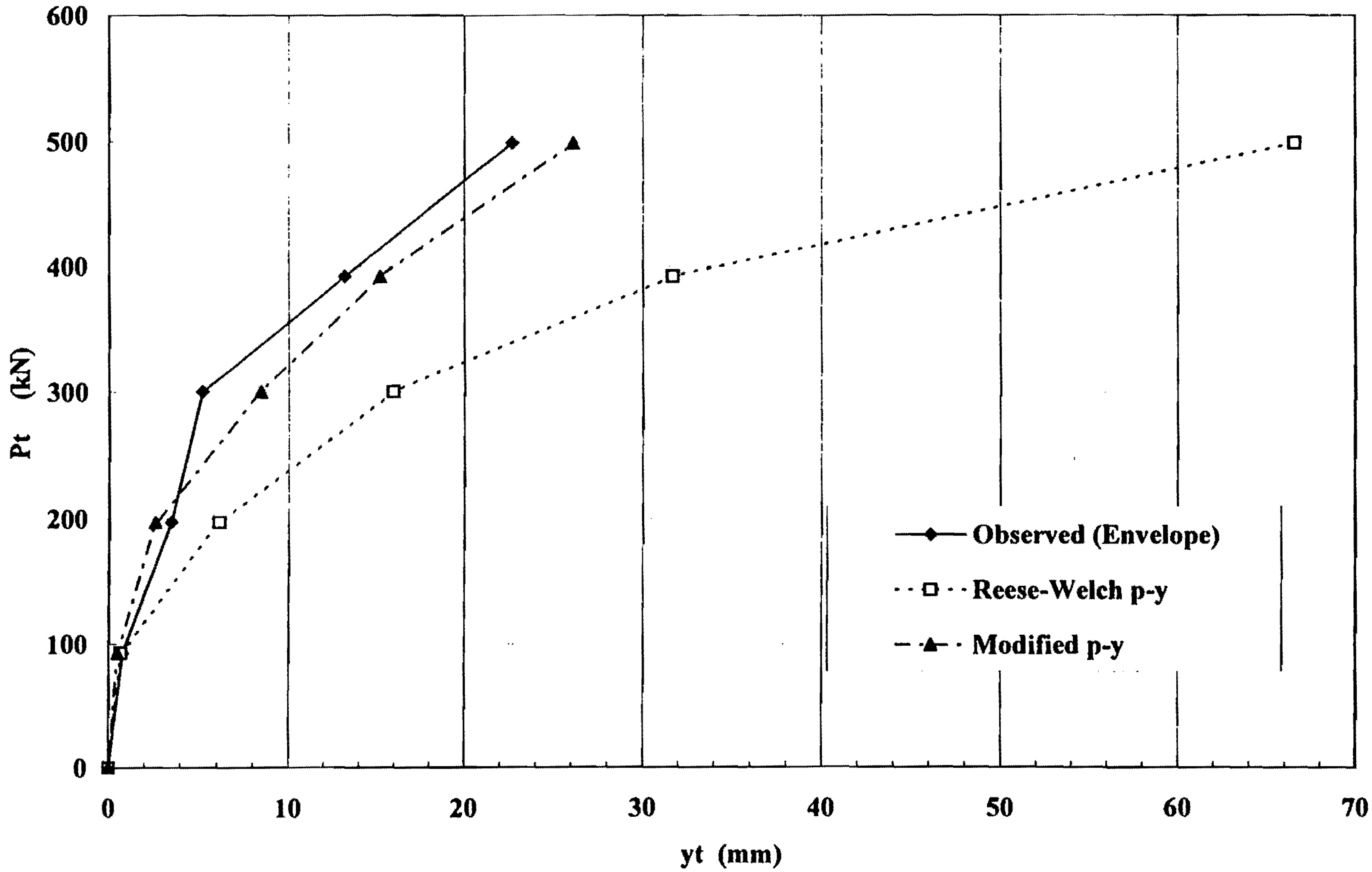

Fig. 3.7. Comparison of computed load-deflection curves for Pile South

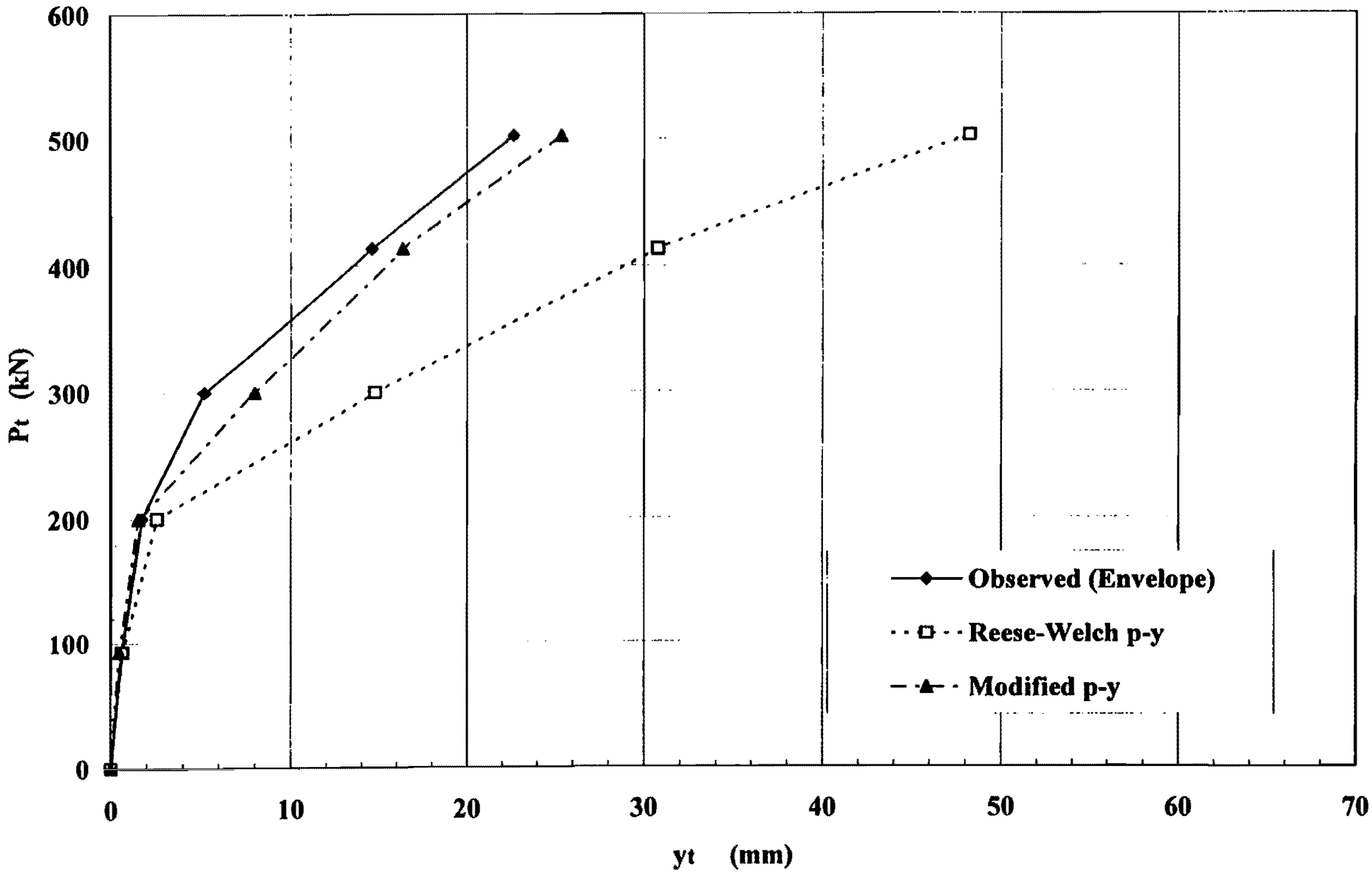

Fig. 3.8. Comparison of computed load-deflection curves for Pile North

CHAPTER 4: SIMPLIFIED DESIGN METHOD FOR CFA PILES SUPPORTING SOUND BARRIERS IN CLAY SOIL

General

This chapter presents a simplified design method for CFA piles subjected to combined ground-line shear load and moment loading. The design method is an extension to the "characteristic load method" developed by Duncan et al. (1994). The recommended design method can be used as an alternative to p-y analyses when the soil near the top of the pile (4-5 diameters) is clay and the pile has a free-head condition. It has, however, an advantage over the original characteristic load method because it has the ability to determine the cracking loads. This is crucial for the design of CFA piles since the tensile strength of the grout is often substantially less than that of concrete in drilled shafts. As a result, the cracking loads of laterally loaded CFA piles will be less than those of comparable drilled shafts. Numerical analyses were thus needed to determine the cracking loads for CFA piles. For that purpose the program LPILE was modified to account for the reduction in the tensile strength of the grout. Based on the results presented in Table 2.2, the tensile strength of grout was taken as 5 per cent of the associated compressive strength. The modified p-y curves presented in Chapter 3 were utilized in the analyses. Figure 4.1 shows a summary of results of the numerical analyses in term of relationships between the cracking load and pile diameter for soils with s_u = 70 kPa to 172 kPa (10 psi to 25 psi). In addition to determination of the cracking loads, the method can be used to determine the ground-line deflections and the maximum moment due to lateral shear and moment applied at the ground line. The minimum penetration of a CFA pile can be also determined.

Design method inputs

Before proceeding to the equations required to simulate the y_t-P_t curve of the CFA pile in clay soil, the user inputs are enumerated. They are:

- Diameter, D, of the CFA pile.

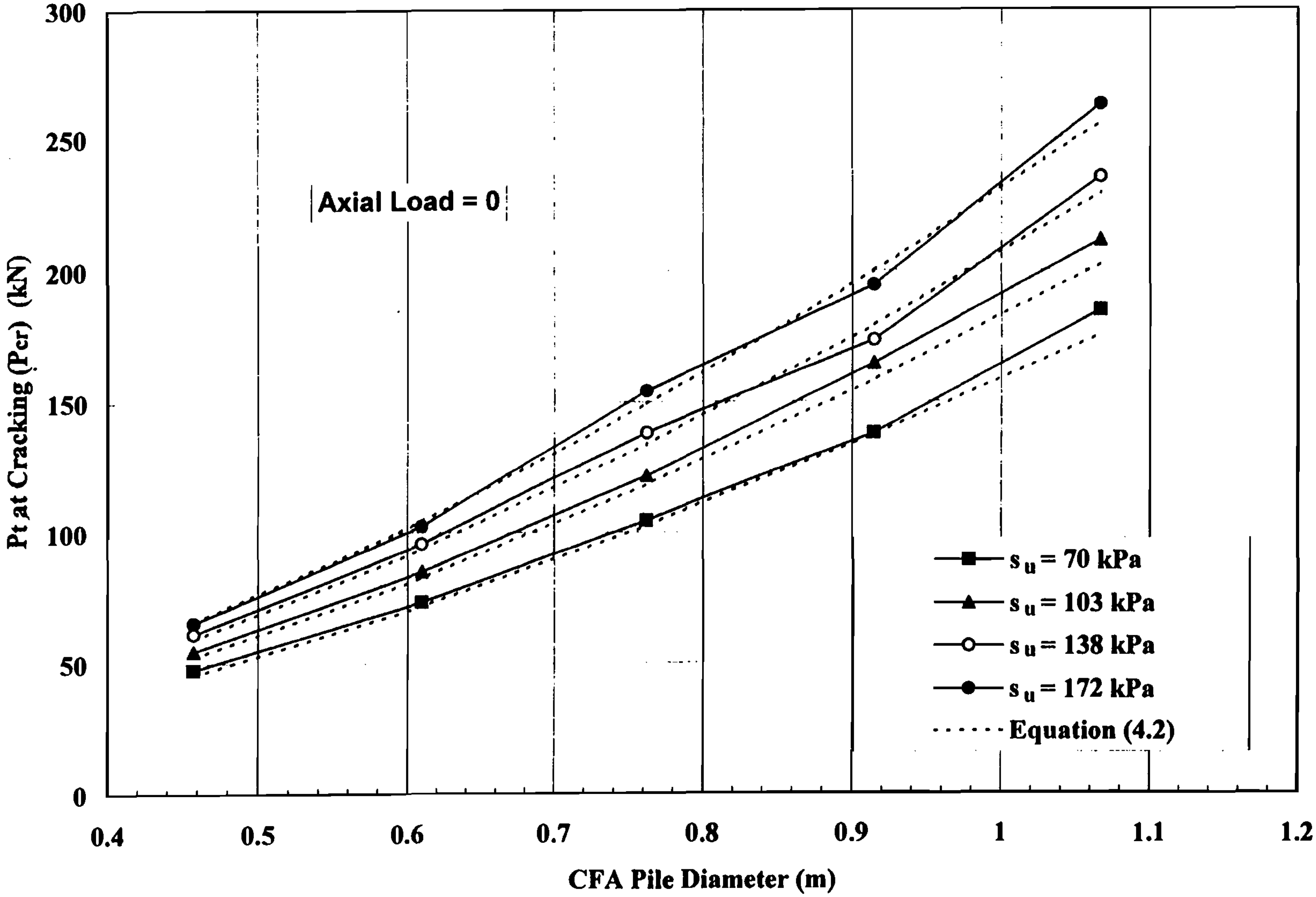

Fig. 4.1. Relationship between ground-line shear at cracking, pile diameter and s_u

- The undrained shear strength, s_u, and the modulus, E_s, of the clay.
- The compressive strength, f'_c, and elastic modulus, E of the grout.

Some commentary on these inputs is in order. First, the values of f'_c that are used for the grout should be related to the strengths that are determined on mini-cylinder samples of 75 mm (3 in.) diameter and 150 mm (6 in.) height. If 50-mm cubes are tested, the results should be reduced to those for 75-mm-diameter cylinders. This can be done, based on the results summarized in Chapter 5 by using the following simple relation:

$$f'_c \text{ (75-mm-cylinder)} = 0.91\, f'_c \text{ (50-mm cube)}.$$

If the Young's modulus, E, of the grout is not measured, it can be taken to be 4070 times f'_c (in MPa) (75-mm-cylinder).

Second, the value of the undrained shear strength of the soil, s_u, should be taken as the average value within the upper 4 D of the pile. If s_u varies substantially with depth within the top 4 D of the pile, it is prudent to use a value near the lower limit of the values within that depth range. If a sand seam is present within that depth range, it can usually be ignored in computing s_u if it is thin (less than 0.5 D) and if it is located at least 2 D below the ground surface. Otherwise, the designer must use his or her judgment in selecting a value for s_u. For example, if the site consists of clay except in the top 0.3 m (1 ft.), where loose, waterbearing silt is present (a common condition in northern Harris County), it is probably prudent to disregard the silt entirely and to assume that the ground surface is at the top of the clay beneath the silt.

It is noted that subsurface investigations at sites where laterally loaded CFA piles are anticipated should involve careful sampling and testing of the soils near the surface, which is the soil that provides the greatest proportion of the lateral resistance. It is also specifically noted that if the upper 4 D of the site consists substantially of coarse-grained soils (silts, sands and gravels), that the procedure described in this report does not apply.

Computational procedure

The design wind load is estimated according to the AASHTO specifications [Eq. (2.6)]. If vehicle impact loads are to be considered, they are computed according to appropriate criteria. The ground-line shear load, P_t, and moment, M_t, are calculated accordingly. The value of M_t is then converted to an equivalent shear load, P_{teq}, by the following equation, whose inputs include s_u (the undrained shear strength of the soil), E_p (the Young's modulus of the pile material), and D (the pile diameter). Units are kilonewtons (kN) and meters (m). Note that s_u and E_p are in kPa (1 kPa = 1 kN/m^2).

$$P_{teq} = [\ 1.83\ M_t^{0.82}\ s_u^{0.30}\ ] / [\ E_p^{0.11}\ D^{0.46}\ ]\ . \tag{4.1}$$

The equivalent total shear load applied at the ground line, $P_t + P_{teq}$, should not exceed the load that produces cracking, P_{cr}, which can be determined according to the results of the numerical analyses shown in Fig. 4.1. Alternatively, the following equations present a best fit of these results:

$$P_{cr\ (kN)} = [34.48\ /\ f'_{c\ (MPa)}]^{\ 0.5}\ [0.7\ s_{u\ (kPa)} + 110)]\ D(m)^{1.6} \tag{4.2a}$$

or
$$P_{cr\ (lb)} = [5000/\ f'_{c\ (psi)}]^{\ 0.5}\ /\ [3.05\ s_{u\ (psi)} + 69.5)]\ D(in.)^{1.6} \tag{4.2b}$$

Equation (4.2) is valid for soils with s_u = 70 kPa to 172 kPa (10 psi to 25 psi).

Next, the ground-line deflection for simple static loading, y_t, is determined by the following equation. Units are kilonewtons (kN) and meters (m). 1 kPa = 1 kN/m^2.

$$y_t = [\ 1.22\ (P_t + P_{teq})^{1.78}\ ] / [\ E_p^{0.57}\ D^{2.56}\ s_u^{1.21}\ ]\ . \tag{4.3}$$

The ground-line deflection under N cycles of load, y_{tc}, is then estimated according to the following equation, if it is desired to model cyclic loading:

$$y_{tc} = y_t (1 + 0.6 \log N) . \tag{4.4}$$

Equation (4.4) is a simplified form of Eq. (3.5). The number of load cycles, N, may be assumed as 5 for design of sound wall foundations if sufficient data are not available.

If the y_{tc} value determined by the Eq. (4.4) exceeds the maximum allowable deflection as specified by the structural engineer, D should be increased and the computational procedure repeated with the new D until the allowable deflection is satisfied.

The maximum moment, M_{max}, associated with the (P_t + P_{teq}) loading condition can be calculated according to the following equation:

$$M_{max} = M_t + [(0.244 P_t^{1.26} D^{0.48} E^{0.14}) / s_u^{0.22}] . \tag{4.5}$$

The pile is then structurally designed as a beam-column section with the axial force and bending moment = M_{max}. The minimum longitudinal steel should be one per cent according to TxDOT requirements.

The final step of the design is to calculate the minimum penetration of the pile. It was demonstrated numerically by Gazioglu and O'Neill (1984) that there is a "critical pile length" beyond which the presence of additional pile length has negligible effect on pile-head-behavior. The minimum pile length, L, can thus be defined, according to their study, as follows:

$$L = 3\,D\,(E_pI / E_sD^4)^{0.286} \tag{4.6}$$

where I = uncracked moment of inertia of the pile, and

E_s = a strength-correlated soil modulus, which can be calculated according to Eq. (2.2).

E_p, E_s, I and D are expressed in consistent units, and L is in the units of D.

Empirically, a penetration of 13.3 D was found to be appropriate for the conditions of the loading tests reported here. That value may not be appropriate for all sites and all loading conditions, but it represents a reasonable value from which the designer can start.

The design method requires some discussion. First, it is suggested that the equivalent ground-line shear $P_t + P_{teq}$ be limited to the value that produces cracking in the CFA pile. In point of fact the CFA pile can take much higher loads than this before developing a plastic hinge and reaching the ultimate limit state. However, it was clear in both the field tests and the LPILE simulations that the load-deformation behavior of the pile softens considerably once first cracking develops. That is, substantial deflection occurs when shears are applied after the pile cracks, which could cause visible movements in the wall which, while not necessarily unsafe, may be unsightly and require repair.

Second, the safety of the wall may be compromised in the if the pile is allowed to remain in the ground in a cracked condition. CFA piles tend to develop maximum moments, and cracks, that are not far below the ground surface, often within the zone of partial soil saturation. In such a case the reinforcing steel has access to oxygen, and rapid corrosion may ensue unless the steel is epoxy coated or otherwise protected.

Whether the cracking load should be considered as the structural failure load, as is suggested here, is a matter of decision by the design engineer. In the event that the cracking load is not considered to be critical, that is, the conditions cited above do not control the design, higher loads can be accommodated. In such a case the piles should be designed using LPILE, or a similar program, using the modified Reese-Welch p-y criterion given in Chapter 3.

Third, there is the issue of the values of the load and resistance factors or factors of safety that should be used in the design to assure safety. The design method reported here does not consider either load and resistance factors or factors of safety, or rather considers all such factors to be 1. The subject of the selection of safety factors is beyond the scope of this study; however, it is the opinion of the authors that P_t and M_t should be factored loads in load factor design for the purpose of assuring that the pile does not crack (assuring that $P_t + P_{teq} < P_{cr}$) and for computing M_{max} [Eq. (4.5)]. In allowable stress design, appropriate factors of safety should be applied to P_t and M_t for this purpose. The loads should probably not be factored for the solution of Eq. (4.3) or Eq. (4.4), which involve deflections, to be consistent with present load and resistance factor philosophy expressed by AASHTO (1994).

Since the design conditions for the soil already involve the effects of cyclic degradation and since either wind buffeting or vehicle impact loads, the loads that are assumed to control the design of sound walls, are expected to produce viscoelastic stiffening in the soil that is not modeled in the method presented here, it appears prudent at this time not to factor the resistance, that is, not to reduce the values of s_u, E, or f'_c used in Eqs. (4.2) and (4.5) in a load factor design analysis. This opinion is also supported by the fact that large reserve capacities are available in the event that cracking actually occurs and that the consequences of cracking failure in the CFA pile will unlikely threaten the immediate collapse of the wall.

Example problem

In order to illustrate the simplified procedure, the following example is given. Consider the sound wall barrier shown in Fig. 4.2. The wall is located in a residential area and subjected only to wind pressure with a maximum wind speed, w, of 90 mph. The foundation of the wall consists of a single row of CFA piles at a spacing of 6.1 m (20 ft). The piles have a strip cap with a depth of 0.5 m (1.64 ft) and a width of 1.0 m (3.28 ft). The minimum compressive strength of the pile grout, f'_c, is 34.48 MPa (5000 psi), and the pile modulus is 24.7 GPa. The soil at the site has an average undrained shear strength, s_u, of 120 kPa. For a pile diameter of 0.61 m (24 in.), the following are required:

1- Estimate the unfactored design load on the CFA piles supporting the wall.

2- Compute y_t and y_c associated with the design load.

3- Compute the maximum bending moment on the CFA pile.

4- Determine the adequate depth of embedment.

The calculations are as follows:

1- The wind pressure, w, can be estimated according to AASHTO [Eq. (2.6)], knowing that C_d = 1.2 and C_y = 0.37 for a sound barrier with a height less than 8.5 m in a residential area, as follows:

$$w = 0.00256 * (1.3 * 90)^2 * 1.2 * 0.37 = 15.56 \text{ psf} = 0.745 \text{ kPa}$$

$$P_t = 6.1 * 6.1 * 0.745 = 27.7 \text{ kN}$$

$$M_t = P_t * H/2$$

$$= 27.7 * 3.05 = 84.5 \text{ kN-m}$$

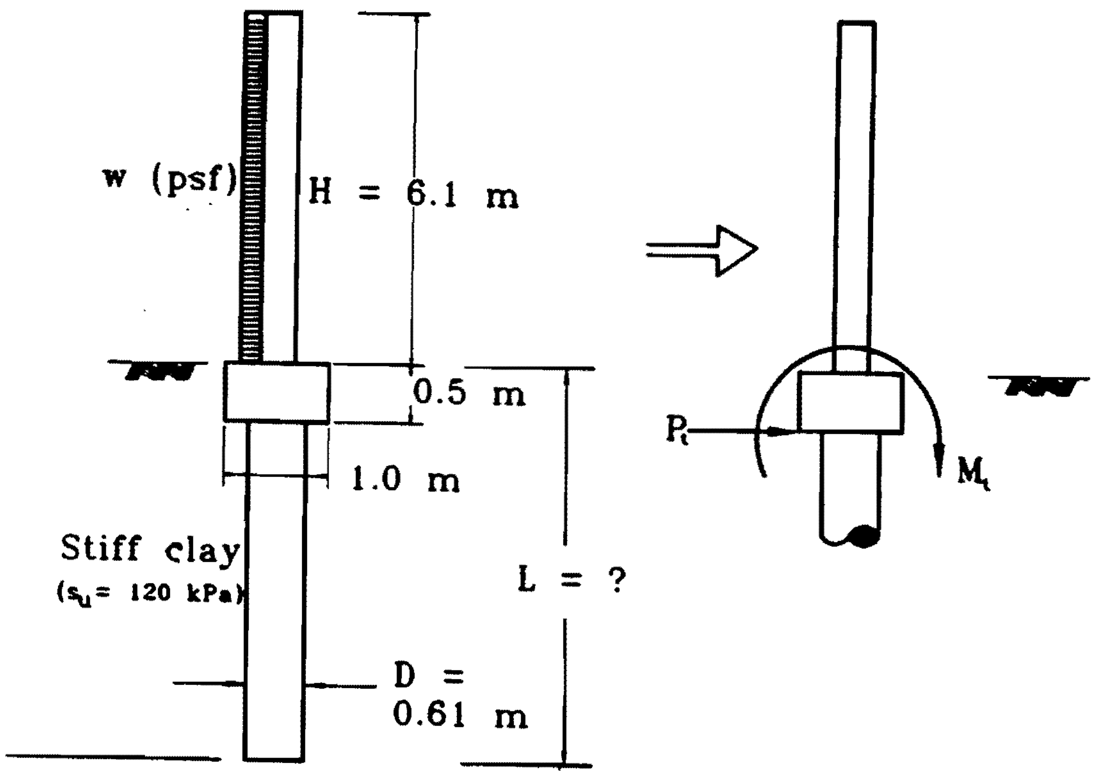

Fig. 4.2. Data for example problem

The equivalent ground shear, P_{teq}, associated with M_t is calculated according to Eq. (4.1), as follows:

$$P_{teq} = 1.83 * (84.5)^{0.82} *(120)^{0.3} \ / \ (24.7*10^6)^{0.11} * (0.61)^{0.46} \ = 56.4 \text{ kN}$$

$$(P_t + P_{teq}) = 27.7 \ + 56.4 = 84.1 \text{ kN}$$

(This load may be factored at the discretion of the designer).

2- Next, the total load P_t + P_{teq} is checked against the theoretical P_{cr}, which is determined as follows, using Eq. (4.2 a):

$$P_{cr} = (34.48/34.48)^{0.5} * (0.7*120 + 110) * (0.61)^{1.6} = 87.97 \text{ kN} > 84.1 \text{ kN} \quad \text{O.K.}$$

3- The deflection at the pile head, y_t, is calculated according to Eq. (4.3), as follows:

$$y_t \ = 1.22 * (84.1)^{1.78} / (24.7*10^6)^{0.57} *(0.61)^{2.56} * (120)^{1.21}$$
$$= 0.00215 \text{ m} = 2.15 \text{ mm}$$

The cyclic deflection, y_{tc}, due to 5 loading cycles is calculated according to Eq. (4.4), as follows:

$$y_{tc} = 2.15 * (1 + 0.6 * \log 5) = 3.05 \text{ mm}$$

If y_{tc} exceeds the maximum allowable deflection, the pile diameter should be increased and the computational procedure repeated. Otherwise, M_{max} is calculated according to Eq. (4.5), as follows:

$$M_{max} = 84.5 + [0.244 * (27.7)^{1.26} * (0.61)^{0.48} * (24.7*10^6)^{0.14} / (120)^{0.22}]$$

$$= 84.5 + 47.8 = 132.3 \text{ kN-m}$$

4- To obtain the minimum pile penetration according to Eq. (4.5), E_s should be determined. An average value of E_s can be calculated according to Eq. (2.2), at a depth of 4D (4 * 0.61= 2.44m), as follows:

$$E_s = (206 + 1.4*2.44)*120 = 24130 \quad \text{kPa}$$

Thus, the minimum pile length from Eq. (4.4) is

$$L = 3 * 0.61\ [24.7*10^6*(\pi\, 0.61^4 /64) / 24130 * 0.61^4]^{0.286} = 5.61 \text{ m}$$

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CHAPTER 5: GROUT BEHAVIOR

Introduction

The major constituent in the CFA pile is the grout. Unfortunately, the grout is not always given adequate consideration for its intended purpose - effective long-term load carrying capacity. CFA pile grouts must have good working properties so that they can be pumped into place. They should also have low shrinkage, but the limits are not generally specified in the literature. Both properties are critical for effective load transfer in the CFA pile and must be specified. A laboratory study was undertaken to evaluate the working and mechanical properties of potential grout mixtures that can be used in CFA piles. The effect of the fluidizer and fibers on the performance of grouts was as studied to assist in the development of a specification for the grout.

