Behavior of Piles in Full-Scale, Field Lateral Loading Tests

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Research Project GC 8286, Task 4
Piles — Lateral Load Testing

BEHAVIOR OF PILES IN FULL-SCALE, FIELD LATERAL LOADING TESTS

by

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16. ABSTRACT

This report documents the results of a full-scale, field lateral load testing program conducted at two sites in Washington state. The sites were chosen to represent soil conditions WSDOT geotechnical engineers commonly encounter and for which little information on the response of laterally loaded piles was available. One site consisted of a deep deposit of soft silt in which 18-inch diameter piles were being installed for replacement of an existing bridge. The other site consisted of a moderately deep deposit of peat that was suspected of causing foundation movements in an adjacent bridge structure. Full-scale, field lateral load tests were performed on two instrumented piles at each site. A high degree of consistency was observed between the results of the two tests at both sites The test results indicate that, for the loading conditions imposed during the tests, the response of the soil to lateral pile movement can be described by the Integrated Clay Criterion previously developed by researchers at the University of Houston. Integrated Clay Criterion parameters for the soils at each site are developed from interpretation of the test results.

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SUMMARY

The purpose of the research described in this report was to investigate the resistance of soils to lateral pile deflection in Washington state soil conditions for which the nature of this resistance has not been previously established. The objectives of the research were accomplished by the following steps: (1) review and prioritization of soil conditions in Washington for which laterally loaded pile behavior is not well known; (2) identification of potential field lateral load test sites and selection of two sites for testing; (3) performance of two full-scale, field lateral load tests at the two sites; and (4) reduction and interpretation of test results.

The pile .load test sites were chosen to represent soil conditions WSDOT geotechnical engineers commonly encounter and for which little information on the response of laterally loaded piles was available. One site consisted of a deep deposit of soft silt in which 18-inch diameter piles were being installed for a bridge replacement. The other site consisted of a moderately deep deposit of peat that was suspected of causing foundation movements in an adjacent bridge structure. Full-scale, field lateral load tests were performed on two instrumented piles at each site. A high degree of consistency was observed between the results of the two tests at each site.

The test results indicated that, for the loading conditions imposed during the tests, the response of the soil at each site to lateral pile movement can be described by the Integrated Clay Criterion previously developed for cohesive soils by researchers at the University of Houston. Integrated Clay Criterion parameters for the soils at each site were developed from interpretation of the results of the pile load tests. The Integrated Clay Criterion can be incorporated into the computer programs WSDOT engineers already use for analysis of laterally loaded piles; consequently, implementation of the research results should be rapid and easy.

INTRODUCTION AND RESEARCH APPROACH

Many large transportation structures, such as highway bridges, are supported on pile foundations. These structures impose vertical, gravity loads on the foundations, but they may also impose significant lateral loads as well. Consequently, the design of a pile foundation must consider the pile response to lateral as well as vertical loads. A number of analytical methods are available for evaluating pile response to lateral loading. These methods range from the very simple to the very sophisticated. The most simple methods are empirically based and, though easy and rapid to use, do not provide the accuracy usually required for design purposes. The most sophisticated methods are rationally based; however, the time and expertise required to perform them render them impractical for all but the most critical of structures. An intermediate method, known as p-y curve analysis of laterally loaded piles, has been accepted as an accurate and reliable method for analysis of laterally loaded piles.

In p-y curve analysis procedures, the resistance of the soil to lateral pile movement is described by p-y curves. A number of procedures have been proposed for the development of p-y curves for different types of soil. These procedures are based on the results of full-scale, field lateral loading tests on piles. A recent study has shown that available p-y curve development procedures for clays and for sands are based on a significant number of carefully conducted field tests on piles embedded in these soil conditions. However, the same study showed that little or no data were available for development of p-y curves for other types of soil that are commonly encountered in the design of bridge foundations in Washington. In particular, the laterally loaded pile behavior of soils such as silts, peats, and gravels has not been studied.

This report presents the results of an experimental investigation of the lateral load resistance of piles obtained from full-scale, field lateral loading tests. Two sets of experiments, each consisting of two pile load tests, were conducted in this investigation.

The first set of piles was embedded in soft silts on the banks of the Deep River near Naselle, Washington. The second set of piles was embedded in the peats of Mercer Slough in Bellevue, Washington. These tests were performed as part of a University of Washington research project sponsored by the Washington State Department of Transportation (Basic Agreement No. GC 8286, Task Order No. 4).

The general approach taken in this research study was to perform lateral load tests on instrumented piles in the field and then to interpret the results of the tests in terms of p-y curve criteria. The interpretation process involved detailed analysis of the pile deformation measured by the pile instrumentation. The instrumentation data were interpreted within the framework of existing p-y curve criteria to determine whether the existing criteria were capable of representing the behavior observed in the lateral load tests, or whether modifications to these criteria were required.

DEEP RIVER LATERAL LOAD TESTS

Background

The Washington State Department of Transportation (WSDOT) is in the process of replacing the Deep River Bridge, which carries SR4 traffic over Deep River approximately 6 miles southeast of Naselle, Washington. The 1,388-foot-long new bridge, located about 45 feet north of the old bridge, is a 32-span, prestressed concrete girder structure that provides two 12-foot lanes and two 6-foot shoulders. Approach embankments up to 20 feet high are at each end of the structure.

Site and Subsurface Conditions

Deep River Bridge spans the Deep River approximately 2 miles north of Grays Bay near the mouth of the Columbia River in southwestern Washington. The elevation of the river, which is lined with small earthen levees near the bridge, is influenced by tidal changes. The ground surface near the river is flat and covered with grasses and small trees typical of such wetland areas. The ground surface elevations in these areas ranges from about +3 feet to +7 feet (National Geodetic Vertical Datum of 1929). At the time of the pile lateral load tests, the approach embankments were under construction.

Bedrock in the Deep River Bridge area is early to middle Miocene bathyal sediments of the Astoria Formation. In a rather quiet environment, the shelf collected thick argillaceous silt to fine sand sediments, which lithified and were uplifted several million years ago. The rock is poorly indurated and generally void of bedding surfaces. The bedrock is unconformably overlain by poorly stratified Quaternary alluvium ranging from argillaceous silts to silty fine sands with minor amounts of wood and organics.

Subsurface conditions at the Deep River Bridge site were described in the reports of two investigations conducted by WSDOT. The first report described the results of a preliminary geotechnical investigation for the Deep River bridge approach fills. (1) The second report presented the results of geotechnical investigations of the bridge and the approaches. (2) In these investigations, 15 borings were drilled and sampled. The locations of the borings, along with the inferred subsurface profiles, are shown in Figures 1 and 2. Standard penetrometer tests were typically taken at 5-foot intervals, and the resulting disturbed samples were retained for visual classification in the field and more detailed classification in the Materials Laboratory. Undisturbed soil samples and rock cores were also obtained. Three cone penetration test (CPT) soundings were also performed as part of a supplementary subsurface investigation in December 1989. Two of the soundings, CPTA and CTPB, were located at Station 145+00 near Pier 21. The other sounding, CTPC, was located at Station 141+30, between Piers 13 and 14. The results of these CPT soundings are shown in Figures 3, 4, and 5. The subsurface investigation indicated that the ground surface is generally underlain by up to 125 feet of alternating discontinuous layers of very loose to loose silty fine sand and fine sandy silt containing varying amounts of wood, clay, and organic material. These soils are underlain by 6 to 133 feet of similar material, which is medium dense to dense and in turn underlain by silty sandstone and fine sandy siltstone bedrock.