Review of grout standards and requirements

Grouts used in the construction of CFA piles are usually rich in cement for the purpose of improving pumpability. Cement content can range from 2.56 to 3.52 kN/m^3, or 8 to 11 sacks of cement per yd^3. For achieving good pumping characteristics, the aggregate is generally limited to concrete sand (ASTM C 33-93). A grout fluidizer, combining the functions of retarder and a pumping aid, is often added to the mix. Field control of grout consistency is maintained by use of the grout flow cone (ASTM C 939-94). To control shrinkage, a prehardening expansive system containing a gassing agent can be added at the job site.

The volume of grout injected to form a pile will always exceed the neat volume of specified pile dimensions. Grout volume as installed will range from as little as 110 per cent in stiff clays to 150 per cent or more in low-density silt. A typical requirement for CFA piles is that adjacent piles within center-to-center spacing of six diameters cannot be placed until after final set of the initial pile.

The reasons for planning for the performance testing of grouts in this study are as follows [Gulyas et al. (1995), U. S. Grout Corporation (1981), ASTM (1995) and Vipulanandan and Shenoy (1992)]:

1. Grouts are covered under the ASTM C 1107-91 specification, which establishes strength, consistency, and expansion criteria. This specification lists three types of grouts, depending on their volume change characteristics, prehardening volume control, and post-hardening volume control. Workability of these grouts is defined by their consistency classification using ASTM C 939-94, Flow Cone. Despite ASTM C 1107-91 being called "Specification for packaged dry, hydraulic cement grout," there is no requirement in the specification for two very important properties of a high quality grout, maximum allowable shrinkage and the requirements for minimum strength.

2. Soil types and thicknesses of the soil layers in which the CFA piles are installed will affect the grout mix design. Grouts can lose water and becomes hard prematurely in some soil formations(e.g., dry sands). Hence, grout designs should consider these factors.

3. All cementitious materials should be protected from ambient temperatures below 4.5°C (40°F). At these temperatures hydration is impeded, adversely affecting strength and expansion development of grouts. At temperatures slightly below 0°C (32 °F), ice is formed, rendering the bond of the grout to other elements ineffective whenever freezable moisture is present.

4. ASTM C 1107-91 does not differentiate grout based on the type of aggregate. But in the authors' opinion, the type and grading of the aggregates may play an important role in the grout behavior for CFA piles.

Use of fly ash will reduce the cement loading and bleeding in grouts [Vipulanandan and Shenoy (1992)]. Fly ash can also reduce the cost of the grout. Use of fibers in the grout can possibly improve the flexural properties of the grout (although fibers will be shown later not to be very effective in the grout used in this study), reduce shrinkage and eliminate the need for a reinforcing cage [Shah and Batson (1987)]. Use of additives such as silica fume is expected to improve the performance of the grouts further. In order to develop a specification for grout, a limited laboratory investigation of the potential grout mixtures for CFA piles was performed considering the factors enumerated above.

Methods

The grout study was divided into three tasks. In the first task, grout used in the field was collected during the construction of the CFA piles. The flow properties of the grout were measured at various levels of grouting. Cylindrical, beam and cubic samples were collected for mechanical and chemical testing. In the second task, the effects of fibers (steel and polymer), silica fume, fly ash and a fluidizer (sometimes called "fluidifier") on the grout behavior were investigated. ASTM C33-93 sand recommended for concrete was used in all the studies. Major variables and the tests for the study are summarized in Table 5.1. The properties of grout behavior of interest were pumpability, shrinkage, mechanical properties and chemical resistance. Chemical resistance of the grouts was studied by immersing the specimens in two different concentrations of sulfuric acid, sodium sulfate, hydrochloric acid and sodium chloride solutions. The pH of the solutions varied from 2 to 7. Grout samples were prepared using a laboratory-size concrete mixer. Based on the literature review and this limited laboratory study, specifications for CFA pile grouts was developed in the third and final task.

Working properties

Setting time (ASTM C 191-94): The Vicat's needle was used to determine the initial and final setting times of the grouts (without aggregates). The penetration of the 1.0 mm diameter needle was monitored with time. By definition, the initial time of set is the time corresponding to a needle penetration of 25 mm and the final time of set is the time corresponding to a needle penetration of less than 1 mm.

Table 5.1. Testing Program for Grouts

Variables		Types of Tests						Remarks
		Setting Time	Flow Cone	Compression	Tension	Shrinkage	Chemical Resistance	
Binder	Cement	X	X	X	X	X	X	Reduce cement and cost of grout. Fly ash can also reduce bleeding and permeability.
	Fly ash	X	X	X	X	X	X	
	Silica Fume	X	X	X	X	X		
Fibers	Steel	X	X	X	X	X		Fibers may improve the tensile/flexural strength and reduce shrinkage.
	Poly-propylene	X	X	X	X	X		
Admixture	Fluidizer (Fluidifier)	X	X	X				Reduce water to binder ratio.
Specimen Size	50.8-mm Cube			X				For developing a QC plan and to verify relationship between cube and cylinder strengths.
	76 X 152 mm Cylinder			X		X	X	
Number of Tests		20	20	40	10	10	17	Specification for grout.

Flow Cone (ASTM C 939-94): The flow cone test is used for routine quality control in the field. It is a static instrument that indirectly measures the viscosity of the grout. The variable measured is the time, in seconds(s), required for a given quantity of grout to pass through the orifice of a standardized funnel. The flow time was measured for the first 950 mL (32 oz) of the grout to flow through the orifice. The measurement obtained is influenced considerably by the rate of gellation and by the density, which varies the hydrostatic head of the column of the grout in the funnel. The flow cone measuring the viscosity gives a measure of the fluidity of the grout by virtue of the "time of efflux" through the orifice. For the purpose of perspective, the "time of efflux" or the flow time for water at 23° C is 28 seconds for an orifice diameter of 13 mm (0.5 in.).

Nondestructive tests

Ultrasonic Pulse Velocity Test (ASTM C 597-83): Compressive wave pulses of high frequency (greater than 20 kHz) are transmitted through the test specimen by an electro-acoustical transducer held in contact with one surface of the test specimen. After traversing through the specimen, the pulses are received and converted into electrical energy by a second transducer located at a distance "L" from the transmitting transducer. The transit time (T) of the pulse is measured electronically. The pulse velocity (V_p) is calculated by dividing L by T. Pulse velocity measurements were made with a commercially available portable V-meter. Lead-zirconate-titanate ceramic transducers with natural frequencies of 50 kHz was used. Castrol water pump grease was used to provide good coupling between the specimen and the transducers. The transit time of the ultrasonic pulse through the specimen under direct transmission, with the transducers on opposite faces along the length, was recorded up to an accuracy of 0.1 ms. The pulse velocity for high-quality concrete is of the order of 4560 m/s (15,000 ft/s).

Mechanical properties

Compressive Strength (ASTM C 109-92; C 39-94): The 50.8-mm cube and 75 X 150 mm cylindrical specimens were used for the compression tests. Compression tests were performed using a screw-type machine with a capacity of 44.5 kN (10 kips) and a servohydraulic Tinius-Olsen machine with a capacity of 1.7 MN (400 kips). The displacement rate was kept constant at 0.03 mm/min. An extensometer with a gage length of 50 mm was used with the cylindrical specimens to measure the axial deformation in the specimen to an accuracy of 2.5×10^{-4} mm (1×10^{-5} in.). The specimens were loaded monotonically during the process of testing. The specimens were trimmed and capped to ensure parallel surfaces. At least three specimens were tested under each condition.

Tension Test (ASTM C 190-90): Tests were performed on dog-bone-shaped specimens of grouts to determine the tensile strength. The screw-type machine of 44.5 kN (10 kips) capacity was used for determining the splitting tensile strength of grouts. At least three specimens were tested under each condition.

Shrinkage (ASTM C 1090-93): In order to determine the shrinkage in the grouts 75 X 150 mm cylindrical specimens were used. In this method, changes in specimen height were measured using a micrometer accurate to 0.02 mm.

Chemical resistance

Chemical resistance of the field grout was investigated by immersing 75 X 150 mm cylinders in various chemical solutions. Sulfuric acid (pH of 2 and 4), sodium sulfate (0.5 and 2 per cent), hydrochloric acid (pH of 2 and 4) and sodium chloride (0.5 and 2 per cent) solutions were selected, and the tests were performed at constant pH. The change in weight, dimensional changes, pulse velocity and total Ca^{2+} in the solution were monitored at regular intervals. Monitoring of Ca^{2+} indicates how fast the grout is being corroded with time.

Materials

In addition to testing the field samples, the laboratory mixer was used to prepare other potential grout mixtures. Table 5.2 summarizes the composition of various mixtures investigated in this study. The field mix had 75 per cent cement and 25 per cent the fly ash in the binder. Mix-1 had 35 per cent fly ash with reduced cement. In Mix-2 silica fume was tested as a replacement for 10 per cent of the cement. Mix-3 and Mix-4 had polypropylene and steel fibers, respectively. A higher percentage of fluidizer was used in Mix-5, while all other mixtures had 0.5 per cent fluidizer.

Table 5.2. Compositions of Field Mix and Trial Mixes

	Field Mix	Mix No. 1 35 per cent fly ash	Mix No. 2 10 per cent silica fume	Mix No. 3 2 per cent propylene	Mix No. 4 1 per cent steel fibers	Mix No. 5 increased fluidizer
Cement kN/m^3	2.560	2.162	2.305	2.560	2.560	2.560
Fly ash kN/m^3	0.766	1.164	0.766	0.766	0.766	0.766
Water kN/m^3	1.420	1.420	1.420	1.420	1.420	1.420
Silica fume kN			0.333			
Additive kN/m^3	0.013	0.011	0.012	0.013	0.013	0.128

Results and discussion

The average unit weight for the field mix was 21.4 kN/m3 (134 pcf). The unit weights for Mix-3 and Mix-4 were 20.8 (130 pcf) and 21.6 kN/m3 (135 pcf), respectively. Other mixtures did not show any significant variation from that of the field mix.

Setting Time: The final setting times for the cementitious grouts with additives or fibers are summarized in Table 5.3. All mixtures except Mix-5 contained 0.5% fluidizer, the same as that used in the field mix. Based on the test results, the setting time was only affected by the fluidizer. Increasing the fluidizer from 0.5 per cent (by weight of cement) to 5 per cent increased the setting time from 5.5 hr. to 22 hr. The 5.5 hr value is more appropriate for most applications.

Flow Cone: Efflux time for various grout mixtures are compared in Fig. 5.1. From the flow cone test, it was observed that the flow time increased with the addition of fiber and silica fume. Efflux time for the field mix was 33 sec. Mix-3 with 2% polypropylene fiber had the maximum increase in the efflux time. Efflux time for Mix-5 was 29 sec., which was close to that of water.

Table 5.3. Properties of Various Grout Mixes

	Field mix	Mix-1	Mix-2	Mix-3	Mix-4	Mix-5
Constituents	**30 per cent fly ash & 70 per cent cement**	**35 per cent fly ash & 65 percent cement**	**10 per cent silica fume**	**2 per cent propylene**	**1 per cent steel**	**5 per cent fluidizer[1]**
Setting time (hours)	5.5	5.5	5.25	5.5	5.5	22
Shrinkage[2] per cent height change	0.015	0.015	0.010	0.015	0.013	-

1- By weight of binder

2- Shrinkage test - ASTM C 1090

Pulse Velocity: Variations of pulse velocity with curing time for the field sample are shown in Fig. 5.2. The pulse velocity of the grout increased from 2,900 m/sec (9,500 ft/s) after one day of curing to 3,400 m/sec (11,200 ft/sec) after 7 days of curing, a 17 per cent increase. Pulse velocity continued to increase with curing time for the grout, and after 28 days of curing, it was 3,800 m/s (12,396 ft/s), a 10 per cent increase over 7-day-cured grout. Pulse velocities for grout mixes with polypropylene fibers (Mix-3) and steel fibers (Mix-4) after 28 days of curing were 3,900 m/s (12,800 ft/s) and 4,000 m/s (13,000 ft/s), respectively.

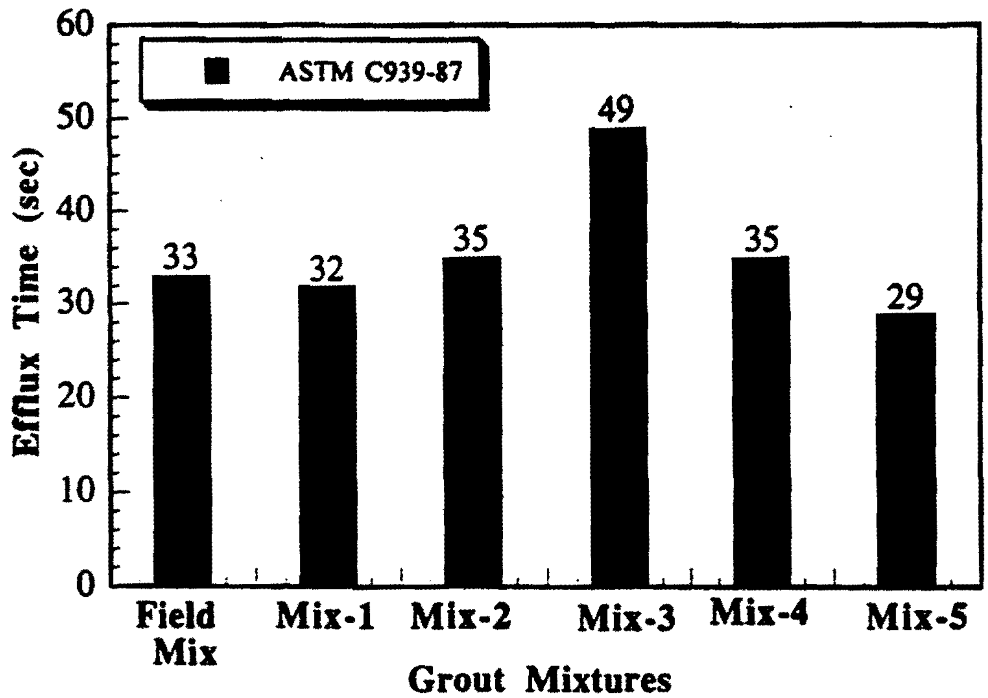

Fig. 5.1. Variation of efflux time with fiber type, silica fume and fluidizer

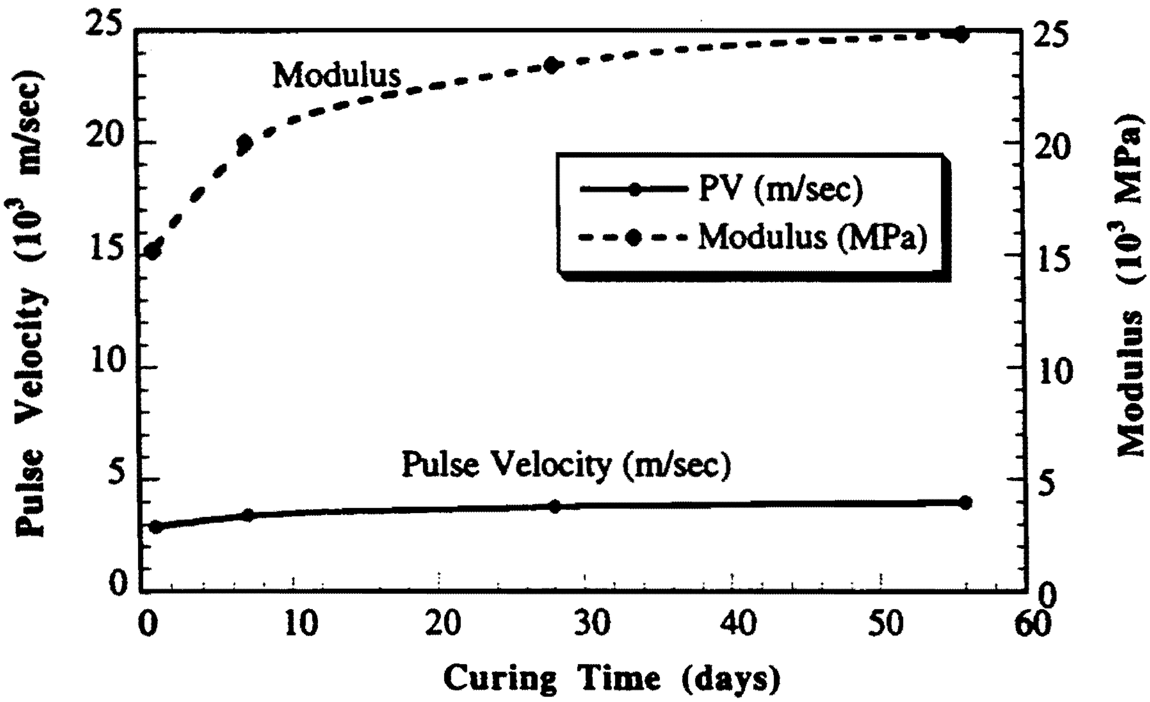

Fig. 5.2. Variation of compressive modulus and pulse velocity of the field grout mix with curing time

Strength

I. Compression: The compressive strength of the field mix was determined using cubic and cylindrical specimens. The variation of strength with curing time is shown in Fig. 5.3. It was observed that the cube strength was higher than the cylindrical strength. Variation of cylindrical strength with curing time agreed well with the ASTM C1107-91 specification for strength development. The cylindrical compressive strength of the field grout mix was increased by 84 per cent from the first day of curing to the 7th day of curing. The 7th-day strength of the field grout was 75 per cent of the 28-day strength.

The ratio of cylindrical strength to cube strength (σ cylinder/σ cube) varied from 0.89 to 0.93 with an average of 0.91 (Fig. 5.4). The ratio recommended in ACI 318-95 (ACI, 1995) value is 0.87. Variation of compressive strength for various mixes are shown in Fig. 5.5. Increasing the fly ash content from 25 per cent to 35 per cent did not affect the 28-day strength or the shrinkage of the grout. Addition of 2 per cent polypropylene fibers reduced the compressive strength of the field mix by 11 per cent. Silica fume increased the strength of the grout by only 3 per cent. A reduction in strength of 23 per cent was observed with Mix-5. Hence, the negative effect of excess use of fluidizer to increase the pumpability of the grout is verified.

II. Tension: The direct tensile strength (σ_t) of the field mix was 2.0 MPa (283 psi), less than 6 per cent of the compressive strength (σ_c) after 28 days of curing. Addition of polypropylene fiber increased the tensile strength of the grout by 7.5 per cent, the maximum tensile strength obtained in this study. Tensile strengths of various grout mixes (after 28 days of curing) are shown in Fig. 5.5. The ACI-recommended strength relationship for concrete,

$$\sigma_t = 7.5\,(\sigma_c)^{0.5} \quad , \tag{5.1}$$

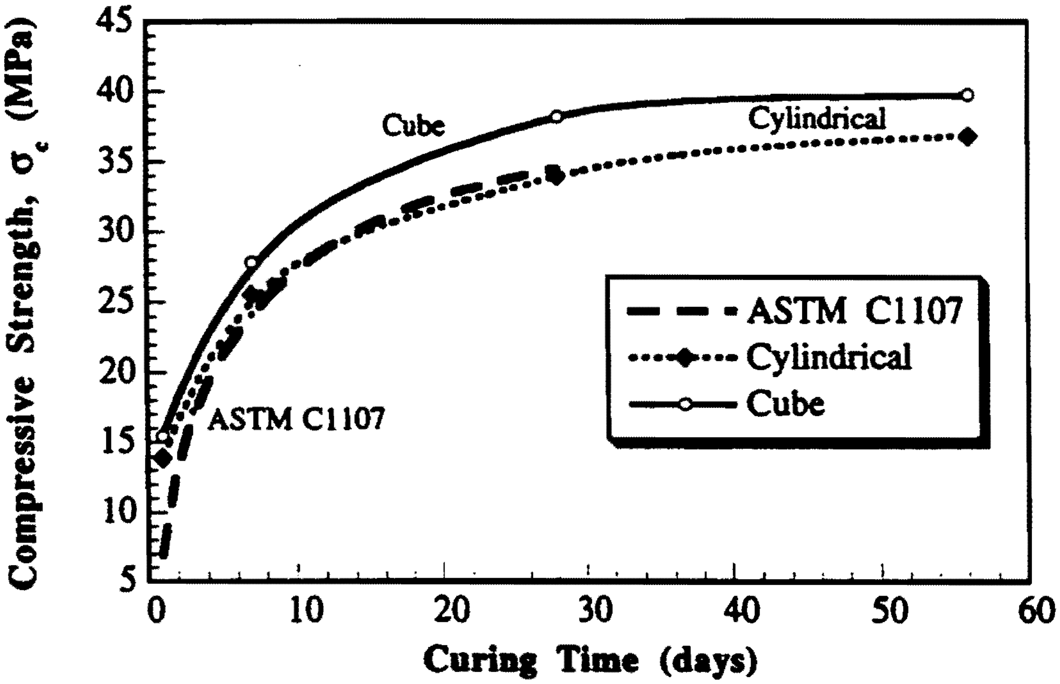

Fig. 5.3. Variation of compressive strength with curing time for the field grout mix and comparison to ASTM C1107-91

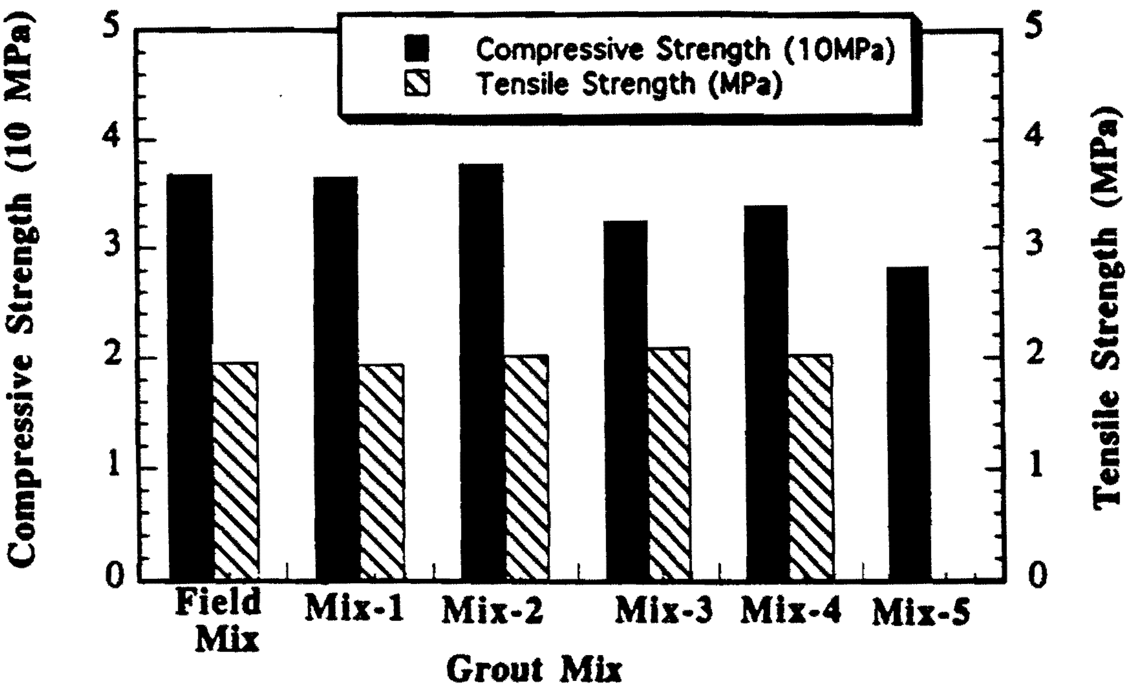

Fig. 5.4. **Variation of compressive strength ratio (cylinder strength / cube strength) with the curing time**

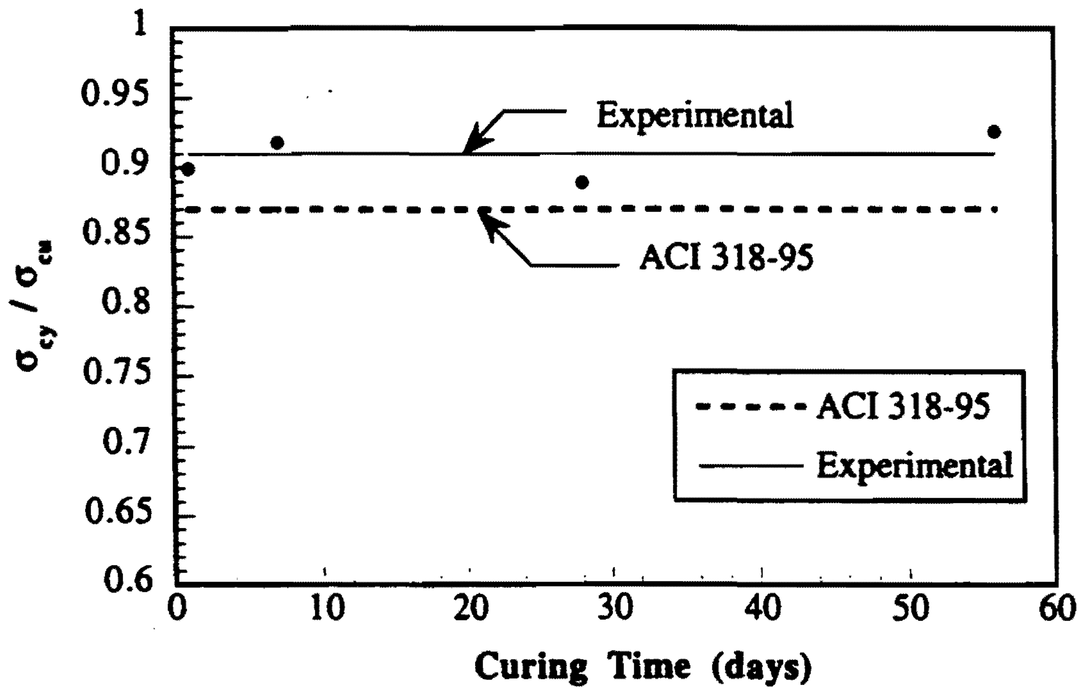

Fig. 5.5. Variation of compressive and tensile strengths of various grout mixes

overestimated the tensile strength of the grout mixtures. In Eq. (5.1) the strengths are in psi. The ratio of tensile strength to compressive strength of the grouts investigated varied from 0.053 to 0.064, which dictated the use of the low tensile strength value to develop the design method in Chapter 4..

Modulus

Variation of compressive modulus with curing time for the field mix is shown in Fig. 5.2. The compressive modulus increased by over 30 per cent from one to seven days of curing. The 7th-day modulus was 85 per cent of the 28th-day compressive modulus of the field grout mix. The relationship between modulus (E_c) and compressive strength recommended by ACI 318-95 is as follows:

$$E_c = 57{,}000\,(\sigma_c)^{0.5} \quad , \tag{5.2}$$

where all the properties are in psi. This relationship overestimated the modulus of the field grout mix by 16 %, as shown in Fig. 5.6. The relationships that best represent the behavior of the field grout are as follows:

$$E_c = 49{,}000\,(\sigma_c)^{0.5} \qquad \text{(in psi)} \quad \text{, and} \tag{5.3}$$

$$E_c = 4070\,((\sigma_c)^{0.5} \qquad \text{(in MPa).} \tag{5.4}$$

Stress-strain relationship

Figure 5.7 shows stress-strain relationships for the various grout mixtures (cured for 28 days) investigated in this study. Although differences in the relationships for various grout mixtures can be observed, the deviation from the field mix is not significant (except when steel fibers are added).

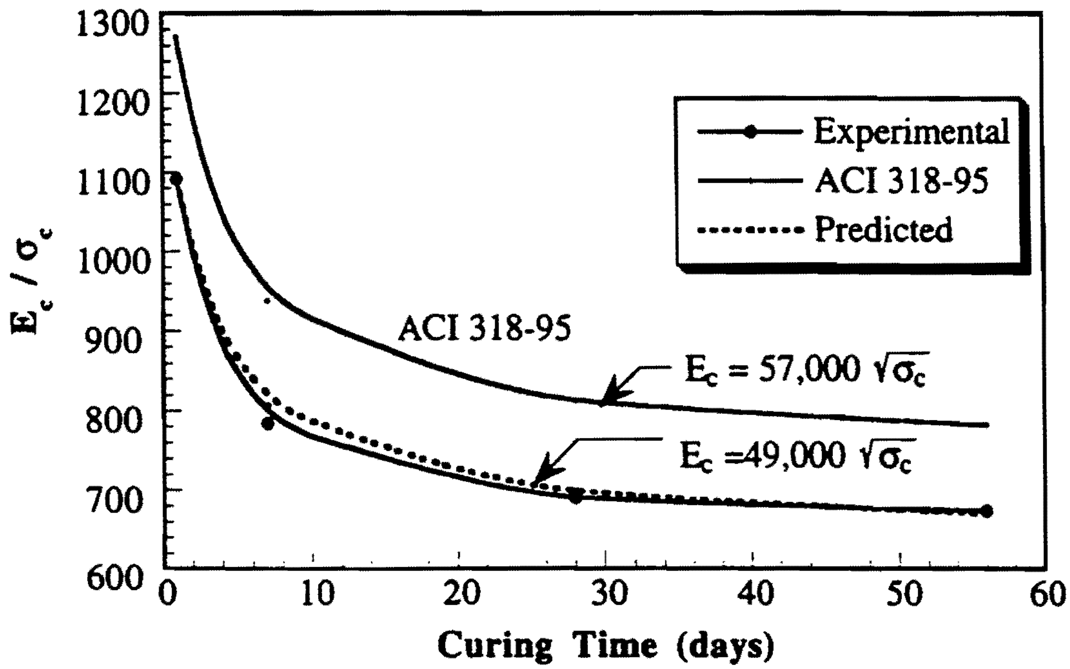

Fig. 5.6. **Variation of modulus-to-strength ratio with curing time**

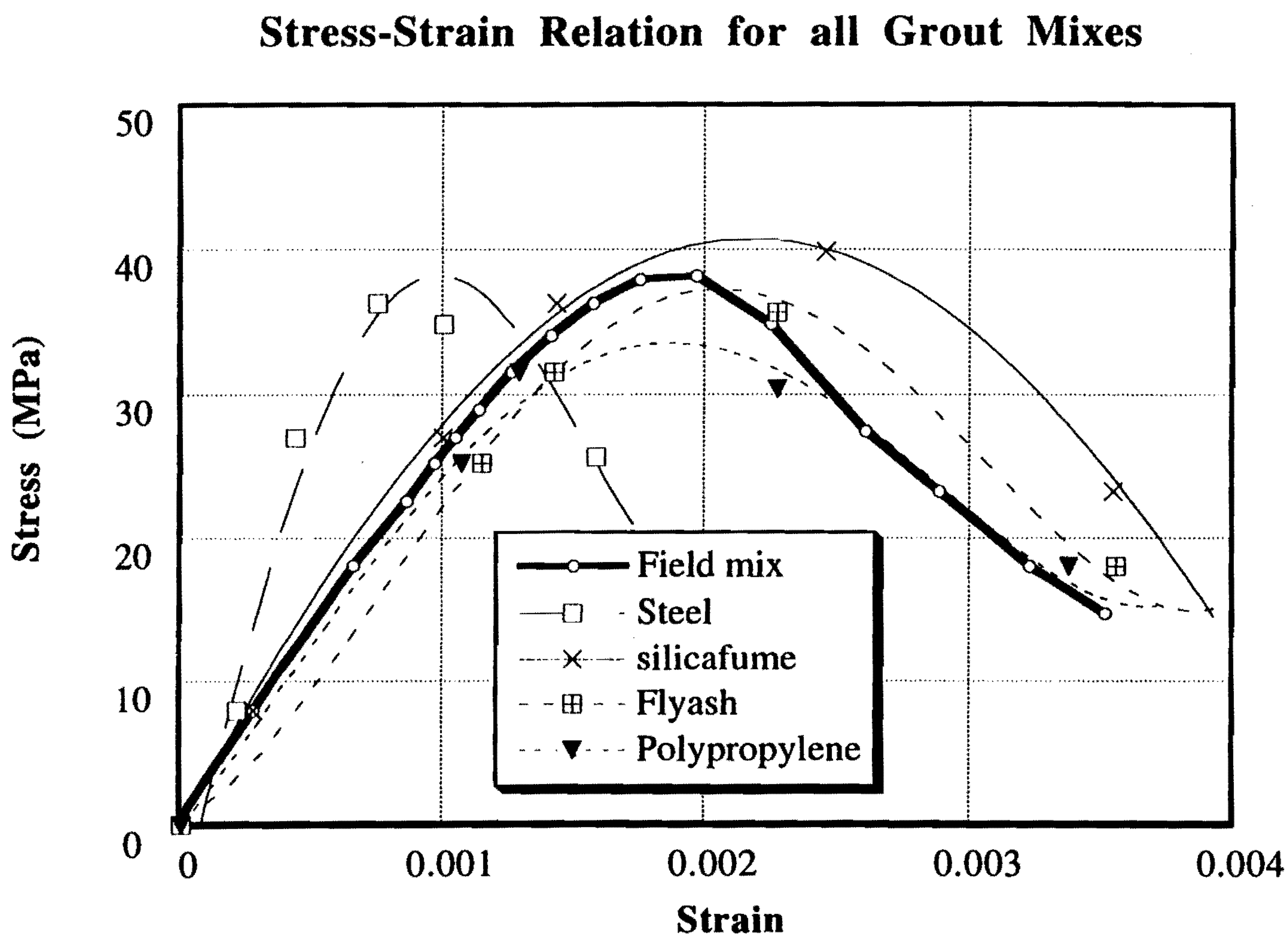

Fig 5.7 Compressive Stress-Strain Relationships for Various Grout Mixtures after 28 days of Curing

Chemical resistance

Results from four months of chemical immersion tests are summarized in Table 5.4.

Table 5.4. Results from the Chemical Immersion Tests on the Grout

Solutions	Content ratios	pH	% Change in Weight		Pulse Velocity (m/s)		Remarks
			2 months	4 months	2 months	4 months	
Water	100%	7.0	0.69	0.72	4077	4054	Control specimens
NaCl	0.5%	7.0	1.11	1.35	4562	4506	Effect of chloride
	2.0%	7.0	2.83	3.05	4240	4204	
Na_2SO_4	0.5%	7.0	0.74	0.83	4196	4055	Effect of Sulfate
	2.0%	7.0	2.25	0.64	4419	4278	
HCl	0.0004%	4.0	0.98	0.28	4526	4208	Effect of acid and chloride
	0.04%	2.0	1.91	1.98	4352	4006	
H_2SO_4	0.001%	4.0	2.18	2.20	4303	4206	Effect of acid and sulfate
	0.1%	2.0	1.19	1.25	4219	4093	

Change in Weight: In the first two months of immersion, all the specimens showed an increase in weight due to infiltration. The weight change observed varied from 0.5 to 3%. With continued immersion the 2% sodium sulfate solution showed surface corrosion and decrease in weight due to spalling off of materials.

Pulse Velocity: During the first two months of immersion all specimens showed increase in pulse velocity. The increase in pulse velocity varied from 100 to 600 m/s. Further immersion (up to 4 months) resulted in a slight decrease in pulse velocities, especially with the acids and sulfate solutions.

No unusual phenomena have been observed, and the changes reported so far are typical for cementitious materials. Chemical immersion tests will be continued to further investigate the chemical resistance of the grout.

Summary of the results

Grouts samples with unit weights of 20.8 kN/m^3 (134 pcf) were obtained from the field and characterized with several other grout mixtures prepared in the laboratory. Effects of using additional fly ash, silica fume and fibers in the grout mix were investigated. The grout mixtures are characterized based on their working and mechanical properties and chemical resistance. Based on the experimental results the following conclusions are advanced regarding the grout for the CFA piles:

1. Setting time: The final setting times for the cementitious grouts were not affected by the addition of fibers or increasing the fly ash (within the range investigated). The use of additional fluidizer affected the setting time.

2. Flow Properties: Flow time for the grouts investigated varied from 29 to 49 sec. Polypropylene had the greatest effect on the flow time. The field grout mix had a flow time of 33 sec.

3. Pulse Velocity: Pulse velocity for the grout increased with curing time. The pulse velocity of the field grouts after 28-days of curing was 3,800 m/s. The field grout attained 90 per cent of this value after 7 days of curing. All modifications to the grout mix had very small effects on the pulse velocity.

4. Strength: The compressive strength of the grouts increased with curing time. The 28-day strength of the field grout was 34 MPa, and the grout attained 75 per cent of that strength after 7 days of curing. Increasing the fluidizer in the grout mix affected the compressive strength of the grout. The average ratio of the cylinder compressive strength to cube strength for the grout was 0.91. Direct tensile strength of the grout mixtures varied from 5.3 to 6.4 per cent of the compressive strength.

5. Modulus: The compressive modulus increased with curing time for the grout mixes. The 7th-day modulus was 85 per cent of the 28th-day compressive modulus for

the field grout mix. The ACI relationship for modulus, which is based on the cylinder strength of concrete overpredicted the modulus of the grout mixtures. The ACI relationship has been modified for the grout.

CHAPTER 6: CONSTRUCTION SPECIFICATION

Introduction

A primary objective of this study was to develop a preliminary construction specification that could be used for the construction of CFA piles, more commonly called augercast piles, for sound wall foundations in the Houston District and clay-rich coastal soil formations in the region of the Texas Gulf Coast in general. The intent is to use this specification, with modifications as desired by TxDOT, as a special provision on sound wall foundation projects in the Houston District. This specification can be appropriately modified and extended to soil and rock types all over Texas as new experience is gained. It is intended that eventually the specification, as modified and extended through usage, will be made a part of the *Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges*.

Several sources of information were used for this specification:

- A survey of DOT practice that was made in April, 1997, as documented in Appendix B. Several state DOT specifications for augercast piles were received and reviewed as a result of this survey, and an industry guideline published by the Deep Foundations Institute was reviewed.
- Texas DOT Special Specification, Item 9000, "Augered Pressure Grouted Piles."
- Discussions with contractors and practicing engineers in the United States.
- Observations of contractor practices in the United States.
- Installation of the test piles described in Chapter 2 of this report.
- The experience of the senior author with construction practices for drilled shafts in the United States and with construction practices for augercast piles for transportation facilities in Europe.
- The experience of the senior author gained from the organization of a symposium on augercast piles and communication with participants from North America and Europe for the Transportation Research Board in 1994.

Construction of augercast piles

The augercast pile, also known as the continuous-flight-auger (CFA) pile or the augered-pressure-grouted (APG) pile, has been used in the Texas Gulf Coast area for over 30 years. Virtually all of its use has been in the private sector, especially for structures in oil refineries and petrochemical plants. A number of mid-rise buildings have been founded on augercast piles in the Houston-Galveston area. Therefore, a significant amount of experience in the construction of augercast piles in Gulf-Coast soils has been developed. Such soils are characterized by relatively high cohesion, and relatively little "running sand" is encountered. This is an important distinction because experience in Europe has indicated that in order to prevent significant loosening and depressuring of such soils the augercast pile rig must have the power essentially to screw the auger into the ground. In the United States most rigs used to advance augercast pile augers have insufficient power to accomplish that task and so typically spin while drilling and tend to allow some lateral squeezing of the soil into the auger flights as the soil is being cut at the bottom of the auger. However, since the soils under consideration here are cohesive, the loosening and depressuring resulting from the spinning is considered to be relatively minor.

The process of extracting the auger while pumping grout into the excavation made by the auger beneath the cutting edge of the auger is a critical operation in construction of augercast piles. If the auger is extracted too quickly, the grout column below the auger may "neck" or reduce in diameter, perhaps even breaking apart entirely. If the auger is not extracted quickly enough it may become lodged in the borehole, requiring the contractor to "jerk" it out, leaving a potential defect. Consequently, the issue of grout placement and auger withdrawal is thoroughly covered in the preliminary specification presented later in this chapter.

The sequence of construction of a typical augercast pile can be described as follows.

- Batch and mix the grout and ensure proper fluidity. Figure 6.1 is a photograph of the performance of a flow cone test on a jobsite to assess fluidity of the grout, which is covered in the specification.
- Position the continuous flight auger that will be used to make the excavation over the center of the pile. At this point the grout line, including the hollow stem of the auger, is charged with grout, and the grout outlet orifice at the bottom of the auger is plugged. See Figure 6.2. Drill the hole.
- Insert the grout and withdraw the auger simultaneously. For critical piles monitor the pump pressure and the flow rate for the grout as a function of position of the bottom of the auger. The flowmeter, pressure transducer, and position indicator used for automated monitoring of this process are shown in Figure 6.3; a closeup view of the flowmeter and pressure transducer in the grout line are shown in Figure 6.4; a view of the auger position indicator is shown in Figure 6.5; and the display for the incremental volumes of grout placed vs. auger depth is shown in Figure 6.6.
- Continue pumping the grout until the auger has cleared the ground surface. At this time the cutting face of the auger should be immersed in grout as shown in Figure 6.7.
- Clean the spoil and excess grout from around the head of the pile, place a sleeve around the top of the grout column to prevent intrusion of loose soil into the fluid grout column, and remove loose soil floating in the fluid grout column using a sieve or other suitable device, as shown in Figure 6.8.
- Place the reinforcing steel cage.

Fig. 6.1. Flow cone test for grout.

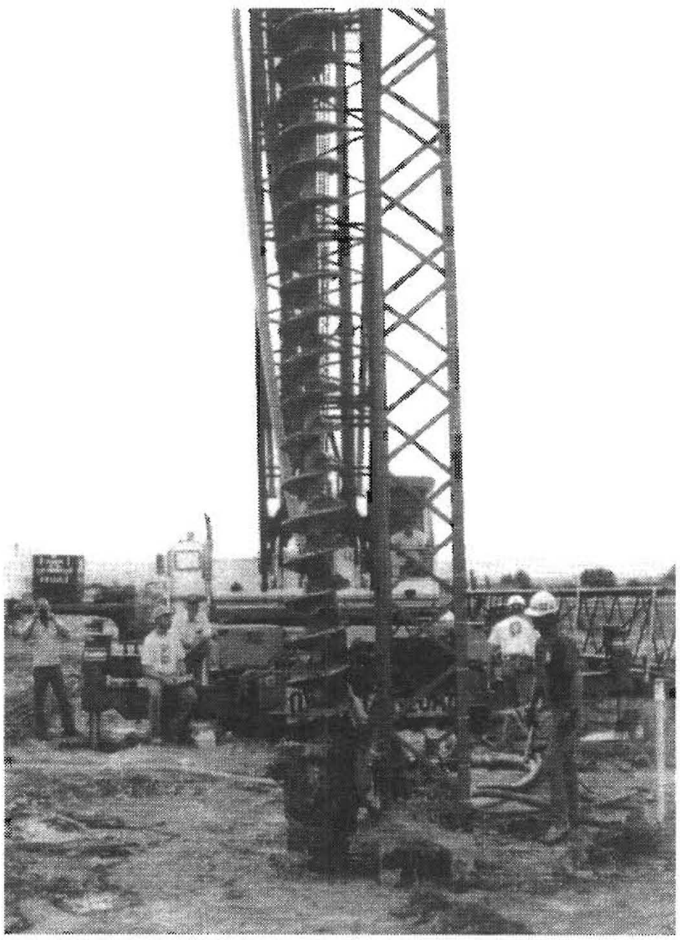

Fig. 6.2. Positioning continuous flight auger.

Fig. 6.3. View of crane and pump line with various indicators.

Fig. 6.4. View of flowmeter and pressure transducer in grout line.

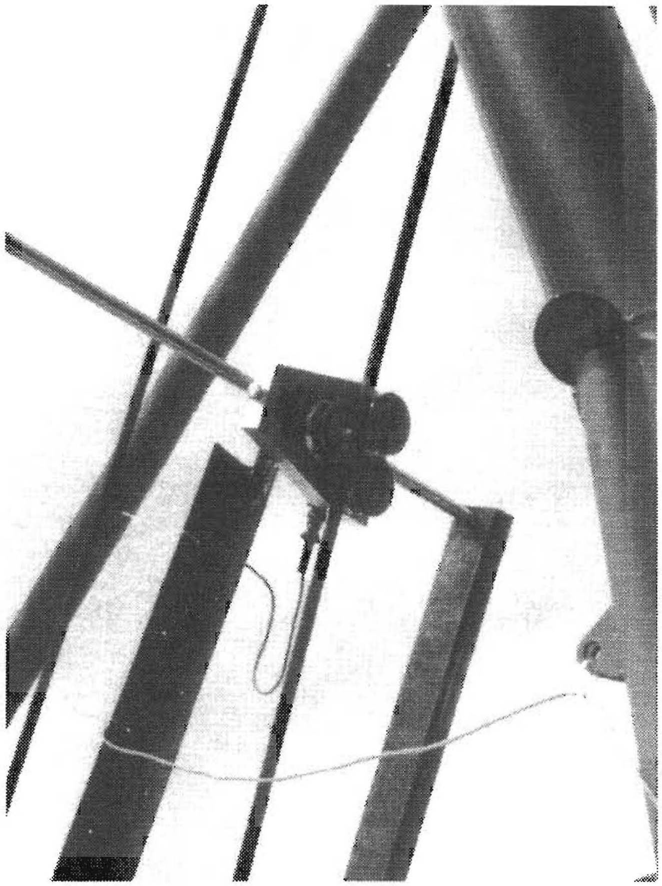

Fig. 6.5. Close-up view of auger position indicator above cab of crane.

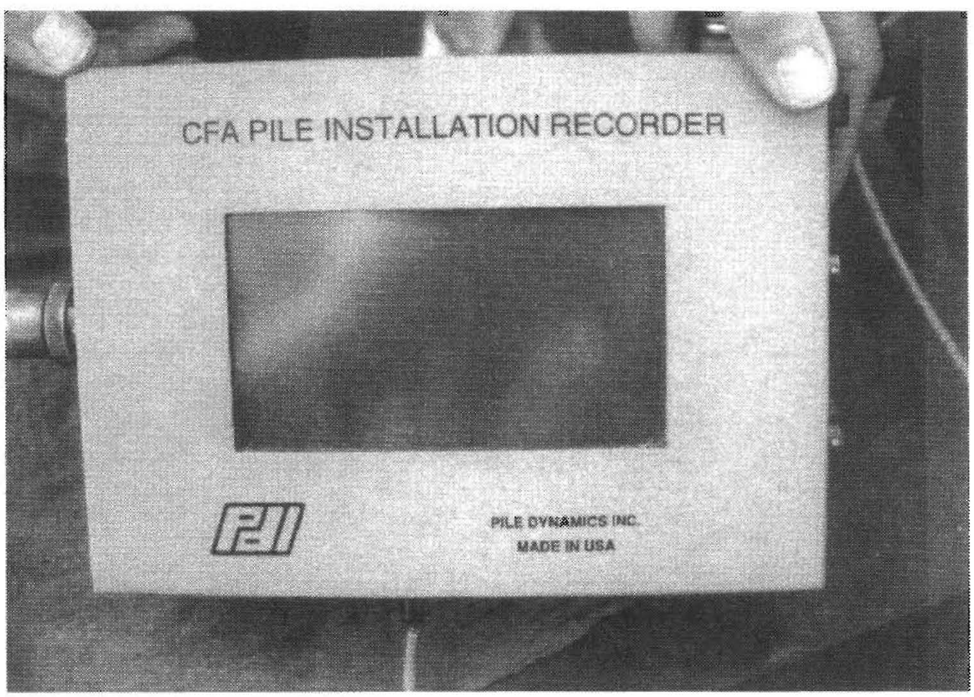

Fig. 6.6. View of output display for automated monitoring instruments.

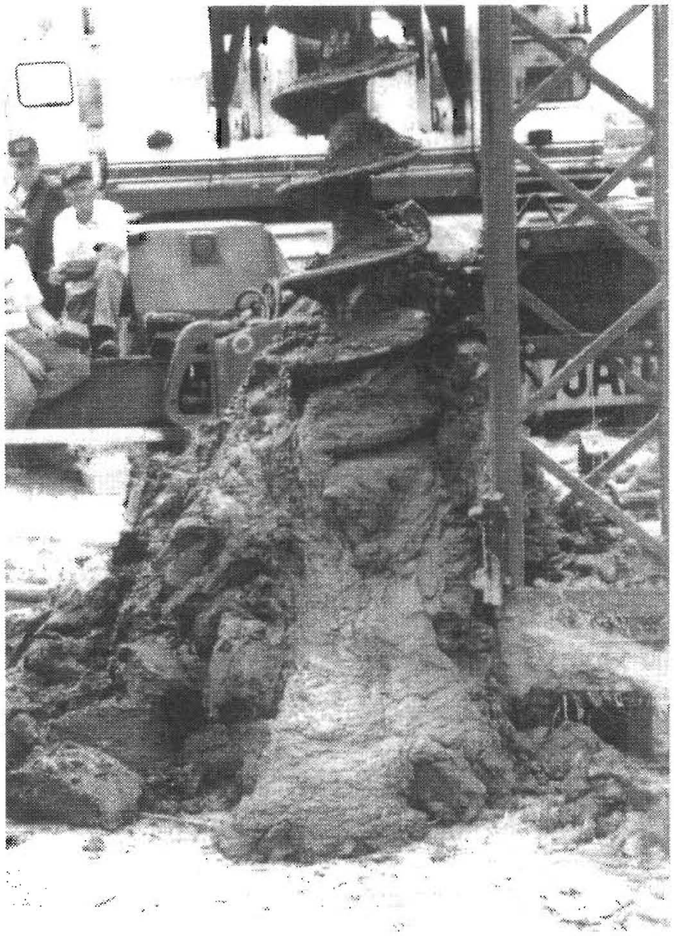

Fig. 6.7. Auger immersed in grout after completion of grouting operation.

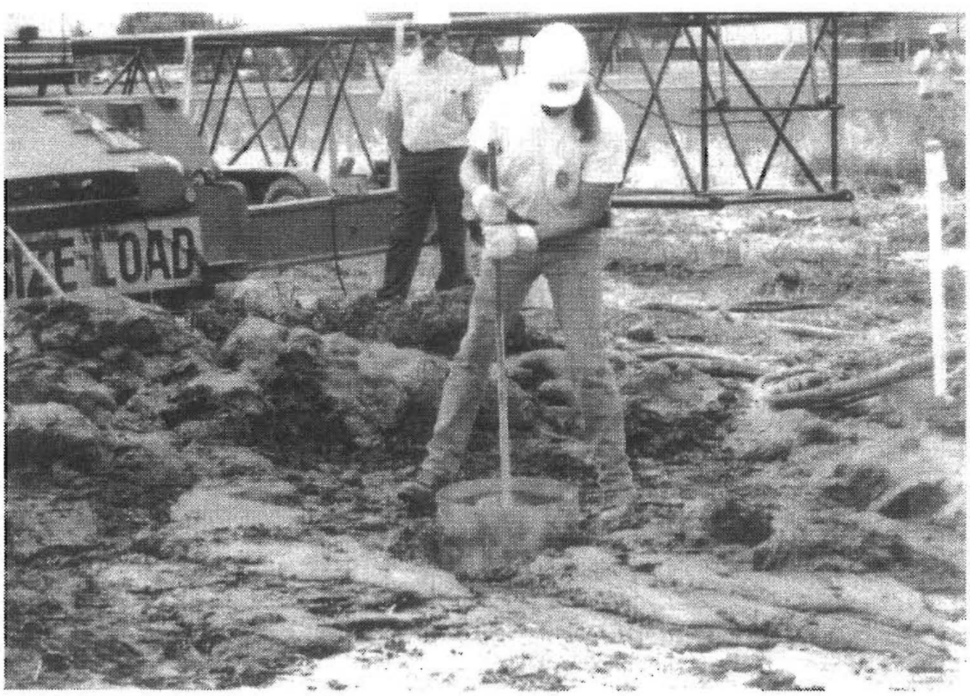

Fig. 6.8. Removing clods of loose soil from grout column.

Preliminary specification

The preliminary construction specification that was developed during the execution of this project is given in this section.

Preliminary Specification for Augercast (CFA) Pile Construction in Clayey Coastal Texas Soils

with *Commentary*

XXXX.1. Description. An augercast pile is defined as any foundation that is constructed by excavating soil or rock with the continuous insertion and rotation of a continuous flight auger into the ground to a specified depth, followed by pumping of fluid grout under pressure through the hollow stem of that auger to a port at the bottom of the auger, at which point it is injected into the excavation as the auger is withdrawn. Reinforcing steel, if specified, is inserted into the column of fluid grout following the completion of grout placement.

XXXX.2. Applicability. This item shall govern the construction of augercast pile foundations of the size and at the locations shown on the plans.

XXXX.3. Contractor Submittals.

XXXX.3.1. Pre-Bid Submittal. The foundation contractor shall provide the Engineer documentation of a minimum of three projects performed in the two-year period preceding the bid date in which augercast piles were installed successfully under subsurface and job conditions similar to those of the current project. The foundation contractor shall also provide documentation that the designated jobsite supervisor has had a minimum of three years of experience in supervision of the installation of augercast piles. Alternatively, the foundation contractor may demonstrate his or her competence to perform the work shown on the plans by installing a demonstration pile to the depth and diameter of the largest pile on the job and removing that pile from the ground for inspection by the Engineer.

Commentary: The quality of augercast piles is highly dependent upon the skill of the contractor and the specific crew that is assigned to the job. It is essential to establish at the time that bids are opened that the bidder is competent to perform the work at hand either through providing documentation of successful completion of prior jobs of a similar nature to the job being bid or by directly demonstrating his or her competence by installing a demonstration pile that does not contain defects and that has been constructed to at least the diameter and depth shown on the plans.

Since augercast pile contractors are usually subcontractors, it may also be possible to prequalify augercast pile subcontractors who have the necessary experience and to permit only those general contractors who employ prequalified augercast pile subcontractors to submit bids.

XXXX.3.2. <u>Pile-Installation Plan</u>. At least 30 days prior to the start of augercast pile installation the Contractor shall submit an augercast pile installation plan. This installation plan shall contain, but not be limited to, the following items:

a. List and sizes of proposed equipment, including cranes, augers, grout pumps, mixing equipment, and similar equipment to be used in construction, including details of procedures for calibrating pressures and volumes of grout pumps.

b. Step-by-step description of pile installation methods.

c. A plan of the sequence of pile installation.

d. Details of methods of reinforcement placement, including support for reinforcing cages at the top of the pile and methods for centering the cages within the grout column.

e. Mix designs for all grout to be used on the job.

f. Procedures for monitoring grout pressures during stroking and during resting of the pump and for monitoring the amount of grout placed in the excavation.

g. Procedures for protecting adjacent structures, on or off the right-of-way, that may be adversely affected by foundation construction operations.

h. Other required submittals shown on the plans or requested by the Engineer.

The Contractor shall demonstrate to the satisfaction of the Engineer the dependability of the equipment, techniques and source of materials to be used on the job.

Commentary: A clearly written pile installation plan can be very effective in reducing misunderstandings between the Engineer and the Contractor and can form the basis for solving potential problems before they occur, thus keeping the job on schedule and minimizing claims.

In reviewing the Contractor's submittal, the key information regarding the equipment that should be scrutinized is (1) the rated capacity of the crane; (2) the torque, rotational speed and weight of the gearbox on the drilling machine; (3) the horsepower of the hydraulic power unit used to power the drilling machine; and (4) the cylinder displacement, pump speed (stroke rate), engine horsepower and cylinder displacement of the grout pump to be used. The stiff, highly plastic clays of the Texas Gulf Coast require special consideration in sizing equipment for large-diameter augercast piles (0.61 m or larger). The minimum torque supplied by the gearbox should be 40.8 m-kN (30,000 ft-

lb), and the weight of the gearbox should be at least 22.3 kN (5,000 lb). The rotational speed should be not less than 40 rpm, which requires the horsepower of the hydraulic unit [=(torque in ft-lb)(RPM) (2π)/33,000] to be approximately 250 or greater. Smaller drilling rigs are widely available but are not capable of installing large-diameter augercast piles.

The contractor's plan for sequence of installation should preclude the installation of piles that are within six diameters of each other, center to center, prior to the time that the first pile installed has attained its permanent set.

XXXX.4. Protection of Adjacent Structures. The Contractor shall be solely responsible for evaluating the need for, design of, and monitoring of measures to prevent damage to adjacent structures, on or off the right-of-way. These measures shall include, but are not limited to, selection of construction methods and procedures that will prevent caving of soils or inward movement of soils into excavations and excessive migration of grout through the ground; monitoring and controlling the vibrations from construction activities, including placement of casings, sheet piling, shoring and similar ancillary features; and protecting utilities.