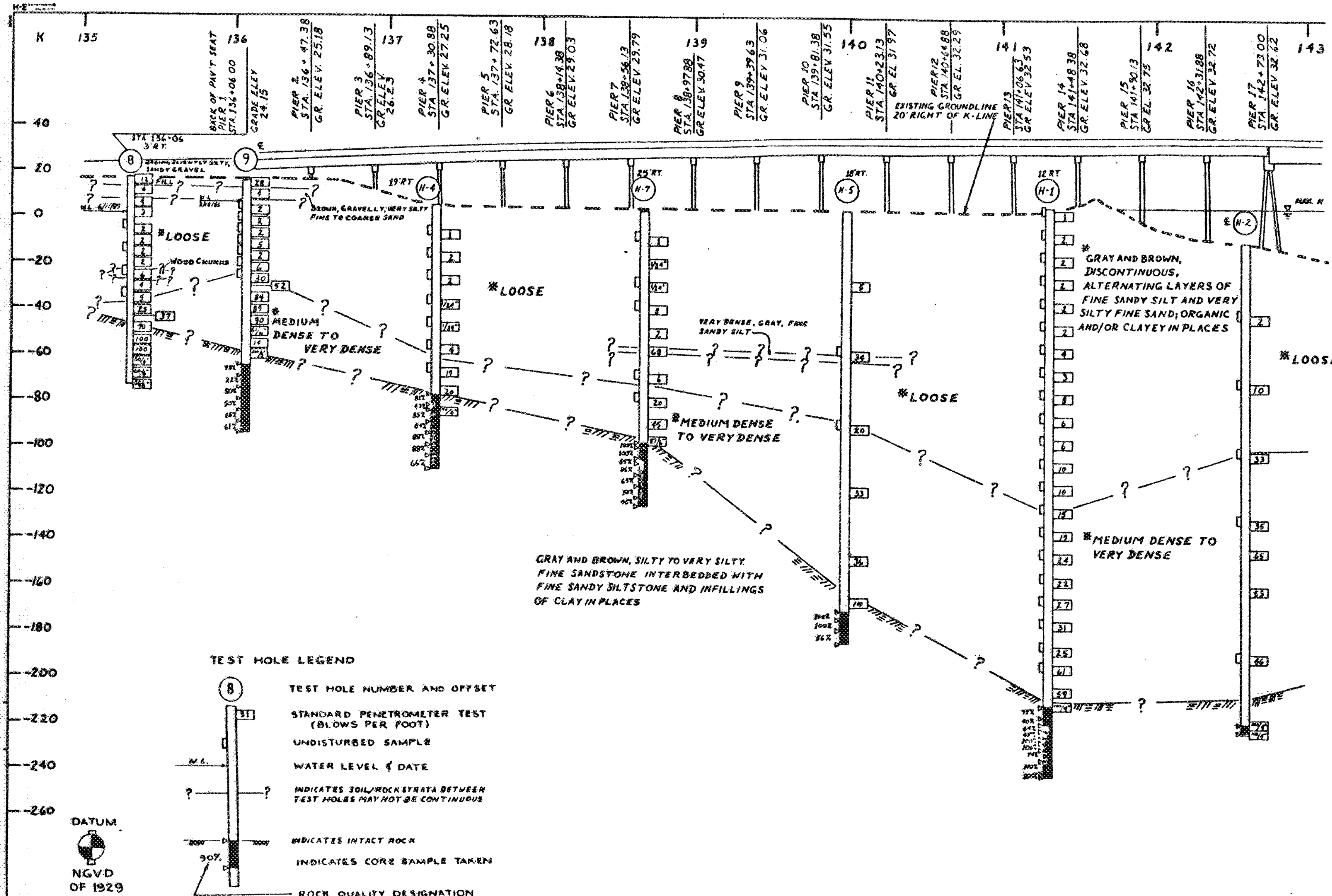

Figure 1. Subsurface Soil Profile with Boring Locations — West End of Deep River Bridge

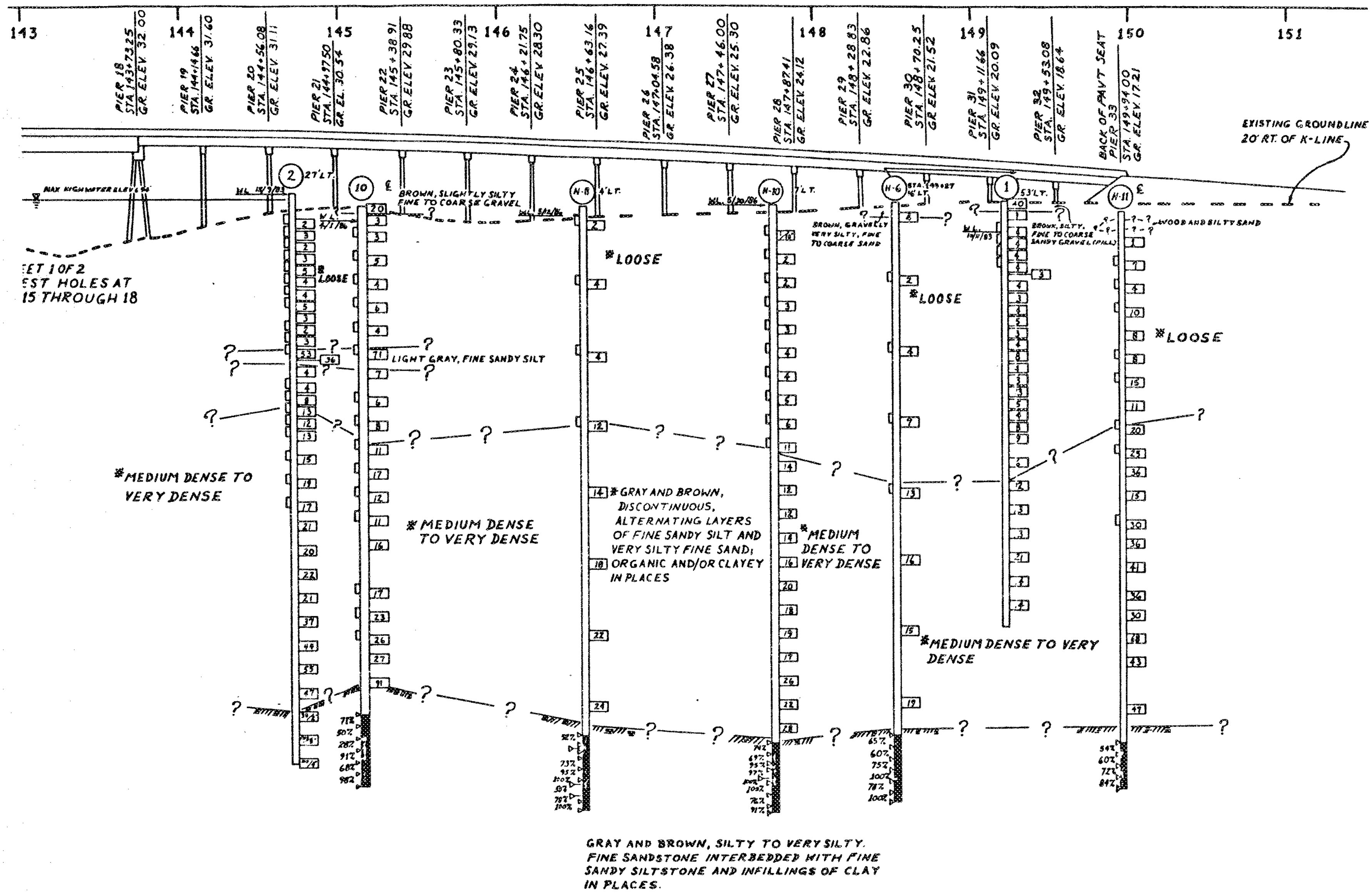

Figure 2. Subsurface Soil Profile with Boring Locations — East End of Deep River Bridge