Structures located within 10 pile diameters clear spacing, or the planned length of the pile, whichever is greater, shall be monitored for vertical and horizontal movement in a manner approved by the Engineer within an accuracy of 0.3 mm (0.01 inch). Monitoring of adjacent structures will be done by an independent party approved by the Engineer and shall begin prior to construction of the pile or any casings, sheet piling, shoring or similar ancillary features. In addition to monitoring for movement, the condition of the adjacent structure, including cracks and crack widths, before and after construction of the augercast piles, shall be documented. Structures that are owned by the Texas Department of Transportation shall be monitored for movement but need not be monitored for condition unless called for on the plans.

The Contractor shall notify the Engineer of any movements detected in adjacent structures as soon as they are detected and shall take any immediate remedial measures required to prevent damage to the adjacent structure.

Commentary: The installation of augercast piles can result in settlement of the ground surface if the rate of rotation of the auger is high relative to its rate of penetration, especially in sandy soils. This action can promote settlement and damage to existing structures near the location of the pile installation. In some soils, although rarely in the stiff clays of the Texas Gulf Coast, the pumping of grout can result in the grout fracturing the ground and moving a considerable distance horizontally under pressure, which can serve to lift the ground surface and structures founded on or near the ground surface, including buried conduits. Careful monitoring of the movements of adjacent structures and changes in the condition of such structures is necessary in order for the Contractor to know when his or her procedures are producing ground movements in order for immediate corrective action to be taken. Condition surveys are needed for the evaluation of the effect of the construction process on the serviceability of adjacent structures by the Engineer. The Florida DOT specification for augercast piles contains an extensive section on vibration monitoring. Such monitoring is only applicable for cases where casing or sheet piling is driven, which is not a common practice in connection with the installation of augercast piles in Texas coastal soils. In cases in which such construction practices may be needed, a special provision on vibration monitoring should be added.

XXXX.5. <u>Materials</u>. The materials that are used in the construction of augercast piles shall conform to the requirements specified in following items in "Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges (1995)," or as otherwise noted.

a. Portland cement (Types I, IP, II, and III):	Item 524 (Hydraulic Cement)
b. Fly ash (Type A or B):	Departmental Materials Specification D-9-8900
c. Fine aggregate	Item 421 (Portland Cement Concrete), Table 2
d. Admixtures	Item 437 (Concrete Admixtures)
e. Water	Item 421 (Portland Cement Concrete - 421.2 (3))
f. Fluidizer (fluidifier)	ASTM C 937
g. Reinforcing steel	Item 440 (Reinforcing Steel)

Notes:

1. Type III portland cement shall not be used when the air temperature for the 12 hours following batching will exceed 15 degrees C.

2. Type B fly ash shall not be used in conjunction with Type II portland cement.

3. All admixtures must be approved by the Director of Materials and Tests, as specified in Item 437.

XXXX.6. Grout.

XXXX.6.1. Mix Design. The grout shall consist of a mixture of portland cement, fly ash, water, sand, fluidizer, and if necessary, retarder, proportioned and mixed so that the grout will exhibit the following properties:

a. All solids shall remain in suspension in the grout without appreciable water gain.

b. The grout shall have a fluid consistency represented by an efflux time of 32 - 36 seconds per 950 mL (quart) when tested with a flow cone in accordance with ASTM C 939 (13-mm-diameter outlet orifice), or 18 - 24 seconds per 950 mL (quart) when tested in accordance with ASTM C 939 (19 mm outlet orifice) unless otherwise specified by the Engineer.

c. The grout shall not exhibit shrinkage in excess of 0.015 per cent in the vertical direction, when tested in accordance with ASTM 1090, and when housed in a 100 per cent humidity room at a temperature of 20 - 23 degrees C.

d. Samples of the field grout mix, recovered and stored in cylinders 152 mm in diameter by 305 mm long, shall exhibit a compressive strength 28 days after casting of at least 27.6 MN/m^2 (4,000 psi), or as otherwise specified by the Engineer. Alternatively, 50.8-mm cube samples may be recovered and tested 28 days after sampling. If such a sampling method is used, the compressive strength 28 days after sampling shall be at least 30.3 MN/m^2 (4,400 psi). Each compressive strength determination shall consist of a minimum of one test on three separate samples, and the compressive strength shall be taken to be the numerical average of the results of three tests.

Commentary: Ideally, grout samples for flow cone testing should be taken at the outlet orifice on the auger of the drilling machine prior to the commencement of drilling, since pumping of the grout may reduce its flowability and increase efflux time. If grout delivery to the jobsite is such that sampling cannot be made at that point, the grout may be sampled from the chute of the ready-mix truck. At the discretion of the Engineer, additional samples may be taken at various times during the grouting process to ensure that consistent fluidity is being achieved. Sampling for strength and shrinkage is covered in XXXX.6.3.

XXXX.6.2. <u>Field Operations</u>.

a. Only pumping equipment approved by the Engineer shall be used in the mixing and handling of grout. All oil, rust inhibitors, residual drilling slurries and similar foreign materials shall be removed from mixing drums, stirring devices, pumps and lines, and all other equipment in contact with the grout before use.

b. All materials used to make the grout shall be accurately measured by volume or weight before they are fed into the mixer, either in the field or at the batch plant. The order of placing materials into the mixer shall be (1) water, (2) fluidizer, (3) other solids in order of increasing particle size. The fluidizer may also be added at the jobsite. If that process is followed, the order of mixing shall be (1) water, (2) other solids in order of increasing particle size, and (3) fluidizer (at the jobsite). The time of mixing shall not be less than one minute. If agitated continuously the grout may be held in the mixer or ready mix truck for up to 2.5 hours if the air temperature is not greater than 20 degrees C, or up to 2.0 hours if the air temperature is between 20 and 38 degrees C, if other than Type III portland cement is used. Grout shall not be placed if the air temperature exceeds 38 degrees C or is less than 4 degrees C.

c. A screen with a mesh with openings no larger 19 mm shall be used between the mixer and the pump, or between the delivery point from a ready mix truck and the pump, to remove large particles that can clog the grout injection system.

d. The grout pump shall be a positive displacement pump with a known volume per stroke that is capable of developing peak pressures of at least 2400 kPa (350 psi) at the pump.

e. The grout pump shall be equipped with, as a minimum, a calibrated pressure gauge that can accurately monitor both the peak and minimum pressures on each pump stroke. The pressure gauge shall be positioned on the immediate outlet site of the pump at ground level in such a manner that it can be easily viewed by the Engineer. The foundation contractor shall provide the Engineer with the results of a calibration performed on the pressure gauge at the beginning of the job that will demonstrate that the pressures indicated by the pressure gauge are within 3 per cent of the values indicated. The foundation contractor shall also provide the Engineer with the value of the volume of grout delivered by each stroke of the pump and shall demonstrate to the Engineer that the volume of grout delivered by each stroke of the pump is within 3 per cent of the value provided. The equipment shall also be recalibrated at such times as the Engineer suspects that the grout delivery performance has changed.

f. For those piles where such testing is indicated on the plans, the foundation contractor shall engage an independent consultant acceptable to the Engineer to place electronic flowmeters in the grout pressure line, electronic pressure transducers in the grout pressure line and an electronic position indicator on the crane line holding the auger to make automatic measurements of grout volume, maximum grout pressure and minimum grout pressure versus depth of the injection point.

Commentary: For noncritical foundations (e. g., sign foundations, sound-wall foundations) the amount of grout placed into the excavation is normally measured by counting the number of pump strokes required to fill the excavation with grout and multiplying by the calibrated volume of grout delivered by each pump stroke. Because some grout will be lost at the surface and because the excavation will ordinarily be slightly larger than the diameter of the auger, the volume of grout that is placed should always exceed the theoretical volume of the excavation. Empirically, the peak grout pressures at the discharge side of the pump should be at least 2070 kPa (300 psi) throughout the entire period of grout placement.

Commentary: For critical foundations (e. g., bearing piles for bridges and retaining walls) a specified number of piles on a job should be monitored more formally by developing and recording graphs of volume of grout placed versus depth of the grout outlet orifice on the auger and both minimum and maximum grout pressures on the pump stroke versus depth of the grout outlet orifice on the auger. Commercial, automated equipment is available through private consultants for acquiring and recording such data. In cases where such monitoring is performed, the volume of grout placed should not be less than 0.97 times the theoretical volume for any 0.61-m (2 foot) depth increment and should not be less than 1.15 times the theoretical volume of the entire pile. The average minimum pressure in the grout at ground level for a 0.61-m (2 foot) depth increment should not be less than the estimated total vertical pressure in the ground at the depth of the grout outlet orifice, and the average maximum pressure over the same depth interval should be at least 2070 kPa (300 psi).

XXXX.6.3. <u>Grout Testing for Strength and Shrinkage</u>. The Contractor shall make six 152-mm diameter by 305-mm long cylinder samples or six 52-mm cube samples for each 38 m^3 of grout placed, but not less than six such samples per working day, nor less than six such samples for each batch of grout produced by the supplier. Grout samples shall be taken from the top of the completed grout column within the augercast piles. Samples shall be made more frequently if specified by the Engineer. The samples will be tested by the Texas Department of Transportation, 2 at seven days after sampling; 2 at 28 days after sampling; and 2 will be held in reserve. Those samples tested at 28 days after casting shall exhibit a minimum compressive strength of at least 27.6 MPa (4,000 psi).

Commentary: Where augercast piles are used for critical foundations (e. g., bearing piles for bridges and retaining walls), a greater frequency of sampling and testing is indicated. No standard has been developed concerning this frequency, but it should be at least as great as the frequency of sampling concrete cylinders for drilled shafts and similar cast-in-place substructure or foundation elements. As a guide for strength *development, grout meeting these specifications typically attains 30 per cent and 70 per cent of its 28-day compressive strength after 1 and 7 days of curing, respectively.*

XXXX.7. <u>Construction Procedures</u>.

XXXX.7.1. <u>Excavation</u>. The Contractor shall perform the excavation required for the piling, through whatever materials are encountered, to the dimensions and elevations shown on the plans.

The center of any pile shall be within 25 mm (1 inch) of the location shown on the plans in a horizontal plane. The completed pile shall be plumb to within two percent, if vertical, or shall be installed to within four percent of its specified batter, as determined by the angle from the horizontal, if planned as a batter pile. Any pile in violation of these tolerances will be subject to review by the Engineer.

Should muck, organics, soft clay or other unsuitable materials be encountered within 1.5 m (5 feet) of the ground surface, such material shall be removed to its full depth, or to a depth of 1.5 m (5 feet), whichever is less, and laterally to a distance radially from the centerline of the pile not to exceed three pile diameters or 1/2 the distance to the closest adjacent pile, whichever is less. The excavation shall be backfilled with soil having a plasticity index of 20 or less, and such backfill shall be compacted to at least 95 per cent of its maximum dry unit weight as specified by AASHTO T 180 at within 2 per cent of optimum moisture content. Excavation of unsuitable surface material and backfilling shall be completed to the Engineer's satisfaction prior to the construction of augercast piles. Should more than 1.5 m (5 feet) of unsuitable surface material be encountered, the Contractor shall advise the Engineer immediately and proceed with work as directed by the Engineer. Should the Contractor suspect that any soils that are excavated are contaminated by hydrocarbons, refuse, or other environmentally hazardous material, he or she shall contact the Engineer immediately and proceed with work as directed by the Engineer.

Adjacent piles within six diameters, center to center, of each other shall not be installed until it can be demonstrated by the Contractor that the grout in the first pile installed is fully set.

Commentary: The 25-mm position tolerance is based on current TxDOT specifications for drilled shafts, which have proved satisfactory. The industry standard for augercast piles is more relaxed, with a position tolerance of 75 mm (3 inches) for individual piles and 150 mm (6 inches) for piles within groups of five or more.

XXXX.7.2. <u>Auger Equipment</u>. The auger flighting shall be continuous from the top of the auger to the bottom tip of the cutting face of the auger, with no gaps or other breaks. The length of any auger brought to the jobsite shall be such that the auger is capable of excavating a hole for the pile, and transporting grout to the bottom of that hole, to a depth that is 20 per cent greater than the depth of the pile shown on the plans. The auger flighting shall be uniform in diameter throughout its length, and the outside diameter of the auger shall not be less than 3 per cent smaller than the specified diameter of the pile. Only single helix augers shall be used. The distance between flights shall be approximately one-half of the diameter of the auger. The hollow stem of the auger shall be maintained in a clean condition throughout the construction operation.

The bottom of the auger flighting and the cutting teeth attached thereto shall be constructed geometrically so that the bottom of the excavation will be flat.

In order to facilitate inspection the auger shall be clearly marked every 0.3 m (1 foot) along its length so that such marks are visible to the unaided eye from the ground.

The grout outlet orifice on the auger shall be located at an elevation lower than that of the cutting teeth on the bottom of the auger. This orifice shall remain closed by a plug while the auger is being advanced into the ground. The plug shall be removed by pressure from the grout once the grouting begins.

The auger shall be guided at the ground surface by a suitable guide connected to the leads of the augercast piling rig. If the auger is over 12 m (40 feet) long, it shall also be guided by a guide above the ground-surface guide approximately half the length of the auger above the ground-surface guide. The leads that carry the rotary unit that powers the auger should be restrained against rotation by an appropriate mechanism.

The auger shall be advanced into the ground at a continuous rate and at a rate of rotation that prevents excess spoil from being transported to the ground surface. The rotation of the auger shall be stopped when the excavation reaches plan depth.

Should refusal be encountered before plan depth is achieved, rotation of the auger shall be stopped, and the Contractor shall inform the Engineer. Refusal is defined here as a rate of auger penetration of less than 300 mm / minute (1 foot / minute) with equipment that is appropriate for the job. The Contractor shall then proceed as directed by the Engineer.

Commentary: The auger should never be rotated excessively, since doing so may cause the soil to migrate laterally into the flights of the auger and be transported up the auger to the ground surface. This action, in turn, reduces the stresses in the ground and therefore the resistance of the pile. When refusal is reached in a predominantly cohesive soil, it may be possible to extract the auger while the excavation remains stable and replace the auger with a smaller auger that can penetrate the hard ground, forming a predrilled hole that can be redrilled with the auger of the proper size. In granular soil, it may be necessary to fill the excavation with drilling slurry to maintain a stable excavation when the auger is withdrawn before reentering the excavation with a smaller auger. Another solution is to grout the pile at the depth of refusal and to install additional piles to carry the required load. The decision on how to proceed when refusal occurs can have an effect on the load-movement characteristics of the foundation and should therefore rest with the Engineer. It is best to make certain that equipment is powerful enough not to meet with refusal for any specific job, which is a reason for the *commentary under XXXX3.2. The definition of refusal provided here is based on use of the properly powered equipment, such as described under XXXX3.2.*

XXXX.7.3. Grout Placement. The placement of grout shall begin within five minutes of the completion of the excavation. Grout shall be pumped through the hollow stem auger into the excavation with sufficient pressure as the auger is withdrawn to completely fill the excavation and any soft or porous zones surrounding the excavation. A head of fluid grout of at least 1.5 m (5 feet) shall be maintained above the grout outlet orifice on the auger at all times. Simultaneous with the initial withdrawal of the auger, grout shall be placed through the grout outlet orifice into the bottom of the excavation at as high a pressure as feasible so as to drive the grout column up the flights of the auger for a distance of at least 1.5 m (5 feet), while slowly turning the auger in the same direction as was employed in excavation. This action is intended to spread the grout around the perimeter of the excavation and so aid in the removal of any loose material from the hole. Once the 1.5-m head of grout has been established within the flights of the auger, rotation of the auger should cease or be reduced to a very small rate, and extraction of the auger shall be commenced at a rate consistent with the rate at which the pump can deliver grout to the excavation.

Satisfactory operation of the coordination of auger withdrawal with grout pumping is indicated by maintaining minimum pressures in the grout at the ground surface, between pump strokes, at or above the value of total vertical pressure in the ground at the depth of the grout outlet orifice and by incrementally delivering grout to the hole in a volume equal to or greater than the theoretical incremental volume of the excavation.

Auger extraction must occur at a steady rate while continuously pumping grout under pressure into the excavation. If the foundation contractor pulls the auger at too slow a rate, the auger may become locked in the hole. If the auger is pulled at too high a rate, which will be indicated by grout pressures below the minimum grout pressures that are indicated in the paragraph above, or by insufficient grout takes, a neck may develop and the structural resistance of the pile may be compromised. Pumping of the grout under high pressure shall be continued until the cutting teeth of the auger have reached the ground surface. This will unavoidably result in some wasted grout, but it is a necessary detail in assuring that the top of the augercast pile will be structurally sound.

The volume of grout that has been placed in the excavation at the time the cutting teeth reach the ground surface shall be at least 115 per cent of the theoretical volume of the excavation, and the cutting teeth shall be visually immersed in grout when they reach the ground surface; otherwise the pile will be considered defective. In such a case the foundation contractor shall inform the Engineer immediately and proceed as directed by the Engineer.

Commentary: If the total volume of grout supplied is less than 115 per cent of the theoretical volume of the excavation and/or if the cutting teeth of the auger are not visibly immersed in grout at the completion of grouting, immediate corrective action will need to be taken by the foundation contractor if the pile is to be acceptable. In addition, if automated monitoring of incremental grout flow and pump pressure is performed and the grout placed is less that 97 per cent of the incremental theoretical volume for any 0.61-m increment of the pile or if the average minimum pump pressure is less than the average total vertical pressure in the ground for any 0.61-m depth increment and the average maximum pump pressure for any 0.61-m depth increment is less than 1550 kPa (225 psi), the pile should be considered as unreliable, which requires immediate action on the part of the foundation contractor. These considerations are not dependent upon whether the material being excavated are able to retain the shape of the excavation without support from the soil-filled auger. They apply to all soil conditions.

An acceptable corrective measure is to reinstall the auger to a depth of at least 3 m (10 feet) into the grout column, or to the bottom of the pile, whichever is less, and regrouting as if the pile were being excavated for the first time. The same conditions for acceptance of the regrouted pile as were applied to initial construction should be used.

XXXX.7.4. Surface Cleaning and Protection. Immediately upon completion of placement of the fluid grout, the foundation contractor shall remove all excess grout and spoil from the vicinity of the top of the excavation and shall place a suitable temporary device within the top of the excavation, extending above the ground surface by at least 0.3 m (1 foot), to keep surface spoil from entering the grout column before the grout sets. It shall be removed without disturbing the natural soil surrounding the top of the pile once the grout has set. Following placement of this device the foundation contractor shall remove any and all loose soil that has fallen into the grout column with a suitable tool before the grout begins its initial set.

XXXX.7.5. Reinforcing Steel Placement. The Contractor shall be responsible for furnishing the reinforcing steel and any anchor bolts or dowels shown on the plans. Any required reinforcing steel shall be placed as shown on the plans by lowering the cage within the grout column within 30 minutes of completion of the placement of grout.

The reinforcing steel shall be free of oil, soil, excessive rust or other deleterious material and shall be centered in the excavation with non-metallic centralizers acceptable to the Engineer.

If cages of reinforcing steel are called for on the plans, the longitudinal bars and lateral reinforcement (spiral or horizontal ties) shall be completely assembled and placed as a unit. Where spiral reinforcement is used, it shall be tied to the longitudinal bars at a spacing not to exceed 0.3 m (1 foot) unless otherwise shown on the plans. Welding of lateral reinforcement to longitudinal bars will not be permitted unless otherwise shown on the plans.

The reinforcing steel shall not be spliced except at locations that are shown on the plans, and the reinforcing steel shall be free of any permanent distortion, such as bars bent by improper pickup. If a pile is required by the Engineer to be lengthened after the steel has been cut and cages have been assembled, the schedule of reinforcing steel, both longitudinal and lateral, shall be extended to the bottom of the pile by splicing. Splices should be as close to the bottom of the pile as possible. Accomplishment of splicing by welding shall not be permitted unless otherwise shown on the plans.

The reinforcing steel shall be placed in the grout column immediately after screening the grout and before the grout begins to take its initial set. The steel may be lowered into the grout by gravity or pushed gently to final position by the foundation contractor's personnel. The reinforcing steel shall be centered in the excavation by means of plastic or cementitious spacers placed at sufficient intervals along the pile and at sufficient intervals around the steel to keep the steel centered. Metallic spacers shall not be permitted.

Commentary: If steel spacers are used, corrosion of the reinforcing steel can be greatly accelerated, particularly above the ground water table. Therefore, they should be avoided.

The reinforcing steel shall not be vibrated or driven into position without the approval of the Engineer.

The reinforcing steel shall be held in position within the fluid grout column by appropriate supports at the ground surface, which shall remain in place until the grout reaches a minimum of 50 per cent of its design strength, or three days, whichever occurs first.

XXXX.8. <u>Inspection and Records</u>. The Contractor shall maintain accurate records for each pile constructed. Similar records will be maintained by the Engineer. These records shall show:

a. Pile location;
b. Ground surface elevation;
c. Pile toe (bottom) elevation;

d. Elevation of top of grout;
e. Pile length;
f. Auger diameter;
g. Flow cone efflux time and volume of grout placed;
h. Theoretical volume of excavation (diameter = diameter of auger);
i. Depth to which reinforcing steel was placed;
j. Date/Time of beginning of drilling;
k. Date/Time of completion of drilling;
l. Date/Time grout was mixed;
m. Date/Time ready-mix grout truck arrived at jobsite;
n. Date/Time of beginning of grout pumping;
o. Date/Time of completion of grout pumping;
p. Date/Time of placement of reinforcing steel;
q. Weather conditions, including air temperature, at time of grouting;
r. Identification of grout samples taken from the pile, if any, and
s. All other pertinent data relative to the pile installation.

Piles that support critical structures that are designated on the plans, or as otherwise required by the Engineer, are to be monitored using automated equipment. For such piles the following records shall be made and retained by the Contractor.

a. Volume of grout placed versus depth of grout outlet orifice for every 0.61 m (2 foot) increment, or less, of pile placed.

b. Average maximum and minimum pump stroke pressures at ground level for every 0 .61 m (2 foot) increment, or less, of pile placed.

These data shall be provided to the Engineer in graphical form within 24 hours of the completion of the pile.

Post-installation structural integrity tests of the piles may be specified. If so, those piles on which such tests are to be conducted will be designated on the plans, and the specific test(s) to be performed will be designated on the plans. Such tests include, but are not limited to, sonic echo tests, impulse-response tests, cross-hole sonic or ultrasonic tests; backscatter gamma tests, fiber-optic television camera tests, and high-strain integrity tests. If such post-installation integrity tests are called for on the plans, the Contractor shall engage an independent consultant, acceptable to the Engineer, to perform those tests and to report the results, with interpretations, to the Contractor and the Engineer. The Contractor shall install access tubes, of a design acceptable to the consultant, to accommodate those tests that require access to the interior of the augercast pile. These tubes shall be secured to the reinforcing steel prior to placing the steel in the fluid grout.

Commentary: Automated monitoring of incremental grout volumes and pressures is a key element in assuring the structural integrity of augercast piles. Such monitoring should be carried out on all bearing piles for critical structures, such as bridge and retaining wall foundations. Such monitoring may also be carried out for selected, representative piles for noncritical structures, such as sound wall and sign foundations.

Post-installation integrity tests are valuable in establishing that a foundation contractor's procedures are producing acceptable piles on any given job. The most reliable of the post-installation integrity tests for identifying anomalies within the pile are those that use down-tube instruments, such as the cross-hole sonic or ultrasonic test, the backscatter gamma test and the fiber-optic television camera test. These tests all require that the foundation contractor attach appropriate access tubing to the reinforcing steel prior to placing the steel in the grout column. They also require intelligent interpretation, which should be performed by experts. Such experts cannot always determine whether an anomalous reading is a defect within the pile, however, and the final decision on acceptability of the pile must be made by the Engineer, based on construction records, the post-installation integrity test expert's report and upon the

Engineer's analysis of the possible effect on foundation performance of the potential defect.

In order to be effective, access tubes should be distributed evenly circumferentially around a reinforcing cage at a frequency of approximately one for every 0.3 meters (1 foot) of cage diameter, but not less than two tubes. It is advisable that tubes used for cross-hole sonic or ultrasonic tests be made of Schedule 40 steel because such tubes will remain bonded to the grout. Polyvinyl chloride (PVC) tubes do not ordinarily remain bonded to the grout beyond a few days after the grout takes its initial set, and debonding will render the cross-hole sonic / ultrasonic tests ineffective. PVC tubes should be used only for backscatter gamma testing unless cross-hole sonic / ultrasonic tests will be performed within 72 hours of casting the grout.

XXXX.9. Unacceptable Piles. Unacceptable piles are defined as piles that will not carry their intended load with allowable deflections. The following constitute construction conditions that produce unacceptable piles:

a. Pile that is out of position by more than 25 mm (1 inch) at the ground surface or not within the plumbness or batter limits defined in Item XXXX.7.1.

b. Pile in which the top of the grout is more than 25 mm (1 inch) below or 75 mm (3 inches) above the elevation shown on the plans.

c. Piles in which the grout strength is less than that required.

d. Piles in which the steel was not inserted as required.

e. Piles that exhibit any visual evidence of grout contamination, structural damage or inadequate consolidation (honeycombing).

f. Piles that are inspected using post-installation integrity testing methods that are judged by the Engineer to be unacceptable.

Unacceptable piles shall be replaced or repaired at the Contractor's expense, as directed by the Engineer.

XXXX.10. Load Tests. Any required load testing of augercast piles shall be in accordance with Item 405, "Foundation Test Load."

Commentary: Expedient load testing methods not covered under Item 405 can also be used to determine the load-carrying capacities of augercast piles if specified by the Engineer. These methods include driving of the completed pile with concurrent measurements of set, stress and velocity at the pile head and subsequent wave-equation analysis of the data to interpret pile capacity, and the Statnamic™ test, in which the pile is pushed rapidly into the soil in such a manner that the capacity can be determined by appropriate analysis of the measured load-movement curve.

XXXX.11. Measurement. Augercast piles shall be measured by the meter between the top of the grout and the bottom of the pile. If load tests are specified, the will be paid as a lump sum per load test.

XXXX.12. Payment. The work performed and materials furnished in accordance with this Item and measured as provided under XXXX.11 ("Measurement") will be paid for at the unit prices bid under the payment categories listed below.

Payment categories:

a. Per linear meter of augercast piling of the specified diameter placed without automated monitoring or post-installation integrity testing;

b. Per linear meter of augercast piling of the specified diameter placed with automated monitoring as indicated on the plans and without post-installation integrity testing

c. Per linear meter of augercast piling of the specified diameter placed without automated monitoring but with post-installation integrity testing as indicated on the plans.

d. Per linear meter of augercast piling of the specified diameter placed with automated monitoring and with post-installation integrity testing as indicated on the plans.

e. Per load test.

The quantities to be paid for will be the quantities in each category shown on the plans unless specific changes are required in writing by the Engineer. Unit prices that are bid will apply to the extension of any pile to a depth up to 120 per cent of the depth for that pile that is shown on the plans when such an increase in depth is required by the Engineer. If subsurface conditions dictate that any pile is to be installed to a depth less than that shown on the plans, and the decrease in length is approved in writing by the Engineer, the length of pile actually constructed will be paid for at the unit price bid. If increases in depth exceeding 120 per cent of the depth shown on the plans are required by the Engineer, or if diameters other than those that are shown on the plans are required by the Engineer, the unit prices shall be renegotiated for those piles involved.

Commentary: If the total length of all piling installed on the job is less than the total length shown on the plans because of field decisions by the Engineer, regardless of the shortfall , the Contractor will be paid only for the lengths actually installed at the unit prices bid.

Commentary: This Item applies to augercast piles constructed in predominantly cohesive soil profiles, in which research has been performed for the Texas Department of Transportation. Its applicability to cohesionless soils or to rocks is unproved (1997). Additional drilling controls and payment items may be needed in such subsurface conditions.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The behavior of laterally loaded CFA piles in stiff clay was investigated. That investigation is documented in this report. Field lateral loading tests were conducted on four full-sized CFA piles. Shortly after construction of the test piles, they were subjected to ultrasonic and fiber-optic integrity tests. The results of these tests showed that test piles were properly installed. This confirmed the data obtained by the Pile Installation Recorder during the construction of the test piles. The results of the field loading tests and the integrity test are presented in Chapter 2. These results were utilized, in Chapter 3, to synthesize appropriate p-y curves for the laterally loaded CFA piles in stiff clay. In Chapter 4, a simplified design method was then developed on the basis of the synthesized p-y curves. An example problem was introduced to illustrate the use of the simplified method for design of CFA piles supporting sound barriers in stiff clay.

In Chapter 5, the results of an experimental study of CFA grout behavior was presented. The results of the study showed that the working and mechanical properties of the field grout mix are not improved significantly by the addition of fibers or by increasing the fly ash content of the mix. They also showed that tensile strength of the CFA piles grout is substantially less than that recommended by the ACI for concrete. This experimental fact was accounted for in developing the simplified design method, and it should be born in mind in if drilled-shaft-oriented software, which may use tensile strengths for concrete, is to be used to design CFA piles.

A preliminary construction specification is included in Chapter 6. This specification is quite detailed because loss of quality control in CFA pile construction may result in serious deficiencies in the foundation. Only contractors who are qualified and prepared to follow these specifications should be permitted to perform CFA pile work for TxDOT.

Recommendations

While this study provided valuable information on the construction of CFA piles and on the design of CFA piles under lateral loading in stiff clay, it was limited in its scope. Pile construction and lateral load behavior may be different in other geologic settings within Texas. TxDOT should continue to monitor critically the construction of CFA piles in coastal Texas clay soils as well as in sands, gravels, mixed soils, and soft rock, as the use of CFA piles increases. As new information is acquired, it is fully expected that the construction specification will be modified and the design method will be improved by TxDOT personnel.

The study did not address the axial behavior of CFA piles, other than through the literature review and the construction specification. Further studies of the axial resistance and settlement of CFA piles constructed under the provisions of the recommended specification are warranted if TxDOT plans to use CFA piles as bearing piles to support structures. Such studies should include the performance of closely controlled full-scale loading tests at sites where soil properties have been carefully determined in order to develop correlations for design.

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Appendix A

Supporting Field Test Data

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A.1. Profile of Lateral Deflections Along the Test Piles

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The profile of lateral deflections along the test pile were measured by a digitilt inclinometer. The lateral deflections along the pile were corrected by multiplying the raw inclinometer data according to the following equation:

$$y = y_m * y_t / y_{tm} \qquad \text{(A.1)}$$

where

y = corrected lateral deflection at a depth = d,

y_m = raw lateral deflection, as measured by the inclinometer, at a depth = d,

y_t = ground-line deflection, as measured by a dial gage and

y_{tm} = raw ground-line deflection, as measured by the inclinometer.

The measured and corrected profiles of deflections for the test piles are shown in Figs. A.1 through A.12.

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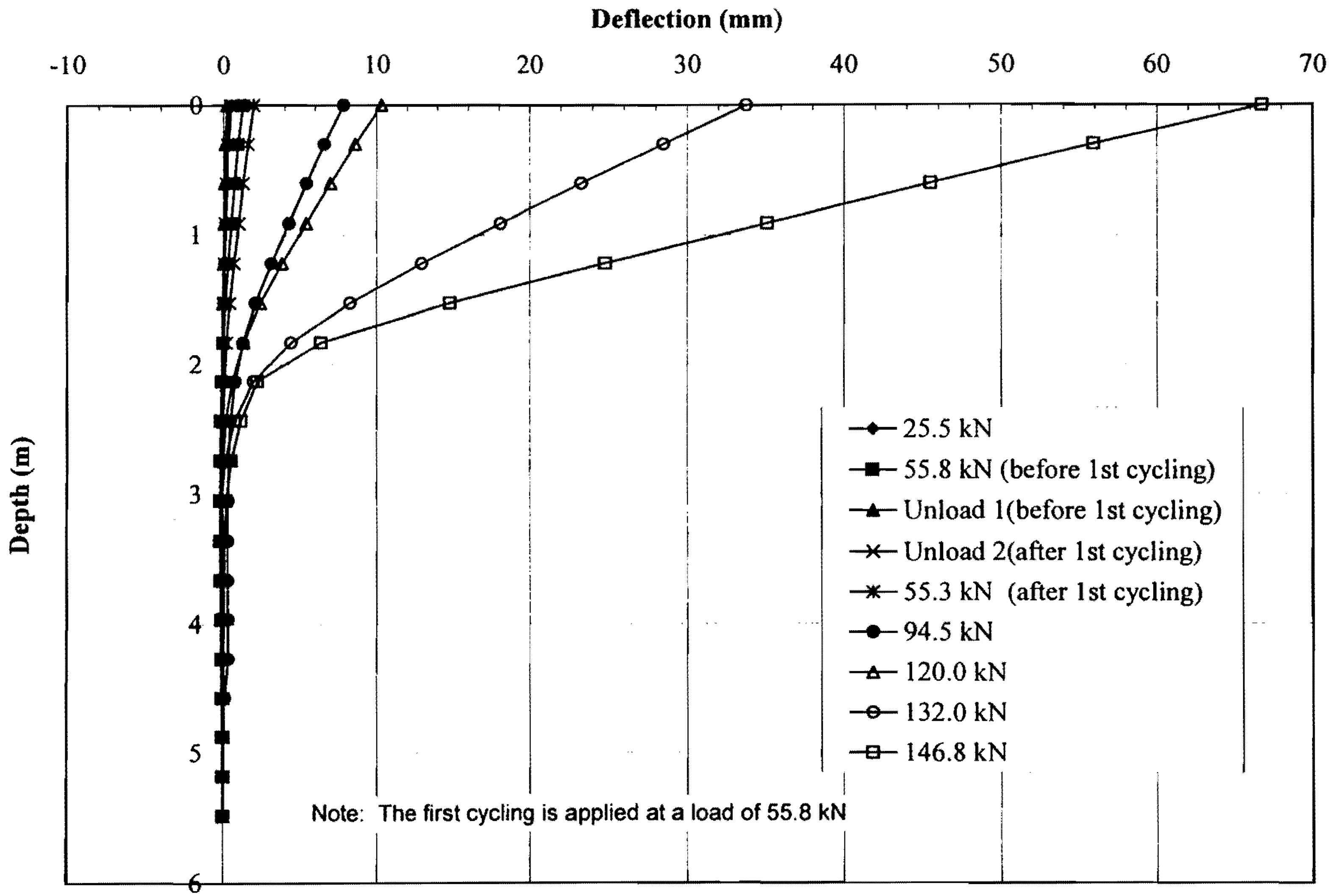

Fig. A.1. Measured profile of lateral displacement along Pile East

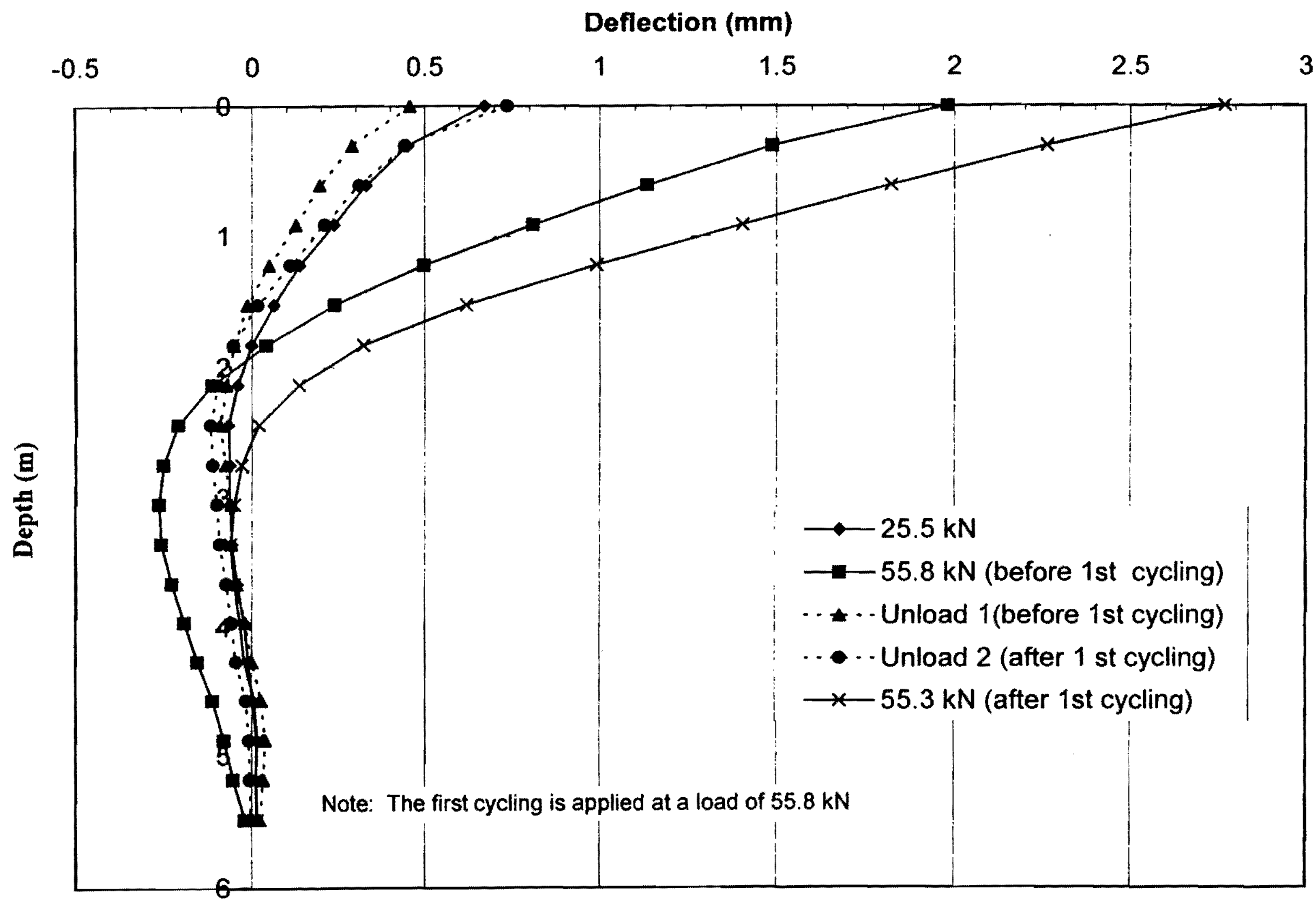

Fig. A.2. Corrected profile of lateral displacement along Pile East (Loading range within design load)

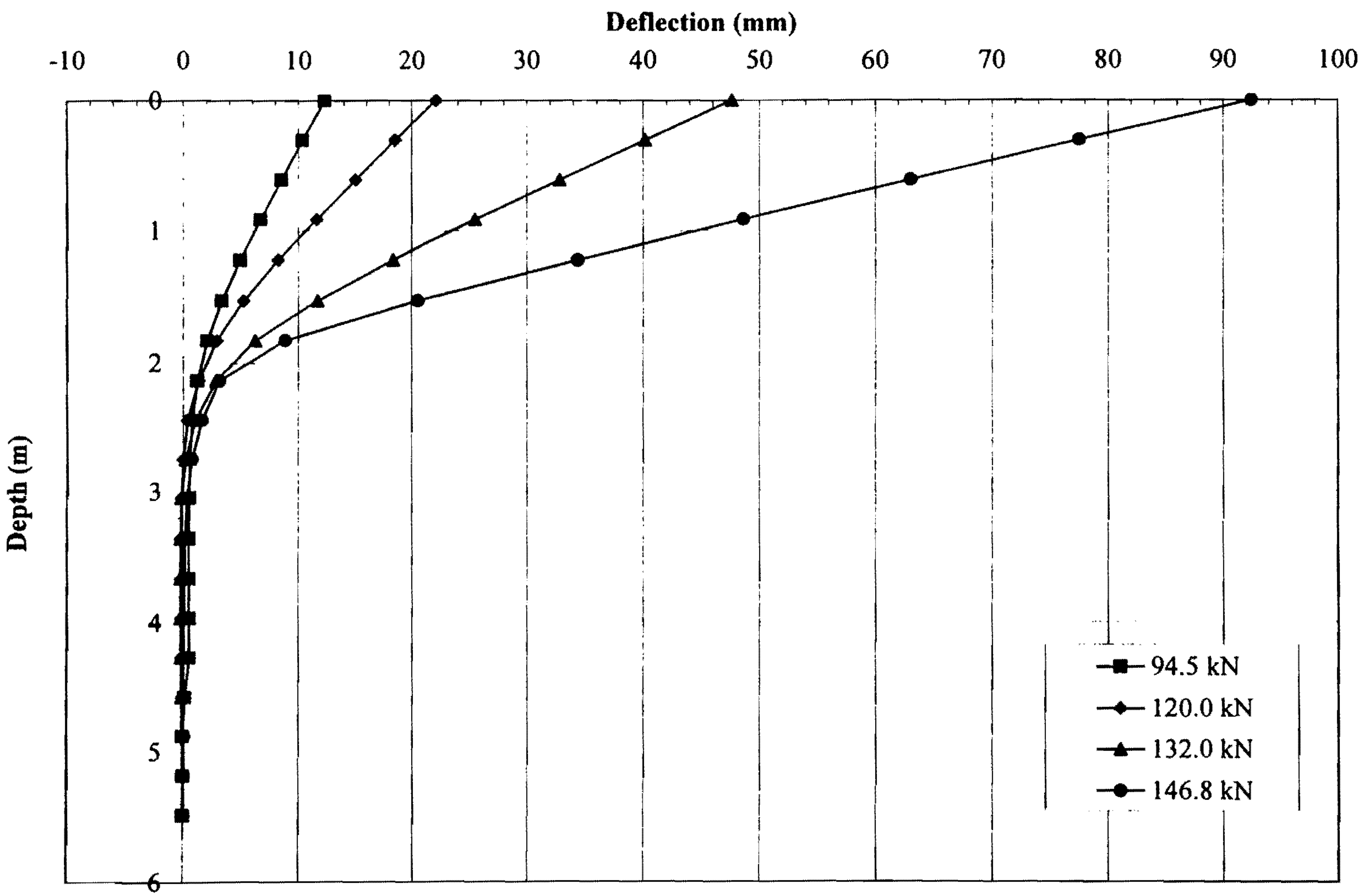

Fig. A.3. Corrected profile of lateral displacement along Pile East (Loading beyond design load)

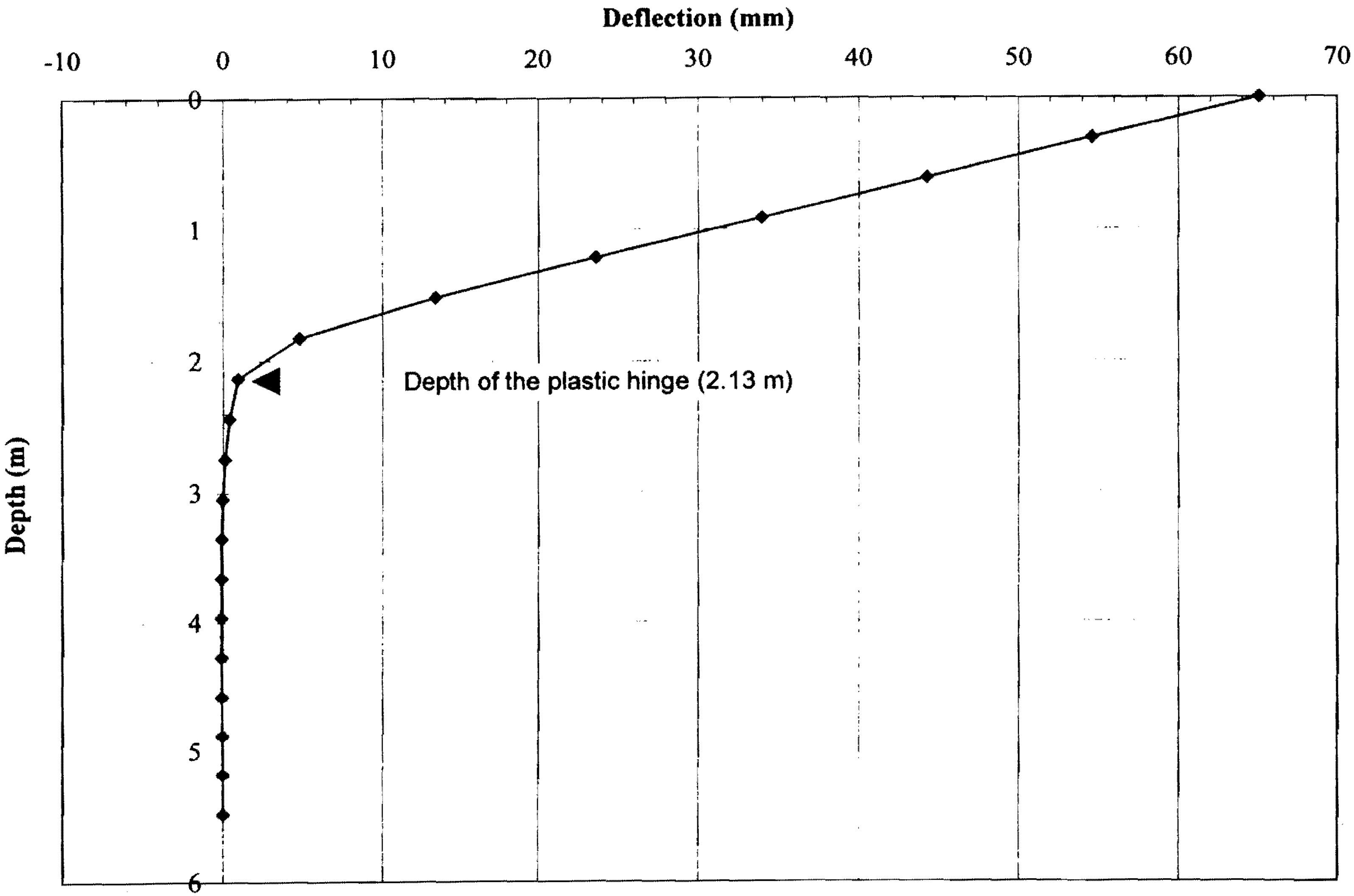

Fig. A.4. Corrected profile of permanent lateral displacement along Pile East

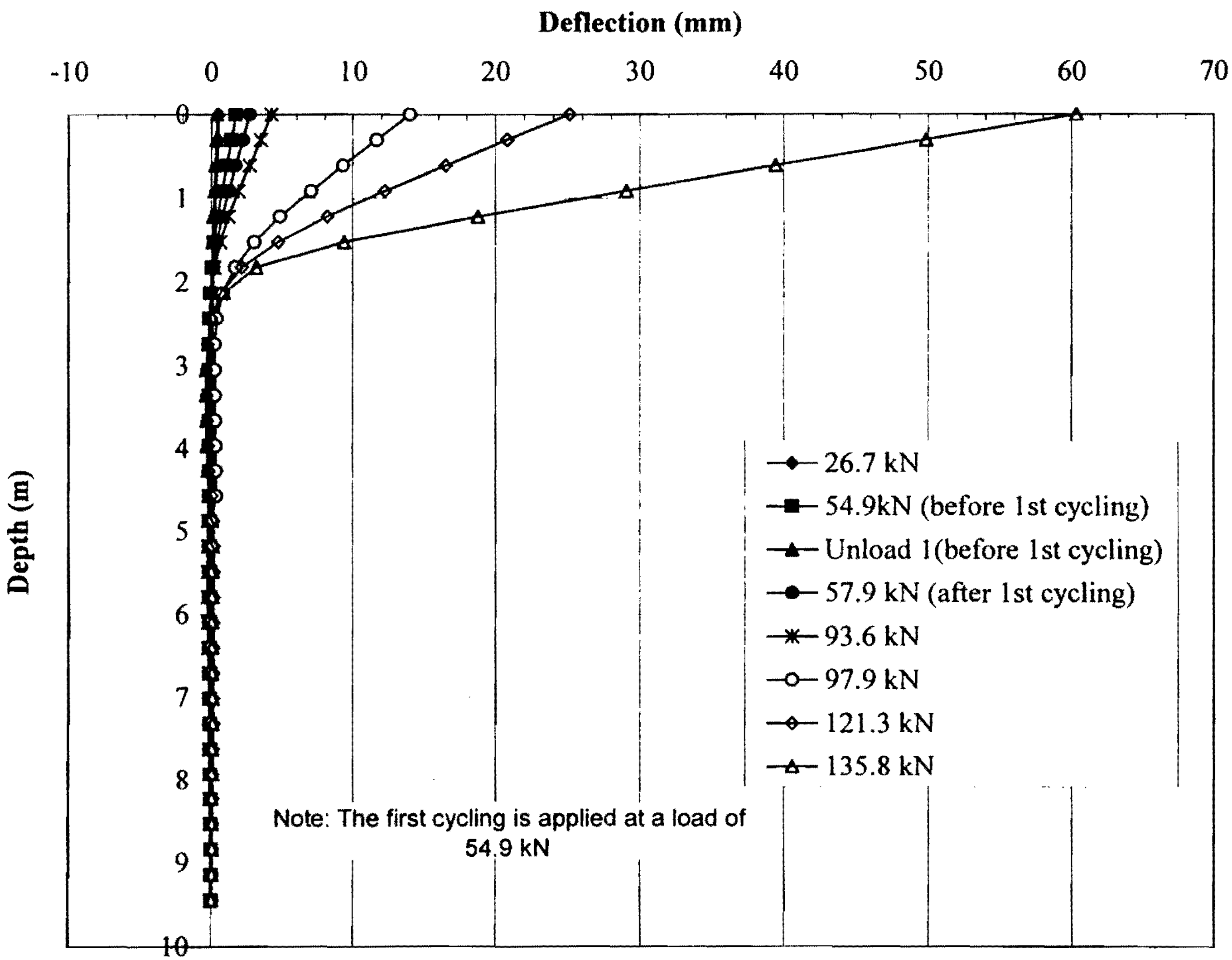

Fig. A.5. Measured profile of lateral defelection along Pile West

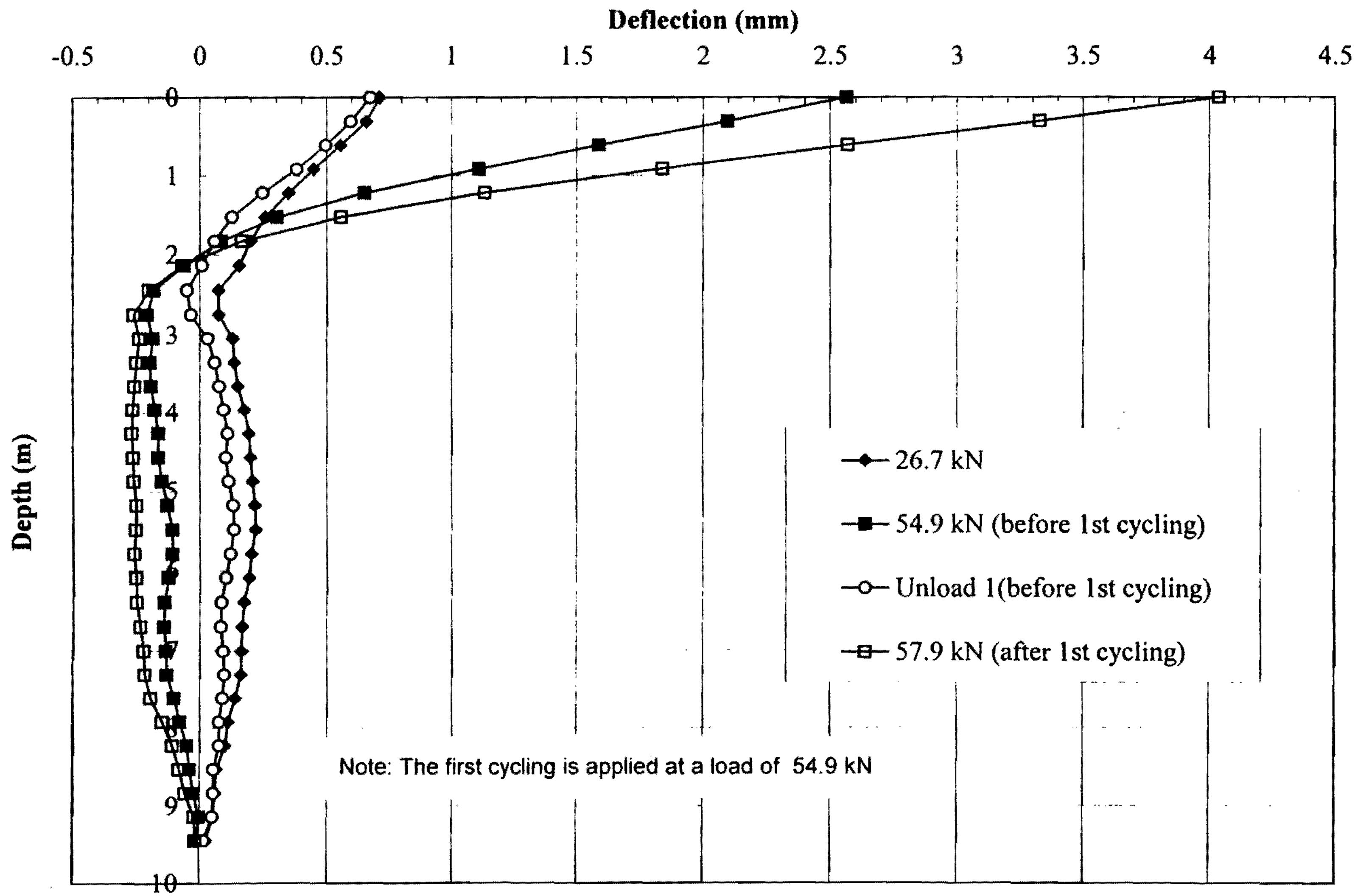

Fig. A.6. Corrected profile of lateral displacement along Pile West (Loading within design load)

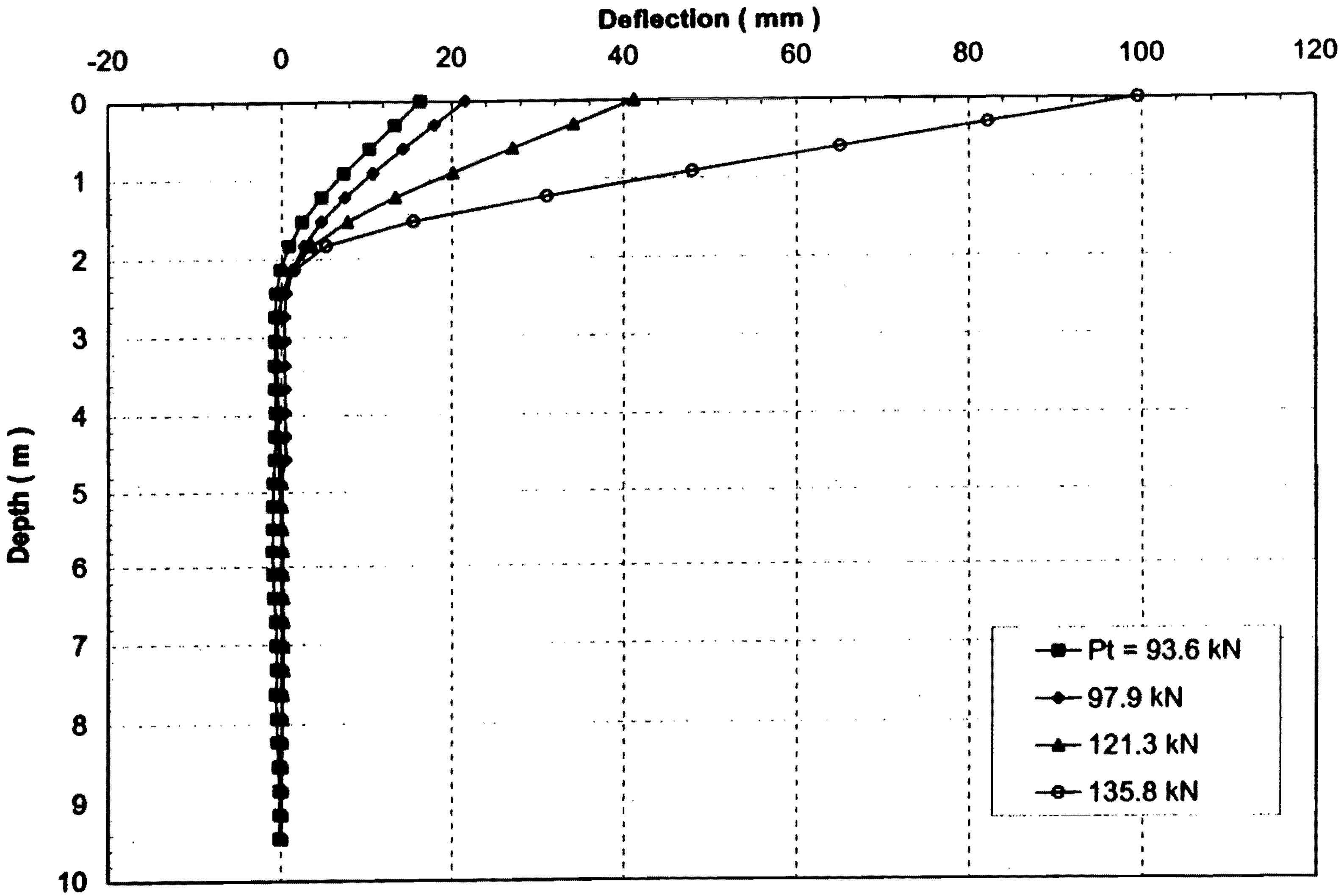

Fig. A.7. Corrected profile of lateral displacement along Pile West (Loading range beyond design load)