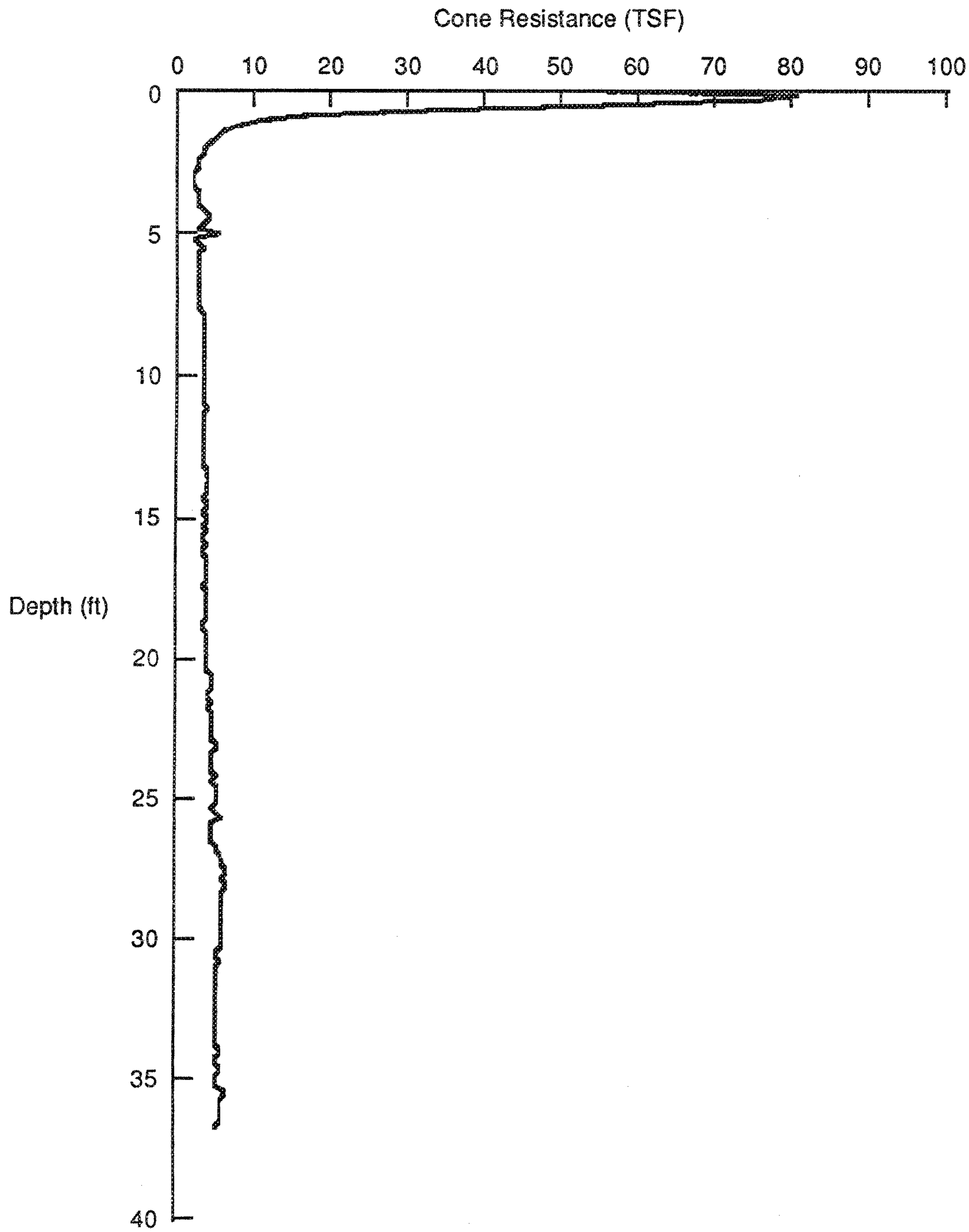

Figure 3. Deep River CPT Sounding CPTA

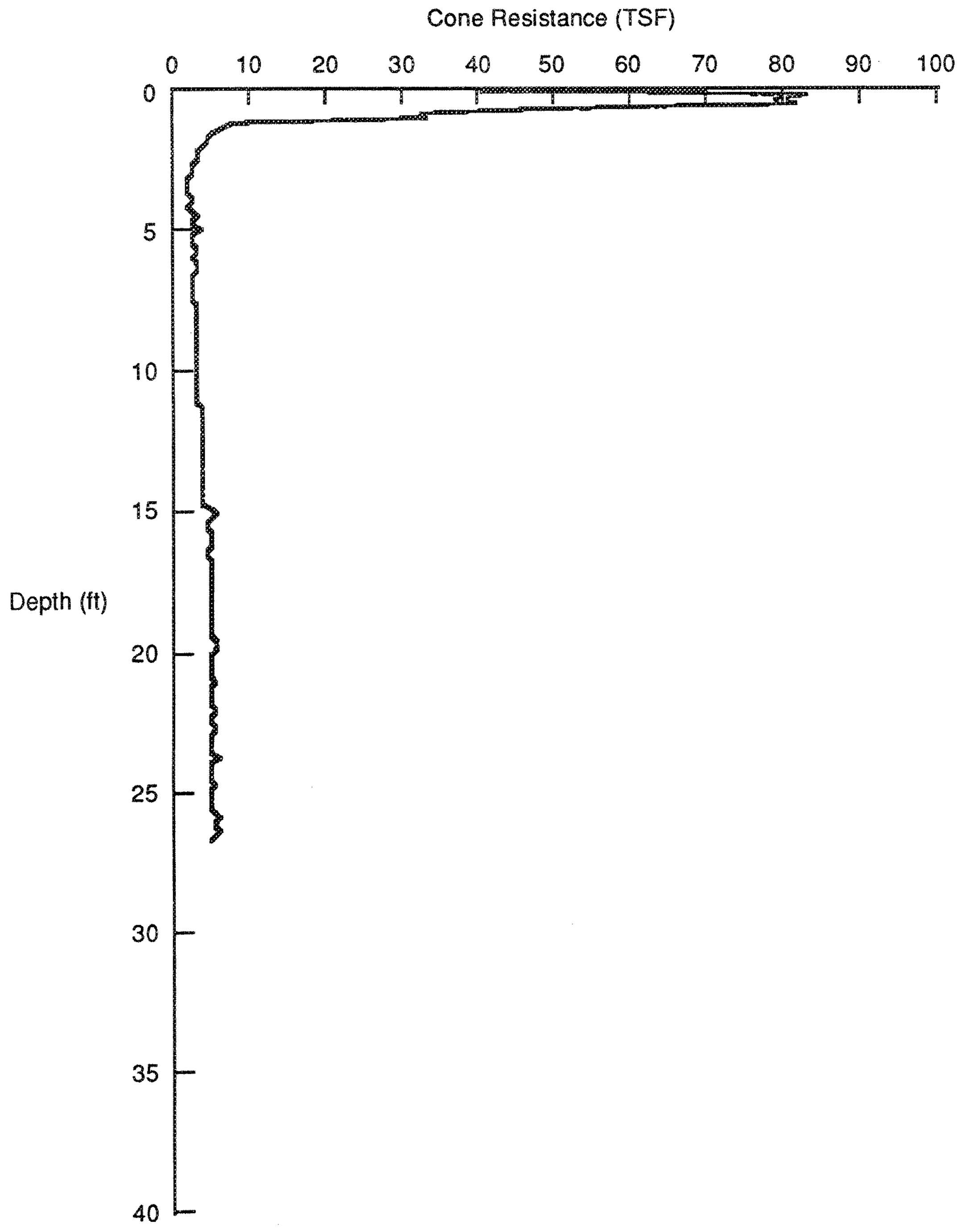

Figure 4. Deep River CPT Sounding CPTB

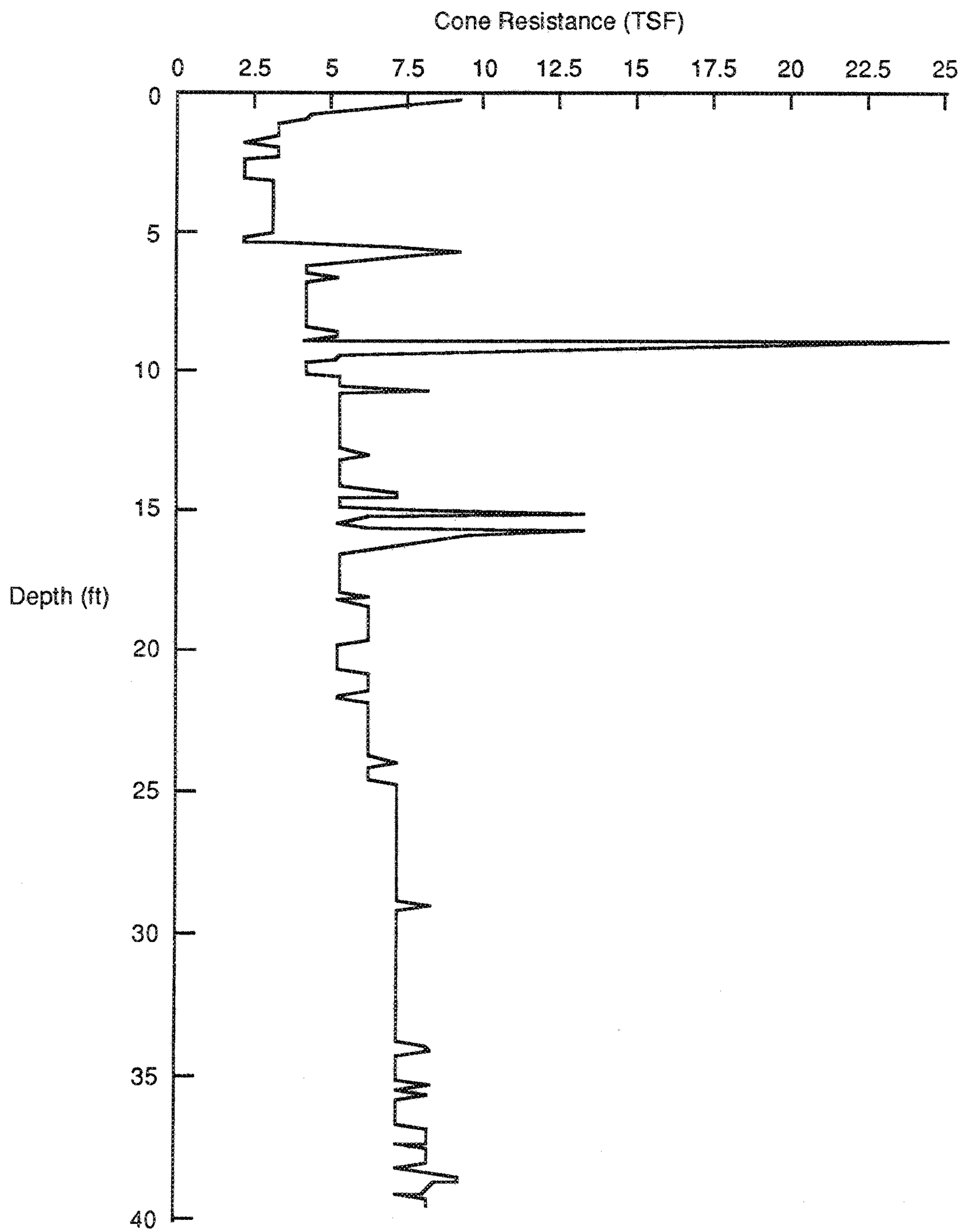

Figure 5. Deep River CPT Sounding CPTC

Test Materials, Instrumentation, and Procedures

Two pile lateral load tests were performed at the Deep River Bridge site. Each of the tests was performed on indicator piles installed solely for the purpose of vertical load testing. The indicator piles consisted of 18-inch diameter, 80-foot long, steel pipe sections spliced above timber pile sections. Each indicator pile was driven so that the top of the pile extended approximately 3 feet above the ground surface. The indicator piles were surrounded by four 14-inch diameter, slightly (1:12) battered timber piles installed for the purpose of supporting a reaction platform for the vertical load tests, as shown in Figure 6. The lateral load tests were performed over 3 months after completion of the vertical load tests.

Test Pile A, hereafter referred to as TPA, was required by contract documents to be installed by the contractor at or between Piers 22 through 29 within 50 feet of the centerline of the new structure. The contractor installed TPA away from the east bank of the river near Pier 24 and Boring H-8. A plan view of the TPA setup is shown in Figure 7. The subsurface investigation indicated that the soft silty soil extends to approximately elevation -90 feet, where it is underlain by about 120 feet of medium dense silty soil over bedrock. Standard penetration resistances ranged from 2 to 4 blows/foot in the upper 60 feet near TPA.

Test Pile B (TPB) was required to be installed at or between Piers 12 through 15 and also within 50 feet of the centerline of the new bridge structure. The contractor installed TPB at the top of the levee on the west bank of the Deep River near Pier 14 and near Boring H-1. The pile was installed in this location apparently for the convenience of the contractor; however, its location in the levee soils was poor from the standpoint of pile lateral load test performance and interpretation. Plan and profile views of the TPB test setup are shown in Figures 8 and 9. The results of the subsurface investigation indicated that, around TPB, the soft silty soils extend to approximately elevation -120 feet, where they are underlain by approximately 80 feet of the medium dense to dense