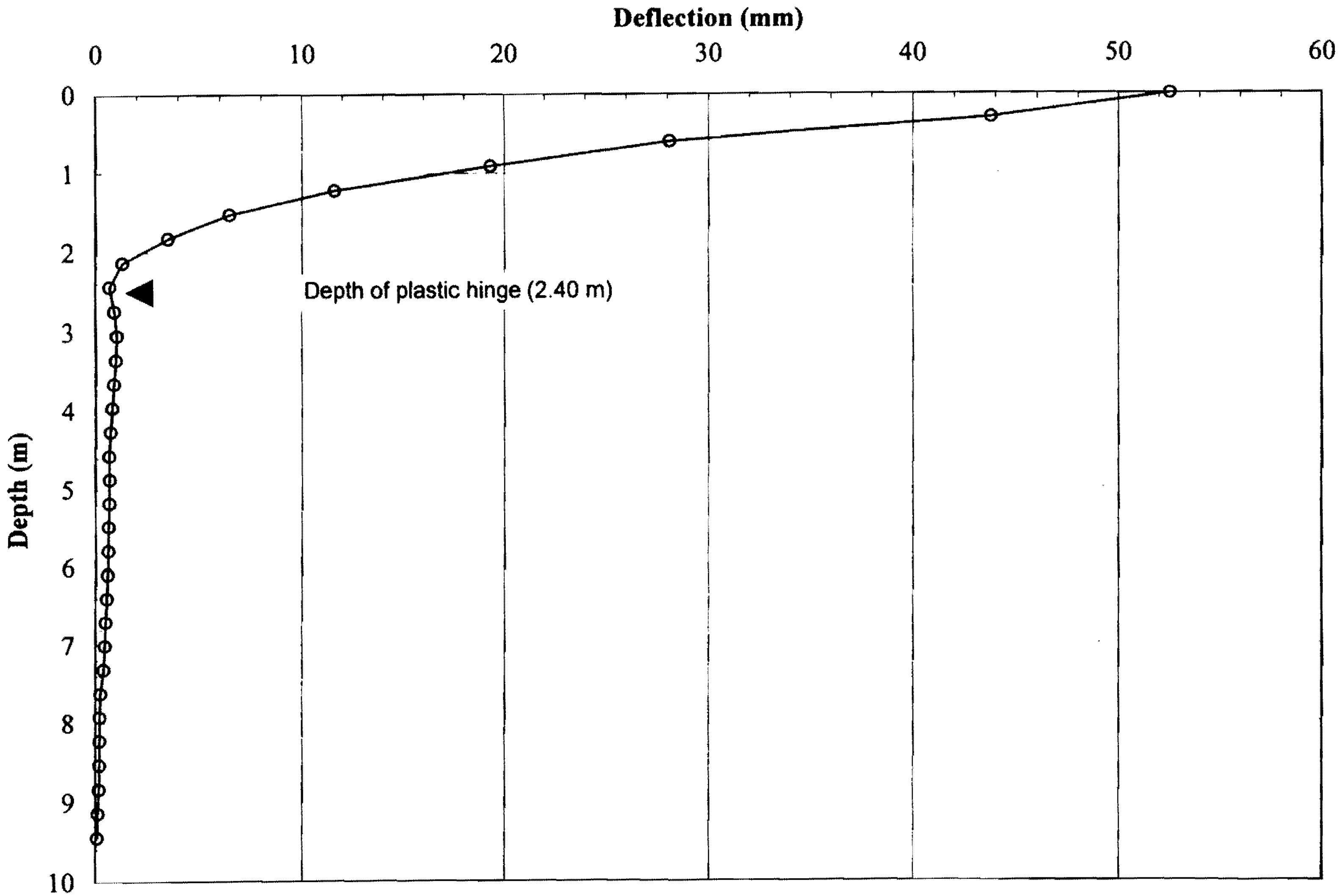

Fig. A.8. Corrected profile of permanent lateral displacement along Pile West

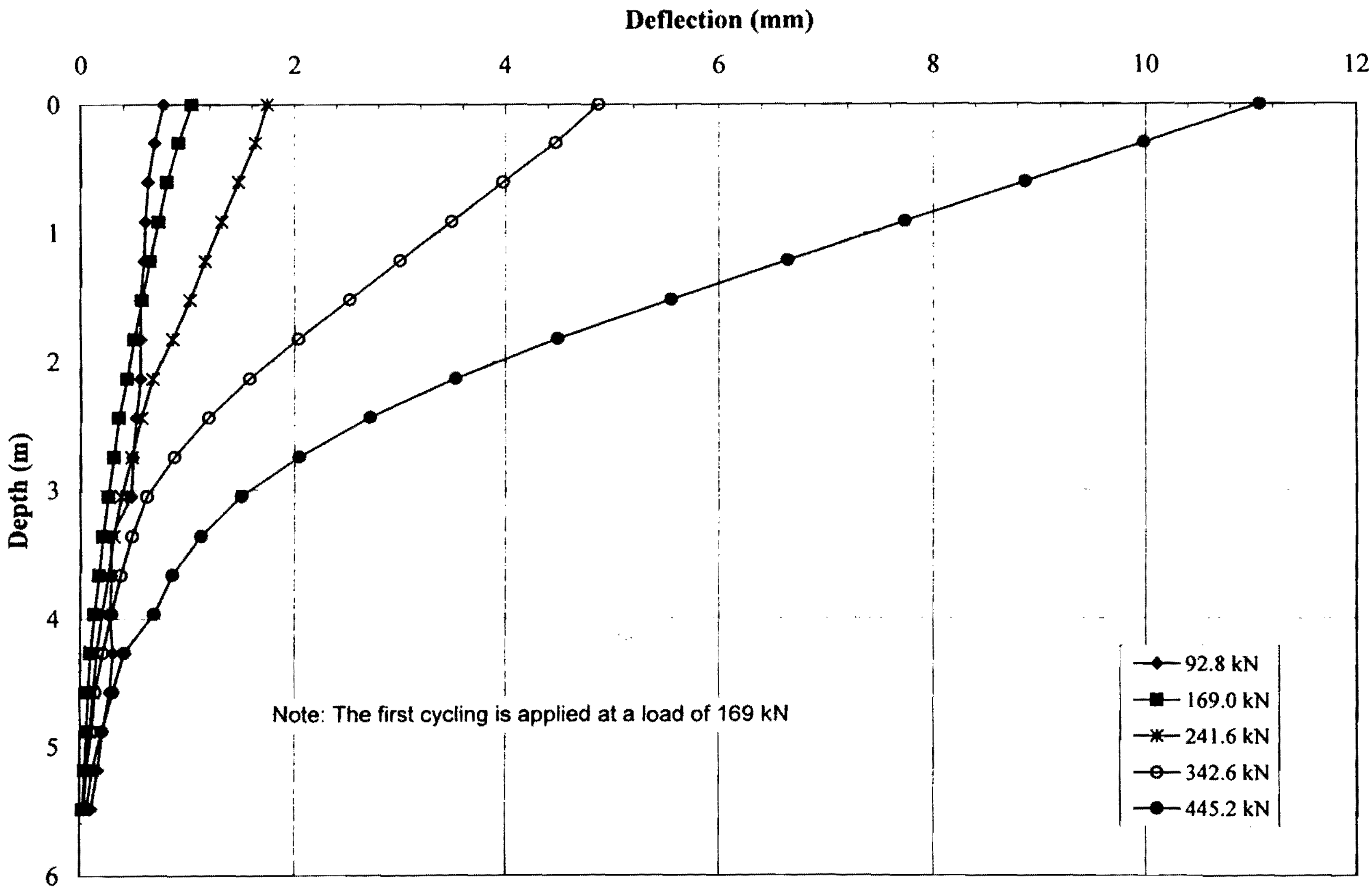

Fig. A.9. Measured profile of lateral deflection along Pile South

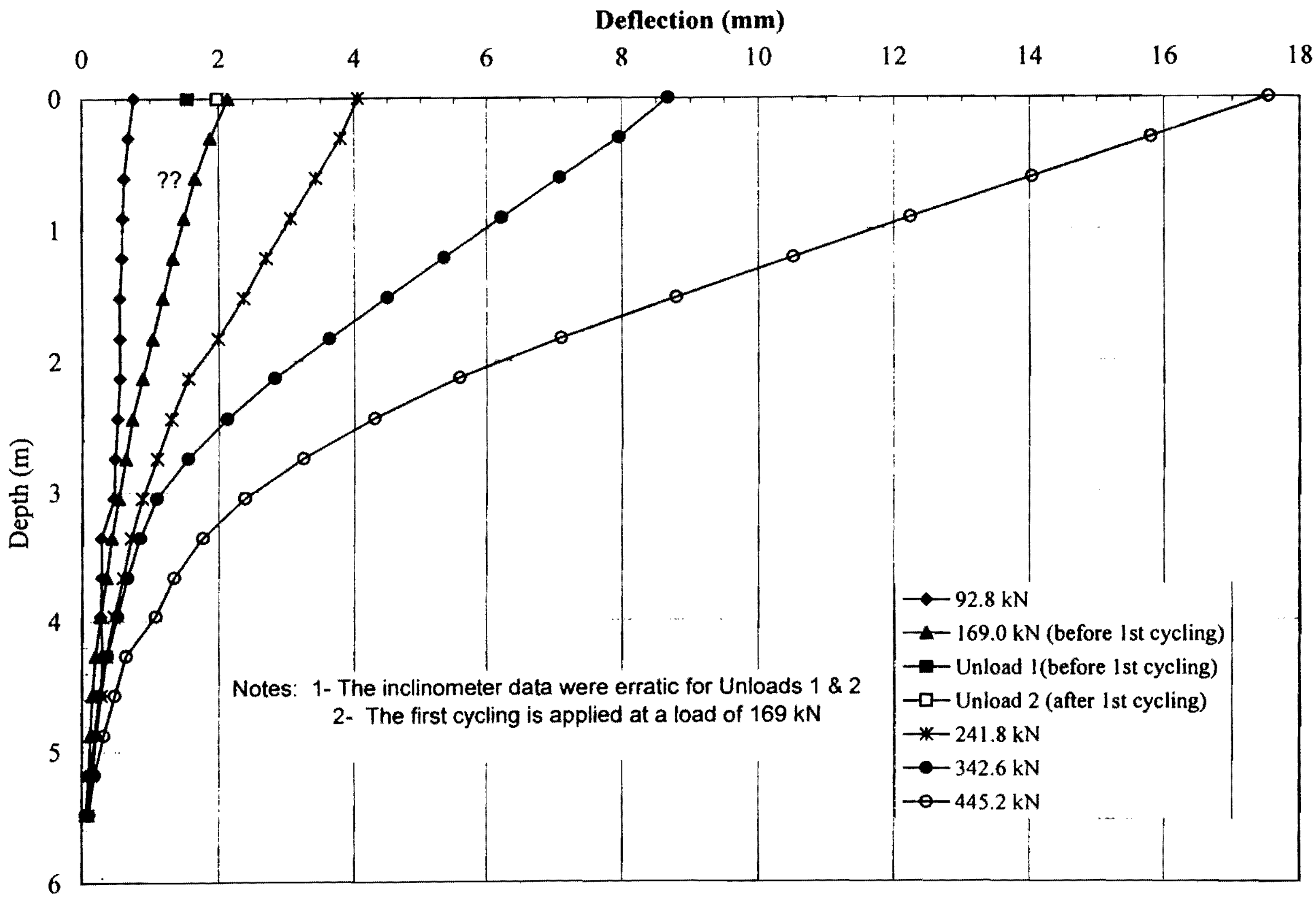

Fig. A.10. Corrected profile of lateral displacement along Pile South

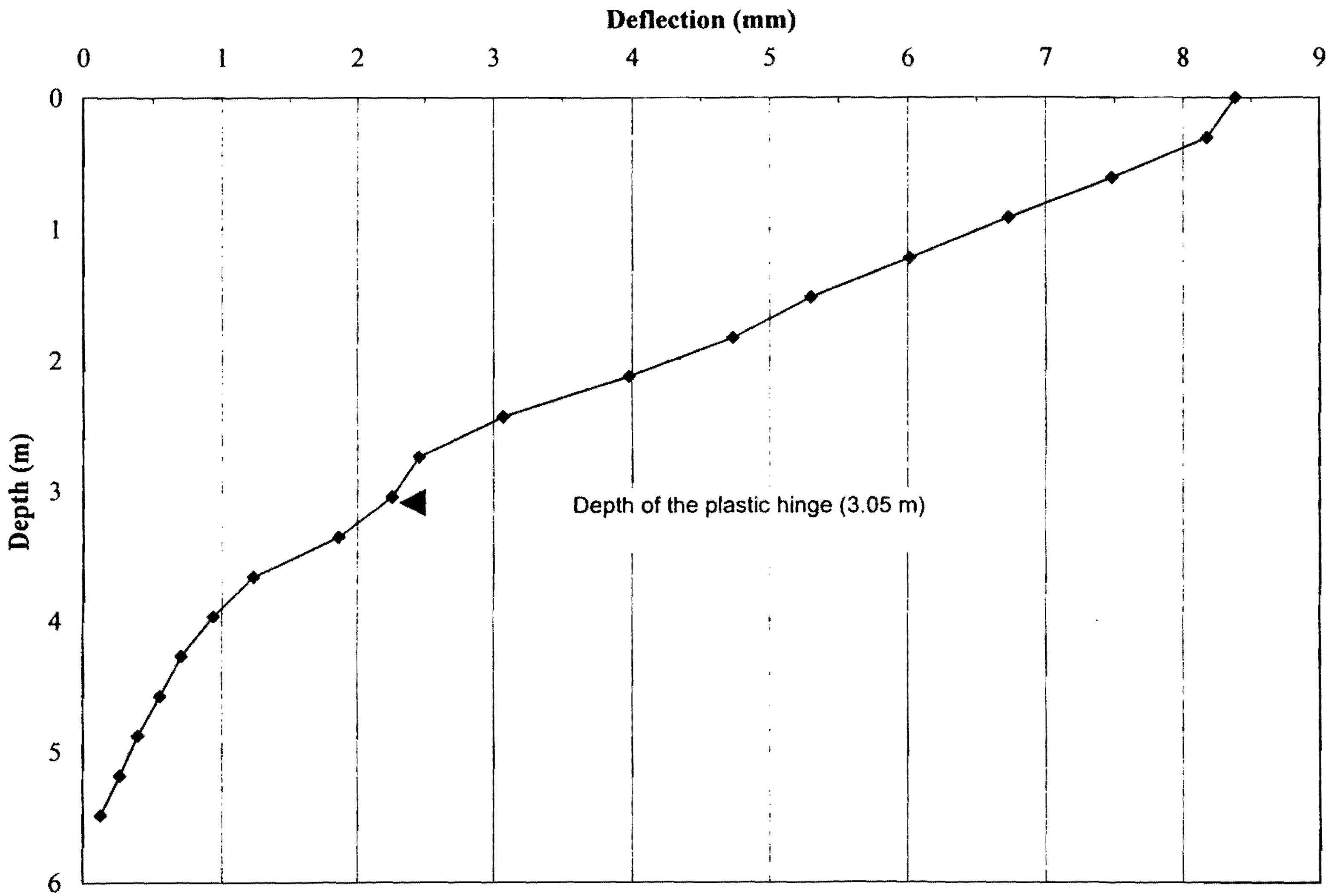

Fig. A.11. Corrected profile of permannent lateral displacement along Pile South

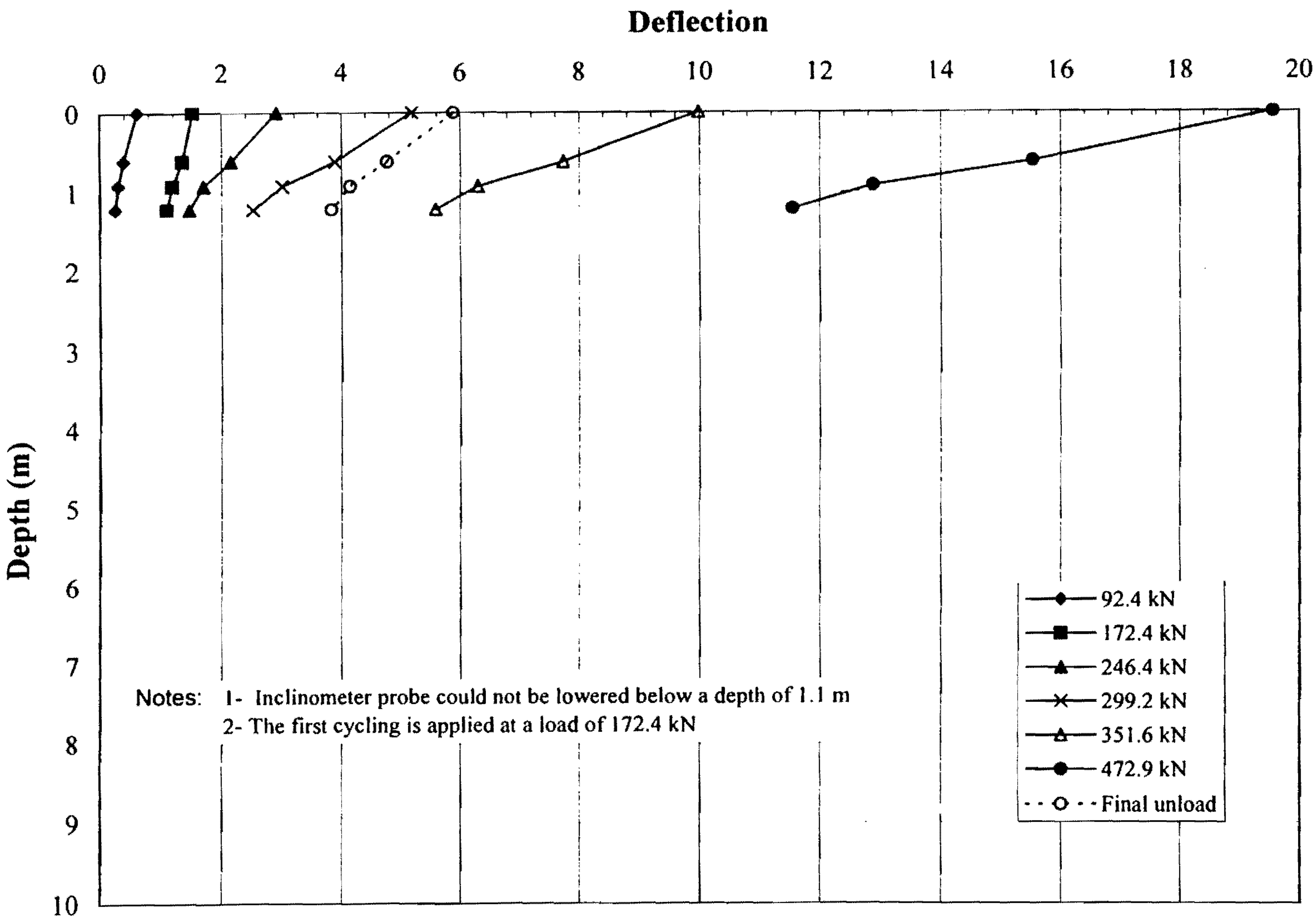

Fig. A. 12. Measured profile of lateral displacement along Pile North

A.2. Ultrasonic Logs of the Test Piles

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The ultrasonic logging was conducted under both wet and set grout conditions. The former was accomplished immediately after the construction of each test pile, and the latter was performed three days after construction. In both cases, cross-hole and single-hole tests were performed. The wet grout tests were not effective in this study. The results of the ultrasonic tests under set grout conditions are shown in Figs. A. 13 through A.20.

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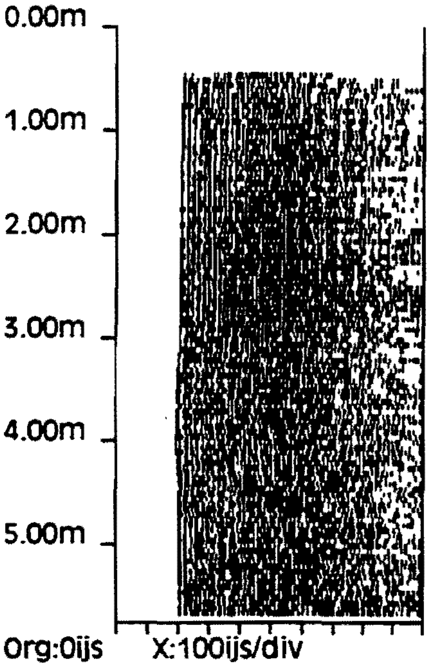
Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : D

File : DS
Page : 1
Date : Feb 02 1997

Profile : N
Recorded : Dec 22 1996
Distance : 20cm
Length : 5.75m



Profile : S
Recorded : Dec 22 1996
Distance : 20cm
Length : 5.55m

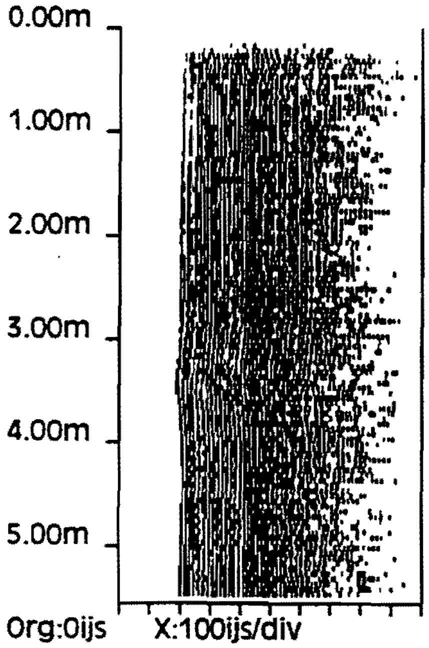


Fig. A.13. Single-hole ultrasonic log of Test Pile East

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES Site : UH APG Report by : TIM ROBERTS Pile ID : D	File : D1 Page : 1 Date : Feb 02 1997

Profile : NS
Recorded : Dec 21 1996
Distance : 25cm
Length : 6.15m

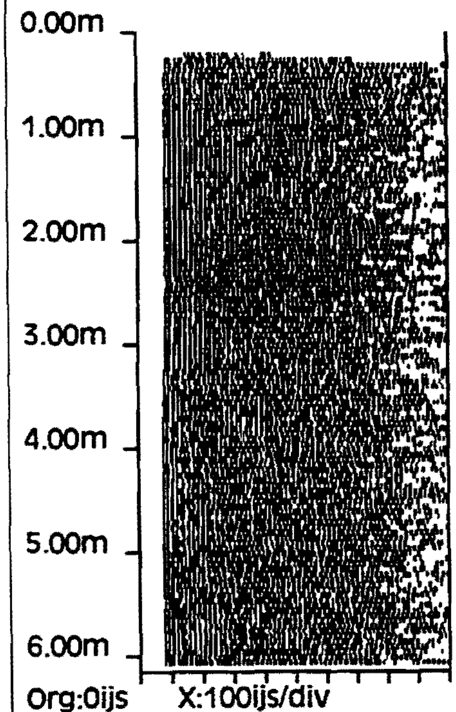
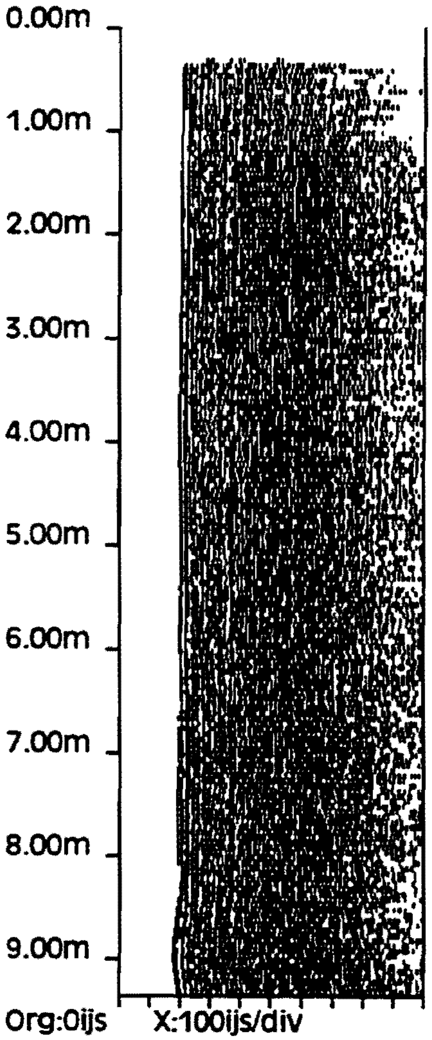