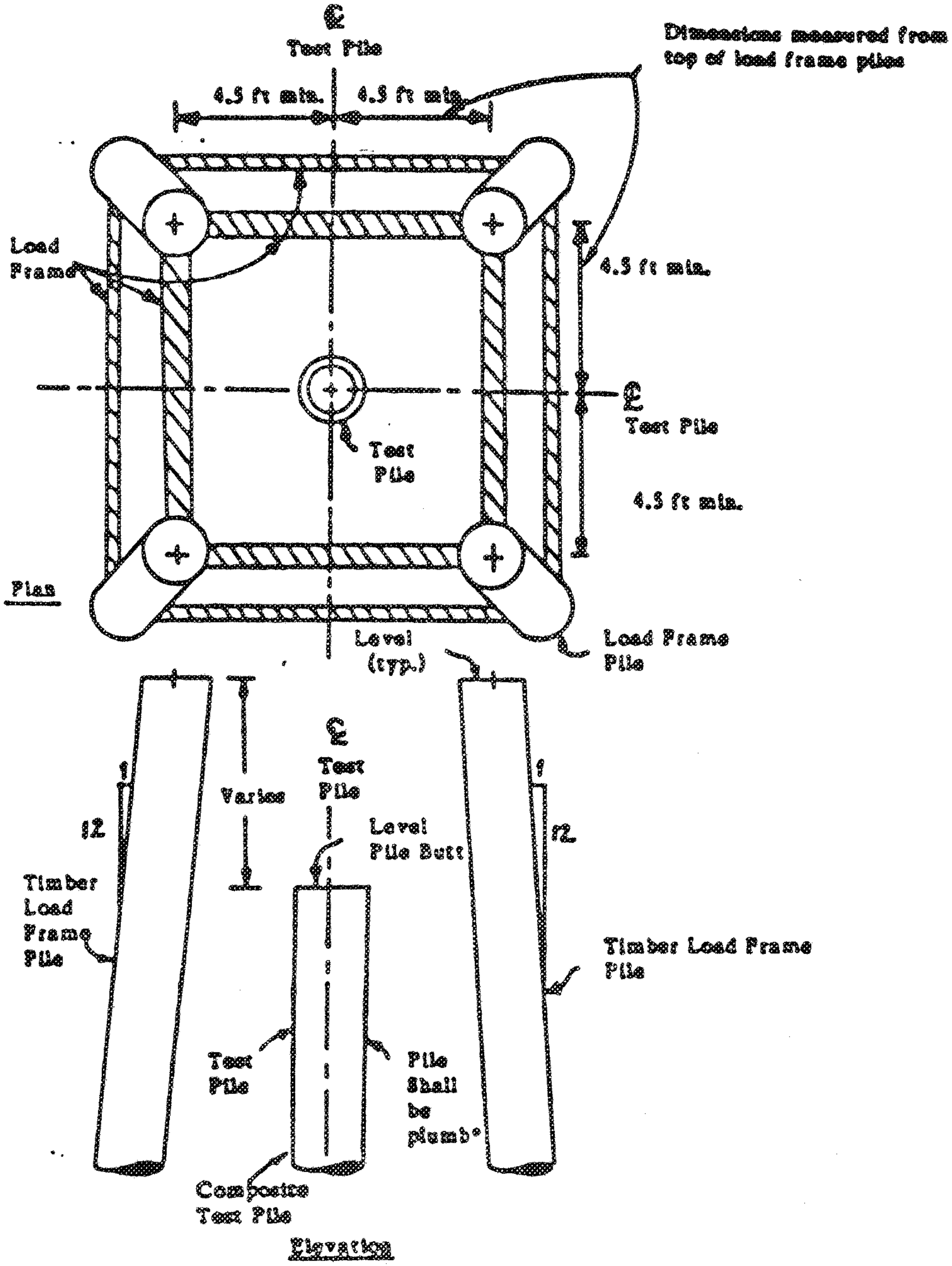

Figure 6. Reaction Pile Assembly for Deep River Pile Load Tests

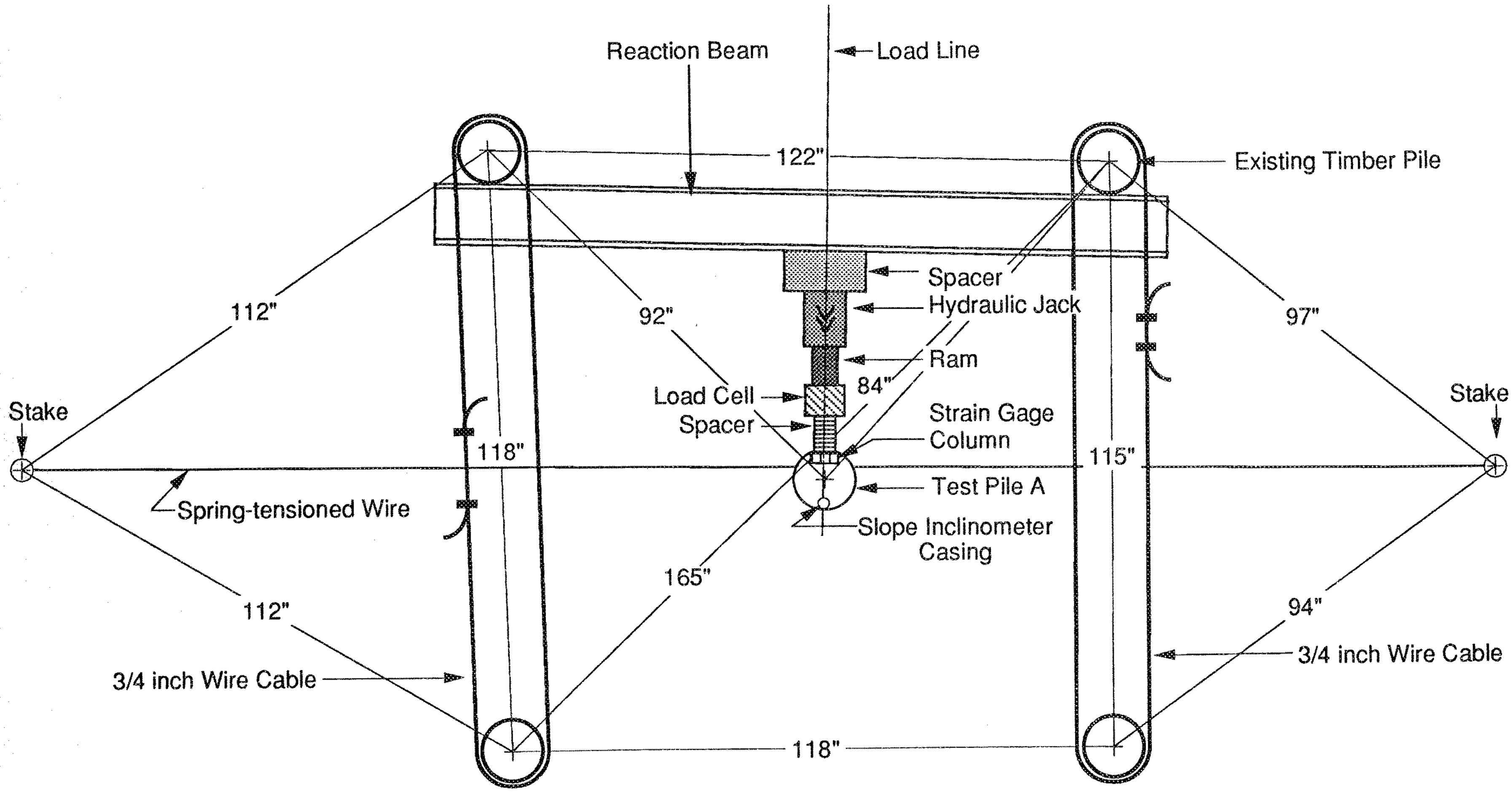

Figure 7. Plan View of Deep River TPA Test Setup

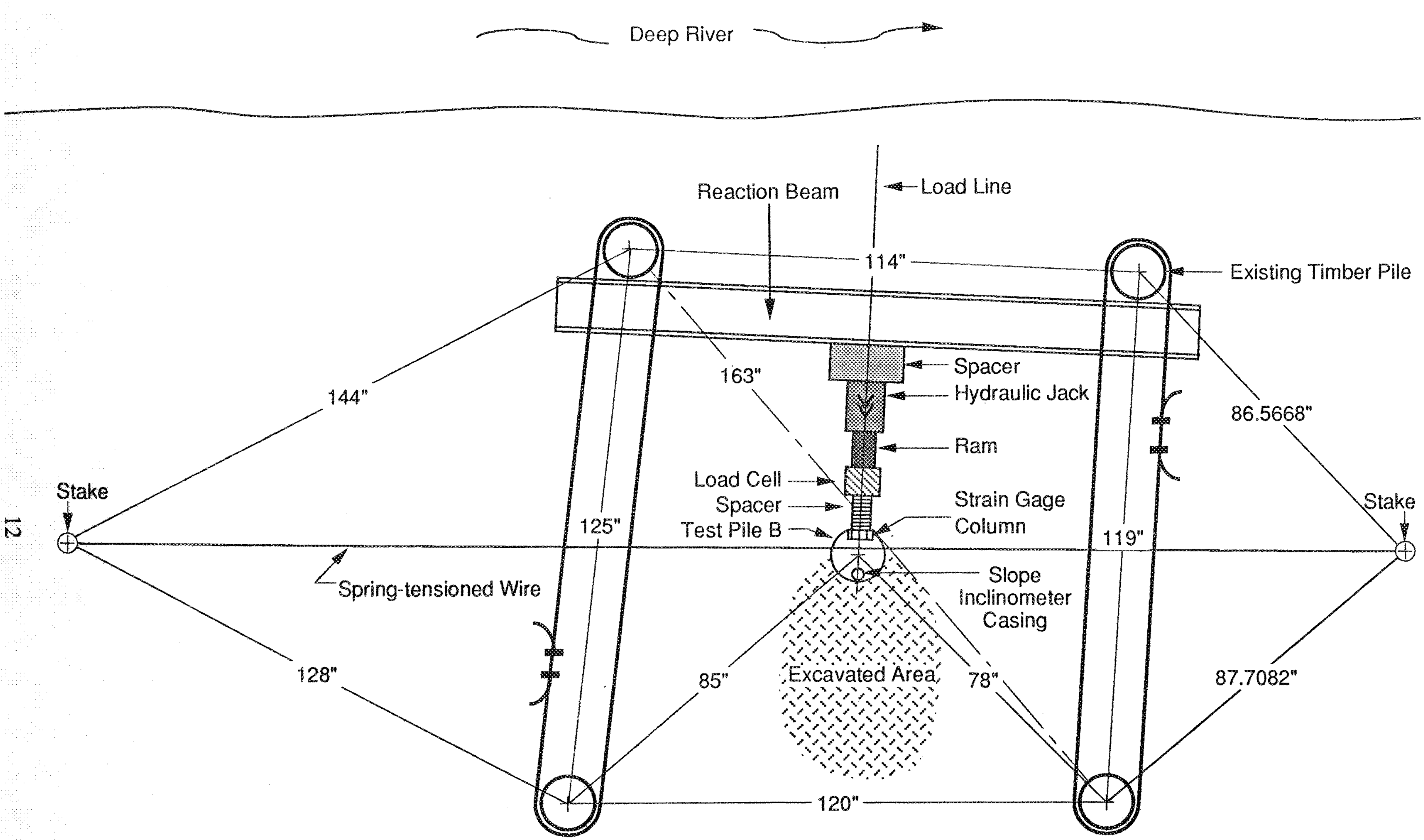

Figure 8. Plan View of Deep River TPB Test Setup

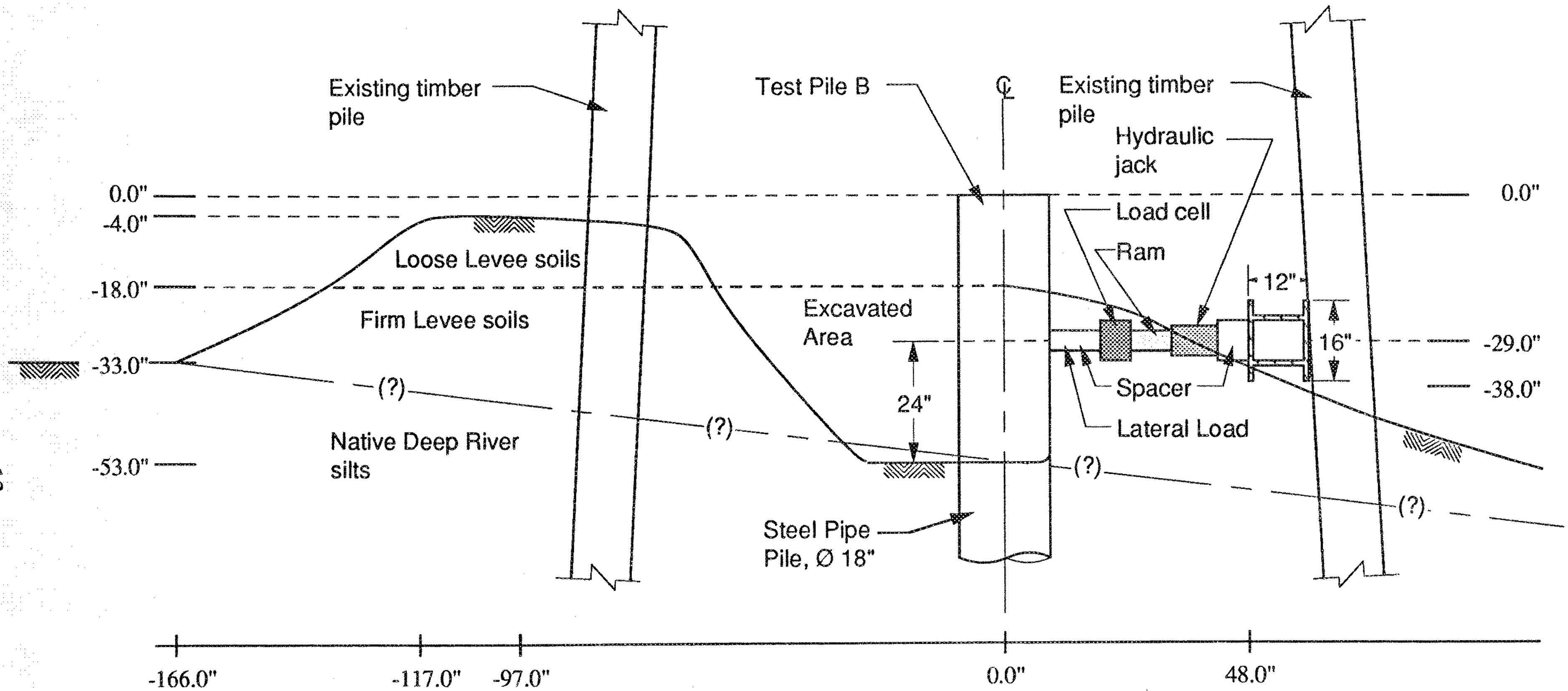

Figure 9. Profile View of TPB Test Setup

silty soil over bedrock. In the upper 50 feet of soft silty soil, the standard penetration resistance was fairly consistent at about 2 blows/foot.

The Deep River pile lateral load tests were somewhat unusual in that the piles had already been installed when the decision to perform the lateral load tests was made. Consequently, there was no opportunity to install conventional instrumentation before the piles' installation. Therefore, an innovative instrumentation system had to be developed. To accurately interpret the p-y behavior of the soil surrounding the pile, the displaced shape of the pile corresponding to the various lateral loads applied at the top of the pile had to be measured. Since such measurements were desired to depths of 35 to 40 feet, and since nothing could be firmly attached to the sides of the pile more than an arm's length below the top of the pile, an instrumentation system had to be developed that could be inserted and operated inside the pile.

An instrumentation scheme consisting of both slope and curvature measurement systems was developed specifically for the Deep River lateral load tests. Slope measurements were made with a conventional slope inclinometer with a plastic inclinometer casing. The plastic inclinometer casing, consisting of four 10-foot lengths aligned and cemented together, was suspended inside the pile with a small chain attached to the top of the pile. Curvature measurements were made with a strain gauge column constructed from four 10-foot lengths of 3-inch square flexible, PVC box sections with a 1/16-inch wall thickness. To fabricate the strain gauge column, each of the box sections was first cut in half lengthwise and laid side-by-side on a flat surface. Resistance strain gauges were then placed at identical locations on each pair of box section