Fig. A.14. Cross-hole ultrasonic log of Test Pile East

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : C

File : CS
Page : 1
Date : Feb 02 1997

Profile : N
Recorded : Dec 22 1996
Distance : 20cm
Length : 9.35m



Profile : S
Recorded : Dec 22 1996
Length : 9.95m

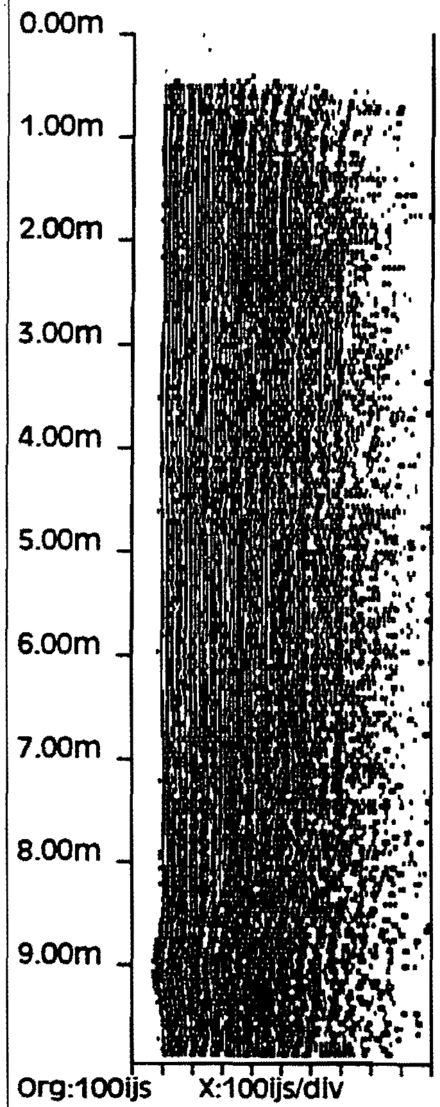


Fig. A.15. Single-hole ultrasonic log of Test Pile West

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : C

File : C1
Page : 1
Date : Feb 02 1997

Profile : NS
Recorded : Dec 21 1996
Distance : 25cm
Length : 10.10m

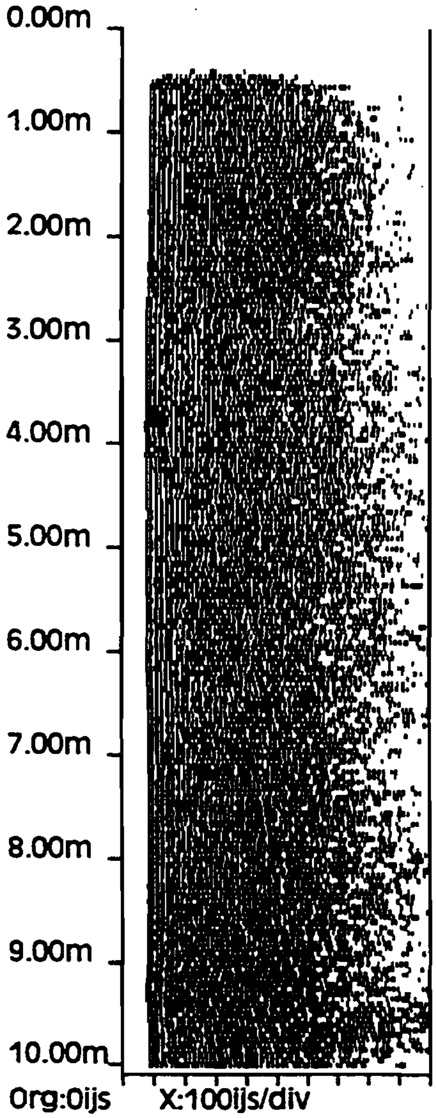


Fig. A.16. Cross-hole ultrasonic log of Test Pile West

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : B

File : BS2
Page : 1
Date : Feb 02 1997

Profile : W2
Recorded : Dec 22 1996

Length : 5.55m

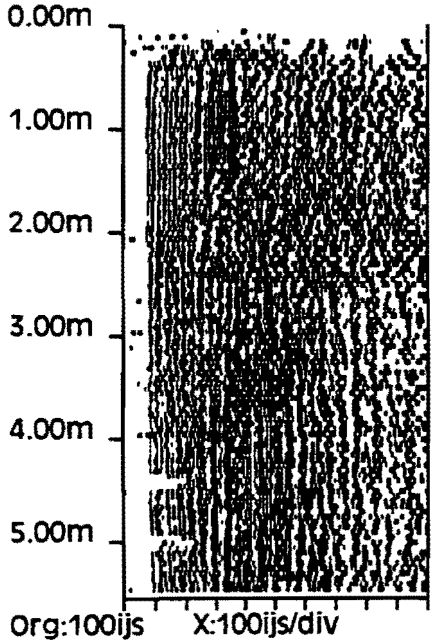


Fig. A.17. Single-hole ultrasonic log of Test Pile South

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : B

File : B1
Page : 1
Date : Feb 02 1997

Profile : EW
Recorded : Dec 22 1996
Distance : 58cm
Length : 6.20m

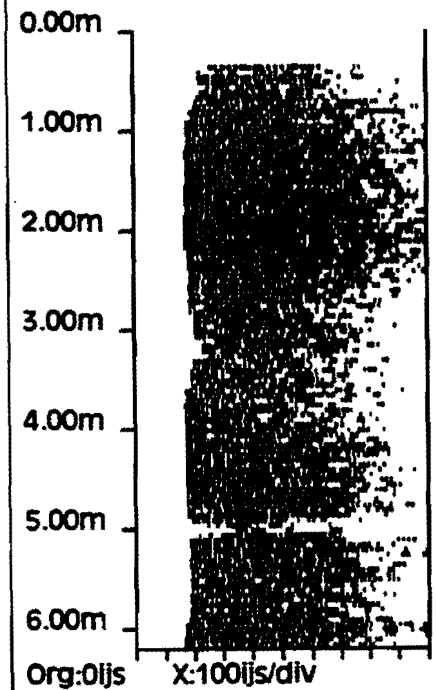


Fig. A.18. Cross-hole ultrasonic log of Test Pile South

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : A

File : A6
Page : 1
Date : Feb 02 1997

Profile : W
Recorded : Dec 22 1996
Distance : 20cm
Length : 10.05m

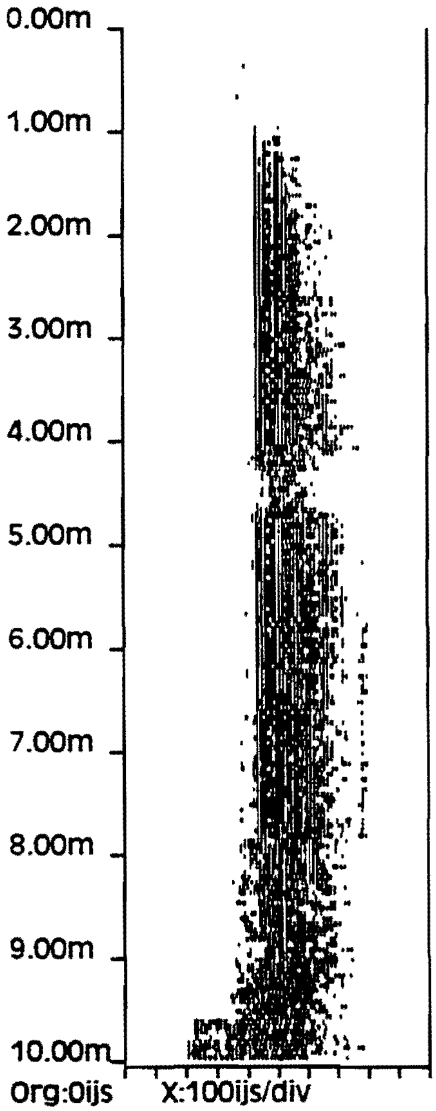


Fig. A.19. Single-hole ultrasonic log of Test Pile North

Project : UNIV.HOU. AUGER CAST-IN-PLACE TEST PILES
Site : UH APG
Report by : TIM ROBERTS
Pile ID : A

File : A1
Page : 1
Date : Feb 02 1997

Profile : EW
Recorded : Dec 22 1996
Distance : 50cm
Length : 8.20m

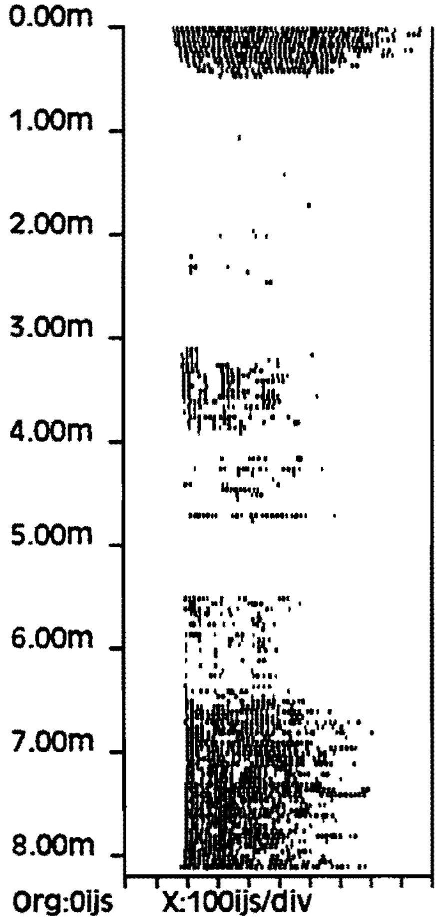


Fig. A.20. Cross-hole ultrasonic log of Test Pile North

Appendix B

Survey of DOT Practice

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Survey of DOT Practice

State and federal DOT practice regarding augercast piles was surveyed in April, 1997. Letters were sent to the geotechnical offices of all 50 state departments of transportation, the Puerto Rico Department of Transportation, and the Federal Highway Administration (FHWA). A copy of the survey letter is included in this appendix. Responses were received from 20 states and from the Federal Lands Office of the FHWA. Three inquiries were returned because of faulty addresses or transferred personnel (California, Wisconsin and Puerto Rico). The return rate on the completed survey was 21 / 52 or 40 per cent (as of the closing date of July 21, 1997), which is considered good.

Table B.1 summarizes the results of the survey. Four states, including Texas, responded with copies of standard or draft construction specifications for augercast piles. These specifications are all reproduced in this appendix. A reference for an industry guide specification, which is not reproduced here because of copyrighting laws, is also given. These state specifications and the industry guide specification were carefully reviewed in preparation for developing the preliminary specification for the Houston District contained in Chapter 6.

Only four of the agencies responding to the survey, including Texas, permit the use of augercast piles (DOT's of Florida, Nebraska, Kansas and Texas). Florida and Texas restrict their use to sound walls at present. Kansas and Nebraska will allow their use in bridges and retaining walls in unusual circumstances. Most of the remaining respondents simply indicated that they did not use augercast piles, but a few states (Kentucky, South Carolina, Mississippi) prohibit their use. The primary concern among respondents who expressed concern was that structural integrity of the completed pile was not verifiable.

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Survey Letter

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April 21, 1997

Dear Colleague,

The Texas Department of Transportation is sponsoring a study on the construction and design of augercast piles for foundations for minor structures, such as sound walls, and there is potential interest in the use of augercast piles as bearing elements in off-system bridges. The University of Houston is the research contractor for that study.

Augercast piles are piles that are constructed by augering a hole in the soil with a continuous flight auger, pumping grout down the stem of the auger as it is retracted, and placing reinforcing steel into the grout after the auger has been withdrawn. Historically, augercast piles have been relatively widely used in the private sector, but have not been used extensively on DOT projects. We are interested in finding out as much as we can about the current state of practice among state DOT's.

As part of our research, we are contacting you and other individuals at several state DOT's to determine whether you currently permit the use of augercast piles for purposes similar to those described above, and, if so, whether you have a construction specification and a set of published design rules that you can share with us. If you can help us, we request that you

1. let us know whether you currently permit the use of augercast piles in
   - sound wall foundations (and foundations for similar structures),
   - retaining wall foundations, or
   - bridge foundations,

and send us a copy of your

2. construction specifications for augercast piles, and
3. design guidelines for augercast piles.

If you can send us this information, we will be very happy to send you a copy of our report when it is completed this fall and to acknowledge your assistance.

Our mailing address is:

Department of Civil and Environmental Engineering
University of Houston
Houston, Texas 77204-4791

Attention: Michael W. O'Neill

Thank you in advance for any assistance that you can give us.

Sincerely yours,

Khaled H. Hassan, Ph. D.
Research Associate

Michael W. O'Neill, Ph. D., P. E.
John and Rebecca Moores Professor
and Project Director

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Table B.1. Summary of Survey of Highway Practice in the United States Regarding the Use of Augercast (Continuous Flight Auger) Piles; April, 1997

Agency	Unre-stricted Use	Re-stricted Use	Do Not Use / Prohibit	Comments
Florida DOT		X		• Use restricted to sound walls. • Will not extend to more critical structures until better integrity evaluation methods are developed. • Draft specification attached
Nebraska DOR		X		• Used sparingly • May be used on all foundations, including bridges • Use limited to cases where nearby structures may be damaged by pile driving vibrations • Specification attached
Kansas DOT		X		• Not used routinely • Use permitted in special cases for bridges, sound walls and retaining walls • Specification attached
Texas DOT		X		• Use permitted on sound walls in Houston • Draft specification attached
Missouri DOT			X	• Do not use
Kentucky DOT			X	• Use not permitted
Nevada DOT			X	• Do not use
New York State DOT			X	• Do not use • Not a popular system in NY, even in private sector
Arkansas DOT			X	• Do not use
Illinois DOT			X	• Do not use
Georgia DOT			X	• Do not use
South Carolina DOT			X	• Use not permitted
Minnesota DOT			X	• Do not use
Tennessee DOT			X	• Do not use / have no objection to using.
New Hampshire DOT			X	• Do not use
Alabama DOT			X	• Do not use / considering using on building
Indiana DOT			X	• Do not use
Colorado DOT			X	• Do not use / have not been proposed
Mississippi DOT			X	• Use not permitted
Utah DOT			X	• Do not use / have no objection to using
FHWA FLO			X	• Do not use

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Florida DOT Draft Specification (April, 1997)

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(This version addresses Rinker's comments, following P. Passe's review with Gainesville.)

STRUCTURES FOUNDATIONS - AUGER CAST PILES. (REV ~~7-8-96~~~~12-3-96~~1-10-97)

Page 452. The following new Section is inserted after Section C455:

SECTION D455
STRUCTURES FOUNDATIONS - AUGER CAST PILES

Index:

D455-1 Description.

The work specified in this Section consists of work necessary to furnish and install auger cast piles used for structural support, other than bridge foundations. Auger cast piles shall be constructed in accordance with this specification and the details and dimensions shown in the plans.

D455-2 General.

D455-2.1 Contractor's ~~Qualifications~~Operations: The Contractors shall submit an Auger Cast Pile Installation Plan in accordance with D455-10. Prior to the start of production piles, the Contractor shall demonstrate to the satisfaction of the Engineer, the dependability of the equipment, techniques, and source of materials to be used.

D455-2.2 Protection of Existing Structures: When the plans require auger cast pile excavations within close proximity to existing structures, the Contractor shall take all reasonable precautions to prevent damage to such structures. The requirements described herein apply to all types of structures (on or off the right of way) that may be adversely affected by foundation construction operations (including phase construction) due to ground loss, dewatering, or vibrations. The Contractor shall be solely responsible for evaluating the need for, design of, and providing all reasonable precautionary features to prevent damage. These measures shall include, but are not limited to:

1. selecting construction methods and procedures that will prevent damaging caving of the shaft excavation

2. monitoring and controlling the vibrations from construction activities (including the driving of casings or sheeting, or from blasting), and

3. protecting utilities, as described in 7-11.6.

Structures within a distance of 10 ~~shaft~~ pile diameters or the estimated ~~shaft~~ pile depth, whichever is greater, shall be monitored for settlement in an approved manner; record elevations to 0.3 mm. The number and location of monitoring points shall be as approved by the Engineer. Elevations shall be taken before construction begins, during the driving of any required casings, during excavation or blasting, or as directed by the Engineer.

When surveys are called for in the plans or specifications, the Contractor shall engage the services of a qualified Professional Engineer registered in the State of Florida. The surveys shall include all structures (except as noted herein), or portions therein, within a distance of 10 ~~shaft~~ pile diameters or the estimated depth of excavation, whichever is greater, or the distance shown in the plans. Surveys shall be completed before ~~shaft~~ auger cast pile operations begin and after ~~shaft~~ auger cast pile operations are completed. The condition of the structures shall be adequately documented with descriptions and pictures. All existing cracks shall be thoroughly documented. Two reports shall be prepared documenting the condition of the structures; one report before ~~shaft~~ auger cast pile construction operations begin and a second report after auger cast pile operations are complete. Both reports shall become the property of the Department. Preconstruction and post-construction surveys of the condition of ~~bridges~~ structures owned by the Department will not be required except when shown in the plans or specifications.

When the plans require excavations for construction of footings or caps supported by auger cast piles, the Contractor shall be responsible for evaluating the need for, design of, and providing any necessary features to protect adjacent structures. Sheeting and shoring shall be constructed according to plans provided by the Contractor except when the sheeting and shoring are detailed in the plans. Sheeting and shoring installed to protect existing structures shall be designed by a Professional Engineer, employed by the Contractor, registered in the State of Florida and who shall sign and seal the plans and specification requirements. Plans and specifications for sheeting and shoring provided by the Contractor shall be sent to the Engineer for his record before construction begins.

Existing structures within a distance of three times the depth of excavation for the footing shall be monitored for movement. The number and location of monitoring points shall be as approved by the Engineer. Elevations shall be taken before the driving of any sheeting, daily during the driving of sheeting and during excavation, measured and recorded to 0.3 mm. The Contractor shall notify the Engineer of any movements detected and immediately take any remedial measures required to prevent damaging the existing structure.

When dewatering is shown in the plans or specifications, the Contractor shall install a piezometer near the right of way line and near any structure that may be affected by ground water. The piezometer shall be monitored and the ground water elevation level recorded daily. The Contractor shall notify the Engineer of any ground water lowering of 300 mm or more near the structure.

When vibration monitoring is called for in the plans or specifications, the Contractor shall engage the services of a qualified Professional Engineer registered in the State of Florida. Monitor and record vibration level during the driving of casings, sheeting, or during blasting operations conducted by the Contractor. Vibration monitoring equipment shall be capable of detecting velocities of 2.5 mm/second or less.

At any time the Contractor detects settlement of 1.5 mm, vibration levels reaching 12 mm/sec, or damage to the structure, he shall immediately stop the source of vibrations, backfill any open auger cast pile excavations, and contact the Engineer for instructions.

D455-3 Materials.

The materials used shall conform with the requirements specified in Division III and herein. Specific references are as follows:

(1) Portland Cement (Types I, II, ~~or~~ III, IP, and IS)	Section 921
(2)Fly Ash, Slag and other Pozzolanic Materials for Portland Cement Concrete	Section 929
(3) Fine Aggregate (Sand)*	Section 902
(4) Admixtures	Section 924
(5) Water	Section 923
(6) Fluidizer	ASTM C 937

* Any clean sand with 100% passing 9.5 mm sieve and not more than 10% passing the 75 μm sieve may be used.

D455-4 Grout Mix Proportions.

The grout mix shall consist of a mixture of Portland cement, flyash, retarder, fluidizer, sand and water so proportioned and mixed as to produce a mortar capable of maintaining the solids in suspension without appreciable water gain and which may be pumped without difficulty and fill open voids in the adjacent soils. These materials shall be so proportioned as to produce a hardened grout of the required strength shown on the plans.

D455-5 Mixing and Pumping Cement Grout.

1. Only pumping equipment approved by the Engineer shall be used in the preparation and handling of the grout. All oil or other rust inhibitors shall be removed from the mixing drums, stirring mechanisms, and other portions of the equipment in contact with the grout before the mixers are used.

2. All materials shall be accurately measured by volume or weight as they are fed to the

mixer. The order of placing materials in the mixer shall be as follows: 1) water, 2) fluidifier, 3) other solids in order of increasing particle sizes.

3. The quantity of water used and the time of mixing shall be such as to produce homogenous grout having a consistency of 18 to 24 seconds, or higher if specified by the Engineer, when tested with a flow cone in accordance with ~~Corps of Engineers Specification CRD-C-79~~ASTM C 939 (19 mm diameter outlet), with a frequency at the discretion of the Engineer. Time of mixing shall not be less than 1 minute. If agitated continuously, the grout may be held in the mixer or agitator for a period not exceeding 2.5 hours at grout temperatures below 20°C; ~~somewhat less at higher temperatures not exceeding 38°C~~2 hours for temperatures from 20 to 38°C. Grout shall not be placed when its temperature exceeds ~~100°F~~ 38°C. If there is a lapse in the operation of grout injection, the grout shall be recirculated through the pump, or through the mixer drum or agitator.

4. A screen no larger than 19.0 mm mesh shall be used between the mixer and pump to remove large particles which might clog the injection system.

5. The grout pump shall be a positive displacement piston type pump capable of developing displacing pressures at the pump up to 50 kPa. The minimum volume of grout placed in the hole shall at least equal the column of the auger hole.

6. The grout pump/system shall be equipped with a pressure gauge to accurately monitor grout flow. The equipment shall be tested and calibrated at the beginning of the work to demonstrate flow rate measurement accuracy to ±3% over the range of grouting pressures anticipated during this work. The equipment shall also be calibrated anytime the Engineer suspects that the grout pump performance has changed.

D455-6 Testing Cement Grout.

The Contractor shall make four 152.4 by 304.8 mm cylinders for each 38 m^3 of grout placed, per day of pile placement. Two cylinders will be tested at 7 days and two cylinders will be tested at 28 days. The minimum strength to be obtained will be as specified on the plans. If the strength fails to meet the minimum strength specified, the grout will be accepted or rejected according to the requirements of 346-10.

D455-7 Pile Installation.

1. The Contractor shall locate the piles as shown on the drawings.

2. Should soft, compressible muck, organics, clay or other unsuitable materials (non A-1, A-3, A-2-4 or limestone materials) be encountered, the unsuitable material shall be removed to a maximum depth of ~~5 feet~~1.5 m and a maximum diameter about the pile centerline, not to exceed 1/2 of the distance to the adjacent pile. The volume shall be backfilled with clean granular backfill materials (A-1, A-3, A-2-4) placed and compacted in maximum 300 mm lifts to at least 95% of maximum dry density as determined by AASHTO T 180. This work shall be completed to the Engineer's satisfaction prior to auger cast pile construction. Should more than 1.5 m or excessive quantities of unsuitable material be encountered, the Contractor shall immediately advise the Engineer and proceed with the work as directed by the Engineer.

3. The auger flighting shall be continuous from the auger head to the top of auger with no gaps or other breaks. The auger flighting shall be uniform in diameter throughout its length and shall be the diameter specified for the piles less a maximum of 3%. The distance between flights shall be approximately half the diameter of the auger.

4. The hole through which the grout is pumped during placement shall be located at the bottom of the auger head below the bar containing the cutting teeth.

5. The pile auger leads shall contain a bottom guide.

6. The length and diameter of piles shall be as shown on the drawings.

7. Piles shall be placed by rotating a continuous flight hollow shaft auger into the ground at a continuous rate that prevents removal of excess soil. Stop advancement after reaching the predetermined depth.

8. Should auger penetration to the required depth prove difficult due to hard materials/refusal, the pile location may be predrilled, upon approval of the Engineer, through the obstruction using appropriate drilling equipment, to a diameter no larger than 1/2 the prescribed finish diameter of the auger cast pile. Auger cast pile construction shall commence immediately upon predrilling to minimize ground loss and soil relaxation. Should non-drillable material be encountered which prevents placing of a pile to the depth required, the Contractor shall immediately advise the Engineer and proceed with the work as directed by the Engineer. Refusal is defined as the depth where the penetration of the standard auger equipment is less than 300 mm/minute.

9. The hole in the bottom of the auger shall be closed while being advanced into the ground with a suitable plug. The plug shall be removed by the grout or with the reinforcing bar.

10. Grout shall be pumped with sufficient pressure as the auger is withdrawn to fill the auger hole preventing hole collapse and to cause the lateral penetration of the grout into soft or porous zones of the surrounding soil. A head of at least ~~5 feet~~1.5 m of grout above the injection point shall be carried around the perimeter of the auger so that the grout has a displacement action removing any loose material from the hole. Positive rotation of the auger shall be maintained at least until placement of the grout.

11. Once the grout head has been established, the speed of rotation of the auger should be reduced as much as possible, or stopped, and extraction commenced at a rate consistent with the pump discharge. Extraction must be at a steady rate, while pulling too slowly can result in a locked-in auger, withdrawing too rapidly can lead to necking of the pile or substantially reduced pile section. Grout should start flowing out from the hole when the cutting head is within 1.5 m of the ground surface. The total volume of grout shall be at least 115% of the theoretical volume for each pile. If the cutting head reaches the ground surface without any grout, the extraction was too fast and the integrity of the pile is in doubt. The pile must be redrilled under the direction of the Engineer. If pumping of grout is interrupted for any reason, the Contractor shall reinsert the auger by drilling at least 1.5 m below the tip of the auger when the interruption occurred, and then regrout.

This method of placement shall be used at all times and not be dependent on whether the hole is sufficiently stable to retain its shape without support from the earth filled auger. The required steel reinforcement shall be placed while the grout is still fluid but no later than 1/2 hour after pulling of the auger.

12. If less than 115% of the theoretical volume of grout is placed in any 1.5 m increment (until the grout head on the auger flighting reaches the ground surface), the pile shall be reinstalled by advancing the auger 3 m or to the bottom of the pile if that is less, followed by controlled removal and grout injection.

13. Accurate records shall be maintained showing the depth to which each pile is placed and the amount of material used in each pile. Any unusual conditions encountered during the installation shall be noted.

14. The Contractor shall be responsible for furnishing the reinforcing steel and anchoring bolts for a proper installation, as shown in the contract drawings.

15. The reinforcement at time of placement must be free of mud, oil or other coatings that adversely affect bond. Reinforcement shall be without kinks or nonspecified bends. Make splices in reinforcement as shown on contract drawings, unless otherwise accepted.

16. Supports shall remain in place until the grout reaches a minimum of 50% design strength or 3 days cure time, whichever is first.

D455-8 Construction Tolerances.

Piles shall be located as shown on the drawings, or as otherwise directed by the Engineer. Pile centers shall be located to an accuracy of ±75 mm. The top of pile elevation shall be within an accuracy of ±75 mm from the plan elevation.

D455-9 Unacceptable Piles.

Unacceptable piles will be defined as piles that fail for any reason included but not limited to the following: Piles placed out of position; are below elevations; are damaged; do not have steel reinforcement inserted as required; have inadequate grout strength; have inadequate consolidation for any reason. (Inadequate consolidation is when the pile is honeycombed.)

To conform to specified requirements, unacceptable piles shall be replaced or repaired at the Contractor's expense, as directed by the Engineer.

D455-10 Auger Cast Pile Installation Plan.

At the preconstruction conference, but no later than 30 days before auger cast pile construction begins, the Contractor shall submit an auger cast pile installation plan for approval by the Engineer. This plan shall provide detailed information including the following:

1. Name and experience record of auger cast pile superintendent or foreman in responsible charge of auger cast pile operations. The person in responsible charge of day to day auger cast pile operations shall have satisfactory prior experience constructing shafts similar to those described in the plans and specifications. Final approval by the Engineer will be subject to satisfactory performance in the field.

2. List and size of the proposed equipment, including cranes, augers, concrete pumps, mixing equipment etc., including details of proposed pump calibration procedures.

3. Details of ~~shaft~~ pile installation methods.
4. Details of reinforcement placement, including support and method of centering ~~in~~ the shaftpile.
5. Required submittals, including shop drawings and concrete grout design mixes.
6. Other information shown in the plans or requested by the Engineer.

D455-11 Inspection and Records.

Pile installation shall be monitored by the Engineer. The Engineer and Contractor shall maintain separate records of each pile installed showing:

1. Pile location
2. Ground elevation
3. Pile length
4. Tip elevation
5. Pile top elevation
6. Pay length (when piles are paid for separately)
7. Overburden length (length cast above the final grade point)
8. Pile diameter
9. Quantity of grout placed
10. Theoretical quantity of grout required
11. Drilling time
12. Grouting time
13. All other pertinent data relative to the pile installation.
14. Grout truck time of arrival to the site and batchtime
15. Flow cone (consistency) results

D455-12 Basis of Payment.

D455-12.1 Protection of Existing Structures: The quantity to be paid for under this item, when included in the Contract Documents, shall be one lump sum. Such price and payment shall include all cost of work shown in the plans or described herein for protection of existing structures. When the Contract Documents do not include an item for protection of existing structures, the cost of settlement monitoring as required by this specification shall be included in the cost of the structure; however, work in addition to settlement monitoring will be paid for as Unforeseeable Work when such additional work is ordered by the Engineer.

D455-12.2 Auger Cast Pile: ~~Auger cast piles shall be paid for as part of the structure.~~ Payment for auger cast piles shall be made at the contract price per meter between tip and required pile top elevations for all completed piles. This price shall include all labor, materials, and incidentals for construction of piles of sizes and depths indicated on the contract drawings or otherwise required under this contract, but shall not include any work outside the contract working area. This price shall also include the removal from the job site the spoil of the augering operation as directed by the Engineer and this price shall also include the cost of all excess grout displaced out of the top of the augered hole. No payment shall be made for piles or holes placed in an incorrect location or for unsatisfactory construction. All abandoned piles and holes shall be backfilled with suitable material as directed by the Engineer.

D455-12.3 Items of Payment: The prices and payments specified in D455-12.1 through D455-12.2 above, shall be full compensation for all the work specified herein.

Payment shall be made under:

Item No. 2455- 18- Protection of Existing Structures - lump sum.
Item No. 2455-112- Auger Grouted Piles - per meter.

Nebraska DOR Draft Specification (April, 1997)

CAST-IN-PLACE AUGERED CONCRETE PILING

SCOPE OF WORK

THE WORK COVERED BY THESE SPECIFICATIONS CONSISTS OF FURNISHING ALL LABOR, EQUIPMENT AND MATERIALS FOR THE PLACING OF CAST-IN-PLACE AUGERED CONCRETE PILES AS SHOWN ON THE PLANS AND DESCRIBED HEREIN.

GENERAL

CAST-IN-PLACE PILES SHALL BE PLACED BY ROTATING A CONTINUOUS FLIGHT, HOLLOW SHAFT AUGER INTO THE GROUND TO THE DEPTH SHOWN IN THE PLANS. AS THE AUGER IS WITHDRAWN, HIGH-STRENGTH MORTAR SHALL BE PUMPED THROUGH THE HOLLOW SHAFT UNDER SUFFICIENT PRESSURE SO AS TO FILL THE HOLE, PREVENT HOLE COLLAPSE, AND CAUSE LATERAL PENETRATION OF THE MORTAR INTO THE SURROUNDING SOIL. A HEAD OF SEVERAL FEET OF MORTAR ABOVE THE INJECTION POINT SHALL BE CARRIED AROUND THE PERIMETER OF THE AUGER FLIGHTING AT ALL TIMES DURING THE RAISING OF THE AUGER, SO THAT THE HIGH-STRENGTH MORTAR HAS A DISPLACING ACTION WHICH REMOVES ANY LOOSE MATERIAL FROM THE HOLE. THE HIGH-STRENGTH MORTAR SHALL BE BROUGHT UP TO THE BOTTOM OF THE FOOTING ELEVATION AND ALL LOOSE MATERIAL CARRIED TO THIS ELEVATION BY THE HIGH-STRENGTH MORTAR SHALL BE REMOVED. THIS METHOD OF PLACEMENT SHALL BE USED AT ALL TIMES AND NOT BE DEPENDENT ON WHETHER OR NOT THE HOLE IS SUFFICIENTLY STABLE TO RETAIN ITS SHAPE WITHOUT SUPPORT FROM THE EARTH-FILLED AUGER. THE CAST-IN-PLACE AUGERED CONCRETE PILE SHALL BE EXTENDED FROM THE BOTTOM OF THE FOOTING ELEVATION TO THE PILE CUT-OFF ELEVATION AS SHOWN ON THE PLANS.

HIGH-STRENGTH MORTAR

THE MORTAR USED TO FILL THE HOLES SHALL CONSIST OF A MIXTURE OF PORTLAND CEMENT, FLUIDIFIER, SAND AND WATER SO PROPORTIONED AND MIXED AS TO PROVIDE A MORTAR CAPABLE OF MAINTAINING THE SOLIDS IN SUSPENSION WITHOUT APPRECIABLE WATER GAIN, YET WHICH MAY BE PLACED WITHOUT DIFFICULTY, AND WHICH WILL LATERALLY PENETRATE AND FILL ANY VOIDS IN THE FOUNDATION MATERIAL. MINERAL FILLER MAY BE ADDED TO THE ABOVE MIX IN LIEU OF A SMALL PERCENTAGE OF THE PORTLAND CEMENT AT THE PILING CONTRACTOR'S OPTION. THE MATERIALS SHALL BE SO PROPORTIONED AS TO PROVIDE A HARDENED MORTAR HAVING AN ULTIMATE COMPRESSIVE STRENGTH OF 3,500 PSI AT 28 DAYS.

THE MORTAR MIX SHALL BE TESTED BY MAKING ONE SET OF 2-INCH CUBES FOR EACH DAY WHICH PILES ARE PLACED. A SET OF CUBES SHALL CONSIST OF THREE CUBES TO BE TESTED AT SEVEN DAYS, AND THREE CUBES TO BE TESTED AT 28 DAYS, EXCEPT THAT THREE EXTRA SETS OF CUBES SHALL BE MADE ON THE ANCHOR AND STATIC TEST PILE. THE EXTRA CUBES SHALL BE TESTED AT 2-DAY INTERVALS, STARTING THE SECOND DAY, TO DETERMINE WHEN THE STATIC TEST CAN BE STARTED.THE STATIC TEST MAY BEGIN WHEN THE CUBE STRENGTH HAS REACHED A STRENGTH OF 2500 PSI. THE CONTRACTOR SHALL SUBMIT FOR APPROVAL, THE DESIGN MIX AND CERTIFIED TEST RESULTS SHOWING 7 AND 28 DAY STRENGTHS. TEST CUBES SHALL BE MADE AND TESTED IN ACCORDANCE WITH ASTM C109, WITH THE EXCEPTION THAT THE MORTAR SHOULD BE RESTRAINED FROM EXPANSION BY A TOP PLATE.

MATERIALS

A. PORTLAND CEMENT: PORTLAND CEMENT SHALL CONFORM TO FEDERAL SPECIFICATIONS SSC192 OR CURRENT ASTM STANDARDS, DESIGNATION C150.

B. MINERAL FILLER: MINERAL FILLER SHALL BE FINELY POWDERED SILICEOUS MATERIAL WHICH POSSESSES THE PROPERTY OF COMBINING WITH THE

C. FLUIDIFIER: FLUIDIFIER SHALL A BE COMPOUND POSSESSING CHARACTERISTICS WHICH WILL INCREASE THE FLUIDITY OF THE MIXTURE, REDU BLEEDING, ASSIST IN THE DISPERSAL OF CEMENT GRAINS, AND NEUTRALIZE TI SETTING SHRINKAGE OF THE HIGH-STRENGTH CEMENT MORTAR. THE CONTRACTOI SHALL SUBMIT TEST RESULTS OF THE FLUIDIFIER INTENDED FOR USE PRIOR TC APPROVAL.

D. WATER: WATER SHALL BE FRESH, CLEAN, AND FREE FROM INJURIOUS AMOUNTS OF SEWAGE, OIL, ACID, ALKALI, SALTS, OR ORGANIC MATTER.

E. FINE AGGREGATE: SAND SHALL MEET THE REQUIREMENTS OF CURRENT ASTM STANDARDS, DESIGNATION C33.

THE SAND SHALL CONSIST OF HARD, DENSE, DURABLE, UNCOATED ROCK PARTICLES AND BE FREE FROM INJURIOUS AMOUNTS OF SILT, LOAM, LUMPS, SC OR FLAKY PARTICLES, SHALE, ALKALI, ORGANIC MATTER, MICA, AND OTHER DELETERIOUS SUBSTANCES. IF WASHED, THE WASHING METHOD SHALL BE SUCH WILL NOT REMOVE DESIRABLE FINES, AND THE SAND SHALL SUBSEQUENTLY BE PERMITTED TO DRAIN UNTIL THE RESIDUAL-FREE MOISTURE IS REASONABLY UNIFORM AND STABLE. THE SAND SHALL BE WELL-GRADED FROM FINE TO COARS WITH FINENESS MODULUS BETWEEN 1.40 AND 3.40. THE FINENESS MODULUS IS DEFINED AS THE TOTAL DIVIDED BY 100 OF THE CUMULATIVE PERCENTAGES RETAINED ON U.S. STANDARD SIEVE NOS. 16, 30, 50 AND 100.

F. THE CONTRACTOR SHALL CERTIFY, IN WRITING, THAT ALL MATERIALS MEET THESE MATERIAL REQUIREMENTS. THE ENGINEER MAY TEST ANY OR ALL MATERIALS HE DEEMS NECESSARY.

MIXING AND PUMPING OF HIGH-STRENGTH CEMENT MORTAR

ONLY APPROVED PUMPING, CONTINUOUS MIXING AND AGITATING EQUIPMENT SHALL BE USED IN THE PREPARATION AND HANDLING OF THE MORTAR. ALL OIL OR OTHER RUST INHIBITORS SHALL BE REMOVED FROM MIXING DRUMS AND MORTAR PUMPS. IF READY-MIX MORTAR IS USED, AN AGITATOR OF SUFFICIENT SIZE SHALL BE USED BETWEEN THE READY-MIX TRUCK AND THE MORTAR PUMP TO INSURE HOMOGENEOUS MIX AND CONTINUITY IN THE PUMPING OPERATIONS. ALL MATERIALS SHALL BE SUCH AS TO PRODUCE A HOMOGENEOUS MORTAR OF THE DESIRED CONSISTENCY. IF THERE IS A LAPSE IN THE OPERATION, THE MORTAR SHALL BE RECIRCULATED THROUGH THE PUMP.

THE MORTAR PUMP SHALL BE A POSITIVE DISPLACEMENT PISTON-TYPE PUMP CAPABLE OF DEVELOPING DISPLACING PRESSURES AT THE PUMP UP TO 350 PSI. THE MINIMUM VOLUME OF MORTAR PLACED IN THE HOLE SHALL BE AT LEAST EQUAL TO THE VOLUME OF THE AUGERED HOLE.

LOCATION OF PILES

PILES SHALL BE LOCATED AS SHOWN ON DRAWINGS OR AS OTHERWISE DIRECTED BY THE ENGINEER. PILE CENTERS SHALL BE LOCATED TO AN ACCURACY OF PLUS OR MINUS THREE INCHES.

ADJACENT PILES SHALL NOT BE PLACED UNTIL THE MORTAR IN THE PILES HAS REACHED ITS INITIAL SET IN ORDER THAT THERE WILL BE NO INTERCONNECTION BETWEEN ADJACENT PILES WHILE THE MORTAR IS IN A FLUID STATE.

OBSTRUCTIONS

SHOULD ANY OBSTRUCTION (INCLUDING BUT NOT LIMITED TO BOULDERS AND TIMBERS) BE ENCOUNTERED, WHICH SHALL PREVENT PLACING THE PILE TO THE DEPTH REQUIRED, OR SHALL CAUSE THE PILE TO DRIFT FROM THE REQUIRED LOCATION, THE PILE SHALL BE COMPLETED IN ACCORDANCE WITH THE PARAGRAPH TITLED, "GENERAL". IN THIS EVENT AN ADDITIONAL ADJACENT PILE SHALL BE PLACED TO THE REQUIRED DEPTH AT A LOCATION AS DIRECTED BY THE ENGINEER. ANY PILE NOT PLACED TO THE REQUIRED DEPTH, DUE TO AN OBSTRUCTION, SHALL BE PAID FOR, PER LINEAR FOOT, AT THE CONTRACT UNIT PRICE, FOR THE ITEM "16" CAST-IN-PLACE AUGERED CONCRETE PILING".

PILE EXTENSIONS

CAST-IN-PLACE AUGERED CONCRETE PILES SHALL BE EXTENDED FROM THE BOTTOM OF FOOTING SHOWN IN THE PLANS TO THE PILE CUT-OFF ELEVATION SHOWN IN THE PLANS USING A SUITABLE REMOVABLE FORM WHICH IS PLACED AROUND THE CAST-IN-PLACE PILE AT THE FOOTING ELEVATION.

AUGERING EQUIPMENT

THE HOLE THROUGH WHICH THE HIGH-STRENGTH MORTAR IS PUMPED DURING THE PLACEMENT OF THE PILE SHALL BE LOCATED AT THE BOTTOM OF THE AUGER HEAD BELOW THE BAR CONTAINING THE CUTTING TEETH.

THE AUGER FLIGHTING SHALL BE CONTINUOUS FROM THE AUGER HEAD TO THE TOP OF AUGER WITH NO GAPS OR OTHER BREAKS. THE PITCH OF THE AUGER FLIGHTING SHALL NOT EXCEED NINE INCHES.

AUGERS OVER 40 FEET IN LENGTH SHALL CONTAIN A MIDDLE GUIDE.

THE LEADS SHOULD BE PREVENTED FROM ROTATING BY A STABILIZING ARM.

METHOD OF MEASUREMENT AND BASIS OF PAYMENT

THE ITEM, "16" CAST-IN-PLACE AUGERED CONCRETE PILING", AS SPECIFIED AND SHOWN IN THE PLANS, SHALL BE MEASURED BY THE LINEAR FOOT, INSTALLED AND ACCEPTED BY THE ENGINEER.

THE ITEM, "16" CAST-IN-PLACE AUGERED CONCRETE PILING", SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT. THIS PRICE SHALL BE FULL COMPENSATION FOR FURNISHING AND PLACING ALL MATERIALS, INCLUDING FORMED PILE EXTENSIONS, PLACING AND FURNISHING THE REBAR CAGES, AND FOR ANY OTHER INCINDENTALS REQUIRED AND NECESSARY TO COMPLETE THE WORK.

ANCHOR PILES

ANCHOR PILES SHALL BE INSTALLED AT LOCATIONS SHOWN ON THE PLANS FOR PRODUCTION PILES. THE ANCHOR PILES SHALL BE THE SAME LENGTH AS THE PILE USED FOR THE STATIC PILE LOAD TEST. AFTER THE TEST, THE ANCHOR PILES SHALL BE INCORPORATED INTO THE STRUCTURE AS LOAD BEARING PILES. ANCHOR PILES SHALL BE PAID FOR UNDER THE ITEM "16" CAST-IN-PLACE AUGERED CONCRETE PILING".

16" CAST-IN-PLACE AUGERED CONCRETE PILING STATIC PILE LOAD TEST

ALL PERSONNEL AND EQUIPMENT TO CONDUCT THE TEST SHALL BE FURNISHED BY THE CONTRACTOR.

A STATIC PILE LOAD TEST, MADE AT THE LOCATION INDICATED IN THE PLANS AND IN ACCORDANCE WITH ASTM D1143-81 "STANDARD TEST METHOD FOR PILES UNDER STATIC AXIAL COMPRESSIVE LOAD", SHALL BE REQUIRED ON THIS PROJECT PRIOR TO PROVIDING THE FINAL ORDER LENGTHS FOR THE CAST-IN-PLACE AUGERED CONCRETE PILING.

THE PILE SHALL BE LOADED TO THREE TIMES THE DESIGN PILE BEARING AS SHOWN ON THE PLANS.

SHOP PLANS DETAILING TESTING PROCEDURES SHALL BE SUBMITTED TO THE BRIDGE DIVISION PRIOR TO TESTING. THE PLANS SHALL SHOW (BUT NOT BE LIMITED TO) ANCHOR PILE LOCATIONS, DYWIDAG BAR PLACEMENT, HYDRAULIC JACK CAPACITY, REACTION FRAME DESIGN AND GAGE READING INTERVALS.

A FINAL REPORT SHALL BE SUBMITED TO THE DEPARTMENT AND SHALL INCLUDE ALL APPLICABLE INFORMATION AS SPECIFIED IN ASTM D1143-81 SECTION 8.

THE ITEM, "16" CAST-IN-PLACE AUGERED CONCRETE PILING STATIC PILE LOAD TEST", SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE FOR EACH. THE PRICE SHALL BE FULL COMPENSATION FOR PROVIDING PERSONNEL AND EQUIPMENT TO PERFORM THE LOAD TEST, SHOP PLANS DETAILING TESTING PROCEDURES, FINAL REPORT, 16" CAST-IN-PLACE AUGERED CONCRETE PILING, INCLUDING FURNISHING AND PLACING ALL MATERIALS, FORMED PILE EXTENSIONS, FURNISHING AND PLACIN(THE REBAR CAGES, FURNISHING AND PLACING THE DYWIDAG THREADBARS IN THE ANCHOR PILE, CUTTING OFF THE DYWIDAG THREADBARS AFTER TESTING IS COMPLETI AND FOR ANY OTHER INCIDENTALS REQUIRED AND NECESSARY TO COMPLETE THE WORI

SUBSECTION 703.18, PARAGRAPH 3 IS VOID.

Kansas DOT Draft Specification (April, 1997)

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KANSAS DEPARTMENT OF TRANSPORTATION
SPECIAL PROVISION
TO THE
STANDARD SPECIFICATIONS
EDITION OF 1990

NOTE: This special provision is generally written in the imperative mood. The subject, "the *Contractor*" is implied. Also implied in this language are "*shall*", "*shall be*", or similar words and phrases.

The word "*will*" generally pertains to decisions or actions of the Kansas Department of Transportation.

DRILLED AND CAST-IN-PLACE PRESSURE GROUTED PILING

1.0 DESCRIPTION.

Furnish materials, equipment and tools for, and complete the installation of all piling. Perform the pile load test on piles shown on the Plans.

BID ITEM	UNIT
Pressure Grouted Piles (*)	Linear Foot
Test Pile (*)	Linear Foot
Load Test	Each
* Denotes Diameter	

2.0 MATERIALS.

Furnish the following materials that conform to the requirements of the Materials Division of the Standard Specifications:

Fine Aggregate . . . . Section 1100*
Reinforcing Steel. Section 1600
Portland Cement (Type 1P). . . . . Section 2000
Flyash (Class C) . . . . Section 2000
Water. . . . . Section 2400

*Subsection 1102(c) with 0% retained on the No. 4 sieve and a fineness modulus between 1.40 and 3.40.

of ASTM ASTM C937.

3.0 CONSTRUCTION REQUIREMENTS.

(a) Mixing and Pumping Mortar.

(1) Use approved mixing and pumping equipment for the preparation and handling of mortar. Measure all materials byvolume or by weight as they are fed into the mixer. Remove all oil or other rust inhibitors from the mixing drum and mortar pumps. Time the mixing to produce a homogeneous mortar. Recycle the mortar through the pump or through the mixer drum or agitator if there is a lapse in the pumping operation. The minimum and maximum mixing times as well as the maximum recycle time are dependent on the Contractor's mix. Determine this during installation of the piles for the load test.

(2) The cement base non-shrinkage mortar defined by ASTM C1107 consists of Portland cement (Type 1P), a special pozzolan, a grouting agent, sand and water. Proportion and mix the mortar to produce a mix capable of maintaining the solids in suspension without appreciable water gain. Proportion the mix so it may be pumped without difficulty and will penetrate and fill any open voids in the adjacent soils.

(3) Submit a mix-design of the cement base mortar to theEngineer for approval before use in this work. Include the following in the mix-design:

1. The test results on the Fine Aggregate showing their compliance with the specifications.
2. The source of the Fine Aggregate.
3. Weights of all materials used for one cubic yard of fresh mixed mortar.
4. Brand name of the Portland Cement (Type 1P), brand name of grouting agent (water reducer and retarder), and source and type of flyash (pozzolan).
5. Compressive strengths of test specimens made and cured in accordance with ASTM C 192 and tested in accordance with ASTM C 39. Proportion the materials to produce a hardened mortar with a compressive strength of 4,000psi minimum at 28 days.
6. Submit a sufficient quantity of the materials proposed for use far enough in advance of use so that the Engineer may conduct applicable tests.
7. The mortar flow as determined by ASTM C-939 with a flow cone modified to a 3/4" opening shall be between 17 and 25 seconds. Test each load for process control and record the flow. Provide the specified flow cone for use by the project inspector.

(4) Maintain the temperature of the grout at the time of placement between 50o F. and 90o F. Grout outside the

(5) Locate pressure gauges on the grout pump and at the auger rig so the grouting pressure can be checked by the operator and the Field Engineer. Maintain the pressure gauges in good condition. Use a mechanical counter on all pumps to monitor the quantity of grout placed. Before placement, the Field Engineer will verify the volume of grout displacement per piston stoke.

(6) Failure of the mixed mortar to meet compressivestrength requirements of paragraph 3.0(3)(5) will be considered grounds for rejection of the pile. Replace the pile at the location determined by the Engineer. Leave the rejected pile in place with no payment made for that pile. Work and materials required for a pile that replaces a rejected pile will not be paid for. Submit the proposed method of constructing the replacement pile to the Engineer for approval before work on it begins.

(b) Strength.

The Kansas Department of Transportation will make and test standard compression cylinders during the progress of the project. Make a minimum of one set of three cylinders for each day's work. From each set of three, test one at seven days, one at 28 days and one as determined by the Engineer.

(c) Records.

(1) Before Commencing Work: Submit to the Engineer and obtain approval for the following:

1. Sketch and description of the pile drilling equipment to be utilized.
2. Complete description of method of installation.
3. Concrete mix design including preliminary mixing and recycling times.
4. The proposed method for calibrating the volume of grout displaced per piston stroke.
5. A dimensioned sketch of the proposed test loading arrangement, and data on testing and measuring equipment, including jack and gauge calibration.

(2) During the course of the work, the Engineer will record the following:

1. Load test reports, if applicable, including all test data, and a graph of load versus settlement.

2. A daily pile report showing the pile number and location, date placed, length of pile, final tip elevation and log of boring. The daily pile report will also show quantity of grout, reinforcing steel, mixing times, delivery times, and unusual occurrences for each pile.

3. Mortar flow tests results.

(d) Load Tests.

Perform load tests with monitoring and evaluation performed by the Kansas Department of Transportation. The cost of anchor piles and all equipment necessary to conduct the pile load test will be paid for as "Load Test". Perform the load test in accordance with ASTM D1143 using the load application method of Section 3.3. Follow Section 5.6 of ASTM D1143 for loading procedures. The time interval the loads will be held and the percent of the design load in each increment will be shown on the Plans, or discussed at the pre-bid or pre-construction conference. The load apparatus will have the capacity to load the piles to three times the design load or failure.

Instrument the test pile with four dial gauges. Anchor the reference beam a minimum of ten feet from the test pile.

Instrument the reaction piles to determine uplift capacity concurrently with the axial load test.

Reinforce the test pile and the reaction piles identically to the production piles. Construct the reaction piles to the same length as the production piles

Perform the load test after the grout has reached its design strength, but not before seven days.

Before beginning the work, submit for approval a dimensioned sketch of the proposed loading arrangement, and data on testing and measuring equipment including jack and gauge calibrations.

(e) Construction Requirements.

Submit to the Engineer for review, an installation plan no later than one month before constructing pressure grouted test piles. As a minimum, provide the following information:

1. Evidence of successful installation of auger-cast piles under similar job and subsurface conditions, including a job superintendent on site with a minimum of five years of method specific experience.

2. List of proposed equipment to be used.

3. Details of mortar pumping and reinforcing steel placement methods.

Construct the test piling to the diameter and length shown on the Plans. Install all finish piles within a tolerance of ± 3 in. from the location shown on the Plans to the center of the pile.

Use a middle guide when augers over 40 feet in length are used.

Prevent the leads from rotating by approved means.

Drill a continuous flight hollow shaft auger into the underlying soil to the required depth. Inject a cement base non-shrinking mortar at a pressure between 120 and 240 psi through the hollow shaft of the auger. Place the mortar in a continuous operation from the bottom to the top of the pile. The minimum volume of mortar pumped into the pile will equal at least 115% of the theoretical volume. As the mortar is injected, slowly withdraw the auger by rotating it in a clockwise direction. Withdraw the auger carefully to prevent the earth of mud from caving into the hole. If the auger is raised by a sudden jerk for any appreciable distance, redrill the hole and restart the grouting operation. During the pumping process, maintain a head of mortar at least ten feet above the point of injection. Check the volume of the mortar pumped in five foot increments. The auger flighting must be continuous from the auger head to the top of the auger with no gaps or other breaks. Use the auger to retain the shape and to remove all loose material from the hole. Since the pile may be placed below the water table, under hydrostatic pressure, exercise extreme care to prevent the lateral pressure of both soil and water from "pinching in" and reducing the pile diameter.

In the event non-augerable material is encountered, remove the obstruction and complete the pile. If the obstruction cannot be removed, place another pile in a location determined by the Engineer. Non-augerable material is defined as material which causes the rate of penetration to be reduced to less than one foot per minute, assuming an applied torque of 10,000 foot pounds. The lineal footage of any piles which encounter non-augerable material above the specified tip elevation, plus the lineal footage of any replacement pile, will be paid for at the Contract unit price bid per lineal foot for "Pressure Grouted Piles."

Coordinate the performance of the load test with the Engineer. Perform the test, with approval of the Engineer, before the general excavation if the test pile is free from the top down to the cutoff point.

The Engineer will make a thorough analysis of the test results and determine the most feasible length required for the conditions encountered.

Use the construction methods developed during the test pile program for the production piles.

Construct the piling to the diameter and length shown on the Plans or as revised after evaluation of the test piles. Install all finish piles within a tolerance of ± 3 in. from the location shown on the Plans to the center of the pile.

Installation of an adjacent pile within five feet of a previously installed pile is to be delayed a minimum of four hours to prevent the possibility of the hydrostatic head causing the mortar to break through the hole being drilled. The Engineer may revise the four hour time limit based on the set time of the mortar used in the test piles.

Completely assemble the reinforcing steel before placement. Place the reinforcing steel cage after removal of the auger and while the mortar is still fluid. Use suitable centralizers to insure that the specified reinforcing steel cover is maintained.

Construct the Pressure Grouted Piles to the elevation shown on the Plans. Float finish and level the top of the piles.

(g) Removal of Waste.

Remove earth and sand accumulated through the piling operation from the site.

4.0 MEASUREMENT AND PAYMENT.

The Engineer will measure test pile and drilled and cast-in-place pressure grouted piles to the nearest foot. Measurement is on the basis of the number of feet of piling drilled and grouted in place below the top of pile elevation. Measurement of Load tests will be per each test pile. Payment for "Pressure Grouted Piles", "Test Piles" and "Load Test" at the Contract unit Price will be full compensation for the specified work.

12-07-95 M&R (JJB)

Texas DOT Special Specification (as of April, 1997)

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SPECIAL SPECIFICATION

ITEM 9000

AUGERED PRESSURE GROUTED PILES

9000.1. Description. This item shall govern for the construction of foundations consisting of reinforced concrete augercast piling of the size and at the locations shown on the plans.

A reinforced concrete augercast piling is hereby defined as a pile excavated with a continuous flight auger with concrete placement occurring through the hollow stem of the auger under pressure while concurrently withdrawing the auger from the excavation followed by placement of a reinforcing steel cage.

The foundation contractor shall provide the Engineer documentation of successfully installed augercast piles under similar job and subsurface conditions. He shall also provide a job supervisor who has a minimum of three years of method specific experience. Should the foundation contractor fail to demonstrate adequate past experience, a demonstration pile shall be installed and removed from the ground to demonstrate the soundness of the completed piling.

9000.2. Materials. Materials required for use under this item shall conform to the following:

Item 421, "Portland Cement Concrete"
Item 440, "Reinforcing Steel"
Item 448, "Structural Field Welding"

The minimum 28 day compressive strength for concrete shall be 4000 psi. Sampling of concrete for strength test specimens shall be from the top of the completed piling or as otherwise directed by the Engineer. The mix design shall be submitted to the Engineer for approval.

9000.3 Construction Methods.

(1) Excavation. The contractor shall perform the excavation required for the piling, through whatever materials encountered, to the dimensions and elevations shown on the plans.

The center of the piling shall be within one (1) inch from the location shown on the plans. Any piling in violation of this tolerance will be subject to a structural review by the Engineer.

(2) Concrete. Concrete placement shall begin immediately after the excavation is complete. Concrete shall be introduced into the excavation by pumping through the hollow stem of the auger. Only approved pumping equipment shall be used. The pump shall be a positive displacement pump capable of developing displacement pressures at the pump of not less than 350 psi. The pump shall be provided with a pressure gauge in clear view of the equipment operator and inspector. The pump shall be calibrated at the beginning of the work to determine the volume of concrete pumped per stroke. A positive method of counting pump strokes shall be provided by the pile contractor. Such methods may include digital or mechanical stroke counters of other acceptable methods.

The rate of concrete injection and rate of auger withdrawal from the soil shall be coordinated so as to insure that the auger is well submerged in the previously placed concrete at all times. As the auger is withdrawn, the inspector will verify that a sufficient volume of concrete has been placed to insure the continuity of the concrete pile.

The excavated soil shall be carefully removed from the vicinity of the completed piling to minimize concrete contamination. The upper five (5) feet of the concrete column shall be sieved to remove soil contamination. The concrete shall be sieved to greater depths if additional contamination is present.

(3) Reinforcing Steel. The cage of reinforcing steel, consisting of longitudinal bars and lateral reinforcement (spiral reinforcement or lateral ties) shall be completely assembled and placed as a unit immediately after concrete placement and sieving.

If the pile is lengthened, the longitudinal bars and lateral reinforcement required in the upper portion of the pile shall be extended to the bottom unless otherwise shown on the plans. These bars may be lap spliced or spliced by welding. Any splices required shall be in the lower portion of the pile.

Where spiral reinforcement is used, it shall be tied to the longitudinal bars at a spacing not to exceed 12 inches. Welding of lateral reinforcement to longitudinal bars will not be permitted unless otherwise shown on the plans.

Spacer devices shall be used at sufficient intervals to insure concentric spacing for the entire length of the cage. Spacers shall be placed at sufficient intervals around the steel cage to insure concentric spacing inside the excavation.

9000.4. <u>Measurement.</u> Augercast piling will be measured by the linear foot between the top of pile and bottom of pile.

9000.5. Payment. The work performed and materials furnished in accordance with this item and measured as provided under "Measurement" will be paid for at the unit price bid per linear foot of augercast piling of the specified diameter. The quantity to be paid for will be the quantity shown on the plans unless specific changes in length have been authorized in writing by the Engineer.

The unit prices bid for the various classifications of augercast piling shall be full compensation for making all excavations; for furnishing and placing all concrete including additional concrete required to fill an oversize excavation; for furnishing and placing reinforcing steel; for all backfilling; for disposing of cuttings; and for furnishing all tools, labor, equipment and incidentals necessary to complete the work. When the bottom of any augercast piling is ordered to be placed at an elevation below plan grade and a splice of reinforcement is required, no direct payment will be made for the extra reinforcement required, but it will be considered subsidiary to the price bid per foot of augercast piling. No additional payment will be made for casing or formwork either utilized or left in place.

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Industry Model Specification Reference

Augered Cast-in-Place-Pile Committee, *Auger Cast-in-Place Pile Model Specification*, Deep Foundations Institute, P. O. Box 281, Sparta, NJ 07871, 1990, 27 pp. (Copyrighted publication of the DFI).