halves. The strain gauges were wired, along with dummy gauges, into temperature-compensating bridges whose input and output cables extended along the length of the strain gauge columns. The opposing halves were then cemented together along their lengths and flexibly connected at their ends. The result was a 40-foot-long, flexible column with pairs of oppositely-mounted strain gauges that could measure curvature of the column at the strain gauge locations. The strain gauge bridges were powered by a separate dc power supply. The flexible connections allowed the column to be folded at the three interior joints so that only a 10-foot long section had to be transported. The strain gauge column was also suspended by a small chain inside the pile directly opposite the slope inclinometer casing.

In order for the shape of the inclinometer casing and that of the strain gauge column to match the deflected shape of the pile caused by lateral loads at the surface, both the inclinometer casing and the strain gauge column had to be forced into contact with the interior walls of the pile. This contact was accomplished with the insertion and subsequent inflation of a polyethylene air bag in the center of the pile. The air bag, which was 50 feet long and had an unconstrained inflated diameter of approximately 3 feet, is shown during test inflation in Figure 10. The air bag was designed with this large diameter to eliminate circumferential membrane tension when inflated inside the 18-inch diameter pile. The unconstrained ends of the bag were heavily reinforced with duct tape. Since both the plastic inclinometer casing and the PVC strain gauge column had very low flexural stiffness, and since the pile curvatures during loading were expected to be low, they could both be forced against the sides of the pile by a relatively low air pressure of approximately 1.5 psi in the air bag. A high volume, low pressure air pump (Shop Vac) was used to inflate and maintain air pressure in the air bag. The air bag system functioned well and suffered only occasional minor air leaks near the top of the air bag. Measurement of pile slope with the air bag/inclinometer system is shown in Figure 11.

Lateral loads were applied to the piles by a 100-ton capacity, 9-inch throw, hydraulic jack. The lateral loads applied to each pile were measured with a GEOKON Model 3000 load cell provided by WSDOT. The deflection at the point of load application was obtained with a measurement of the horizontal distance between each pile and a spring-tensioned horizontal wire stretched between stakes placed firmly in the soil outside the piles' zone of influence. The slope at the top of the piles was determined with

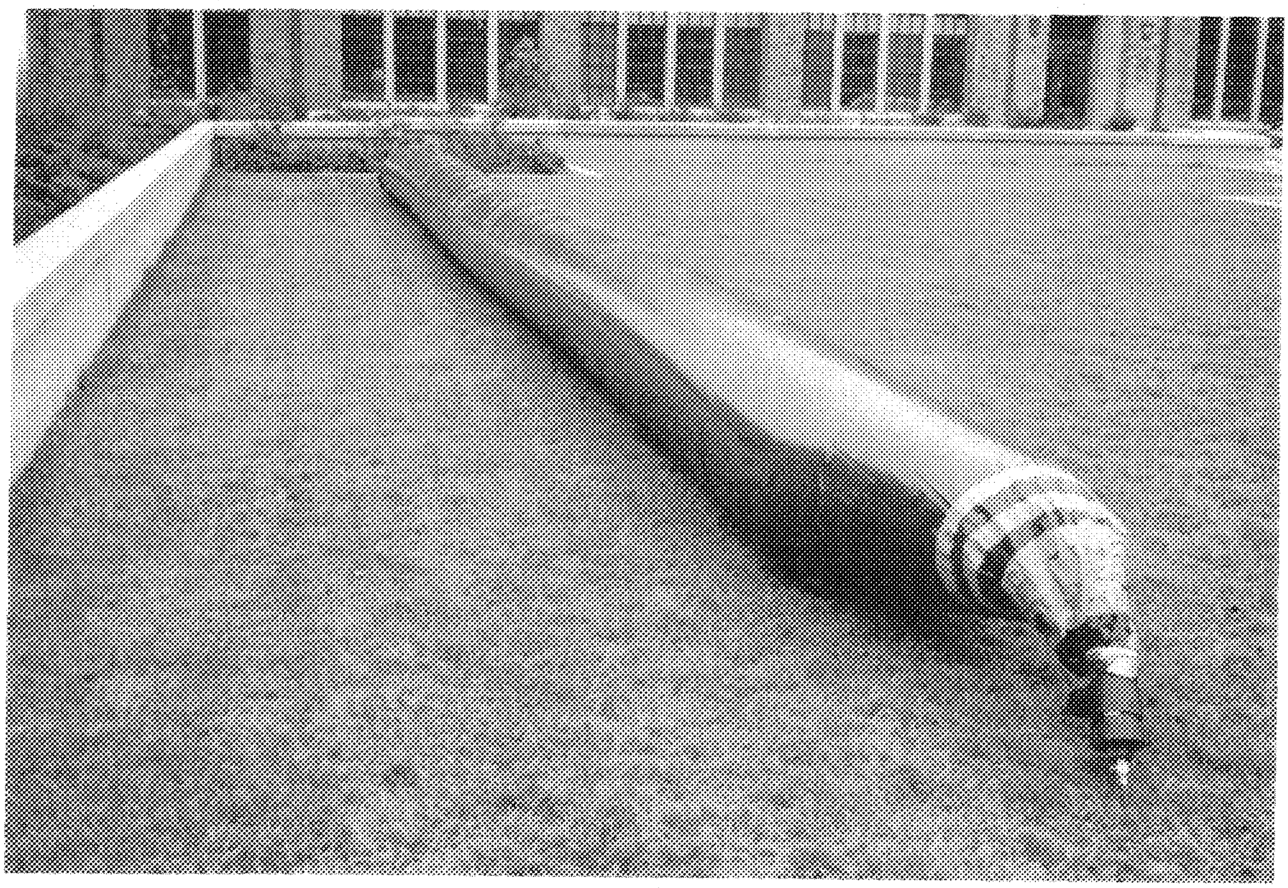

Figure 10. Deep River Test Air Bag During Test Inflation

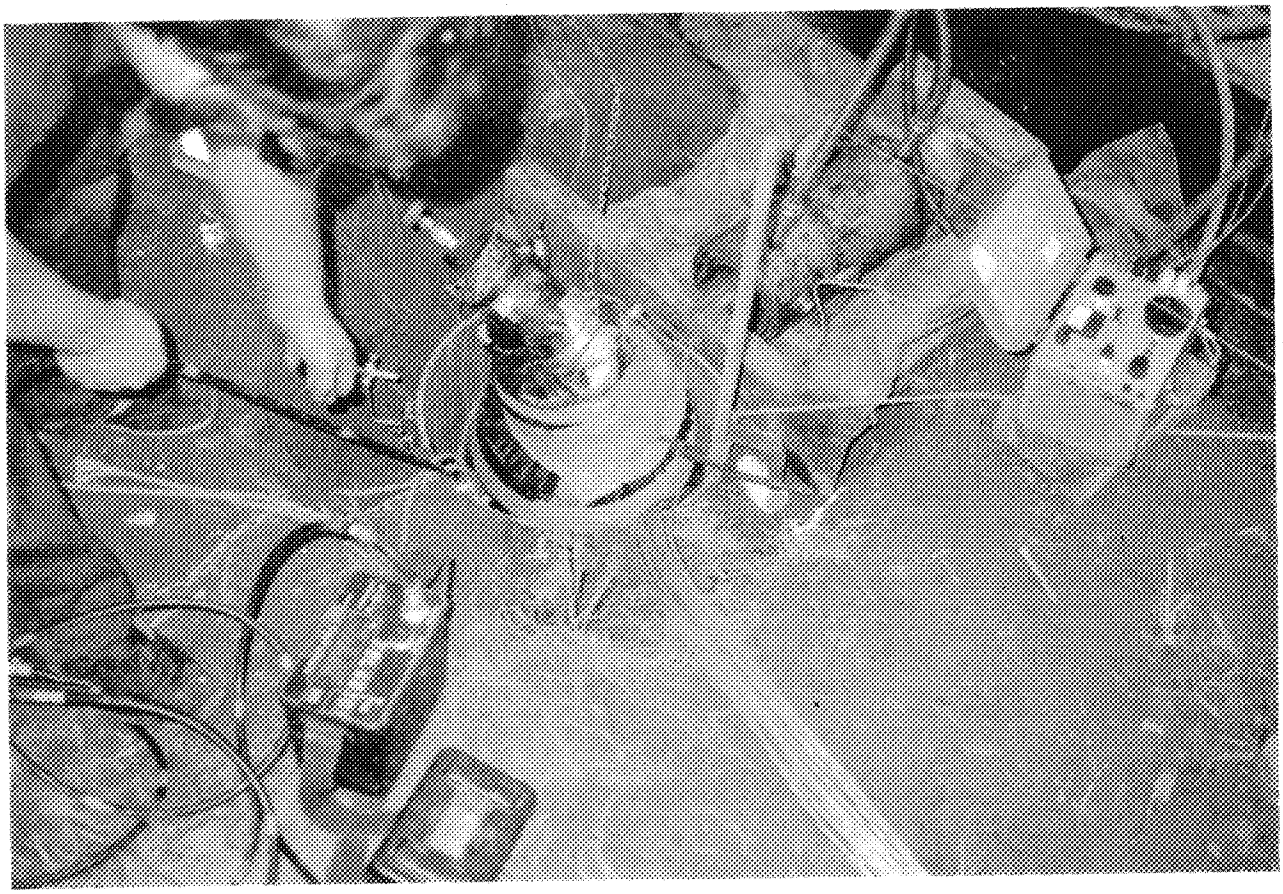

Figure 11. Inclinometer Reading on TPA

a system in which the difference in elevation between two points located 24 inches apart on a straight plate was measured with a level and rule. The subsurface deformation of each pile was monitored by both the strain gauge column and the inclinometer. The strain gauges were monitored during testing by a PC-based automatic data acquisition system powered by a dedicated portable generator. WSDOT District 4 personnel made slope measurements at 2-foot intervals along the length of the inclinometer. WSDOT Materials Lab personnel reduced the slope inclinometer data.

A site visit approximately 2 months before the load tests were performed revealed that the test piles had been installed with their tops approximately 3 feet above the ground surface. The load reaction system, which was designed on this basis, consisted of a 12-foot-long steel WF section that would be laid on the ground surface just inside and adjacent to two of the timber piles. These piles were to be lashed with steel cable to the other two timber piles, and the reaction beam was to be shimmed to vertical against the battered timber piles. The hydraulic jack and load cell assembly was to be placed between the reaction beam and the test pile so that the lateral load applied to the pile would be resisted by all four of the timber piles.

However, upon returning to the site to perform the load tests, the researchers found that the portion of TPA that had extended above the ground surface had been cut off, leaving the top of the pile essentially at the ground surface. In order to apply lateral load to the pile, the researchers had to manually excavate trenches in which the reaction beam, hydraulic jack, and load cell assembly could be placed. The contractor had also excavated around each timber reaction pile to a depth of about 2 feet, thus reducing the lateral stiffness of the reaction system. The test configuration for TPA, including the reaction beam and loading system trenches, is shown in Figure 12. The high groundwater level can be seen in the excavation around the timber reaction pile. TPB had been installed, apparently for the convenience of the contractor, near the crest of the levee on the east bank of the river. While installation in the levee would have negligible effect on

Figure 12. TPA Test Setup

the results of the vertical load tests for which TPB was installed, the levee soils would have considerable influence on the results of the lateral load tests performed in this investigation. For this reason, the researchers had to manually excavate the levee soil immediately surrounding the pile to a depth of up to 4 feet on the side toward which the pile would be deflected before performing the test. This excavation depth was sufficient to reach the apparent boundary between the levee soils and the underlying natural soils. The test configuration for TPB is shown in Figure 13.

MERCER SLOUGH LATERAL LOAD TESTS

Background

WSDOT had observed subsurface soil movement in the area of the east-bound collector-distributor (EBCD) and west-bound collector-distributor (WBCD) ramps of the SR-90 Bellevue Transit Access project. Subsurface soil movement had apparently resulted in movement of pile-supported structures in the area. A draft subsurface exploration and geotechnical engineering report described geotechnical conditions at the site. (3) Because of the results of the geotechnical investigation, preliminary analyses of laterally loaded piles in the Mercer Slough peat were based on a modified form of the Soft Clay Criterion proposed by Matlock. (4) Because of the variability of the subsurface materials and the uncertainty of the application of these criteria to these soils, two lateral load tests were performed in this research investigation to evaluate the lateral load resistance of the peats of Mercer Slough and to supplement the preliminary analyses.

Site Conditions

The site is located on the SR-90 right-of-way immediately west of Lake Washington Boulevard. The site is traversed in the east-west direction by a Seattle Water Department water line and four SR-90 elevated bridge structures. The site is part of Mercer Slough, which in this area has a generally flat and level surface covered with marsh grasses and small trees. The groundwater level at the site is approximately at the

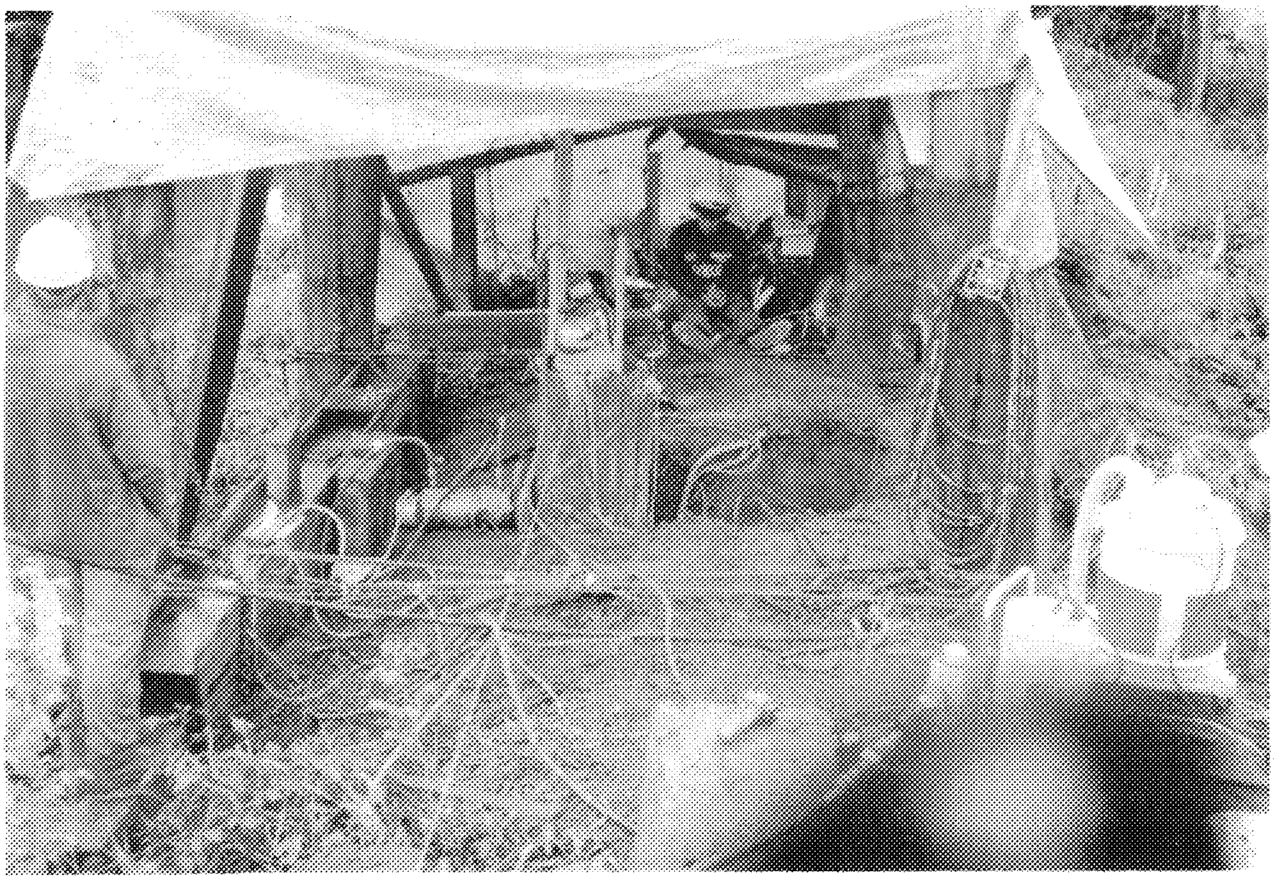

Figure 13. TPB Test Setup

ground surface. The eastern edge of the site is bordered by the Lake Washington Boulevard embankment fill, which rises to an elevation about 15 feet above the remainder of the site.

The site subsurface conditions are dominated by a peat deposit of variable thickness, as described by a preliminary geotechnical report by Rittenhouse-Zeman Associates, Inc. (3) The peat deposit generally overlies clay and silt deposits, which overly granular materials. Artesian pressure conditions have been observed in the soils underlying the peats. The peat was described as a "brown, fibrous, organic material, with a low dry density and shear strength and high water content and compressibility." (3) Vane shear tests indicated that the shear strength of the peat is variable, with an average value of 105 psf.

Test Materials, Instrumentation, and Procedures

In order for significant pile curvature to develop in the very soft soils of Mercer Slough, the use of flexible piles was required. The piles used in the lateral load tests were 8-inch diameter steel pipe piles with a 0.25-inch wall thickness. The piles were nominally 60 feet long with open ends. They were allowed to penetrate under their own weight as far as possible, and then an additional static vertical load was supplied by the boom of a WSDOT boom truck. Test pile 1 (TP1), located just south of WBCD Pier No. 35, penetrated approximately 15 feet under its own weight and was then pushed to refusal at a tip depth of about 51 feet. The pile, originally 59 feet long, was then removed and reinstalled nearby after 5 feet were cut off the bottom. Test pile 2 (TP2), located adjacent to EBCD Pier No. 32, penetrated about 20 feet under its own weight and was then pushed to refusal at a tip depth of approximately 43 feet. The pile, originally 60 feet long, was removed and reinstalled nearby after 14 feet were cut off the bottom. After installation, the tops of TP1 and TP2 were 5 and 3 feet above the ground surface, respectively. A site plan indicating the locations of TP1 and TP2 is shown in Figure 14. A profile showing subsurface soil conditions along the EBCD ramp is shown in Figure 15.

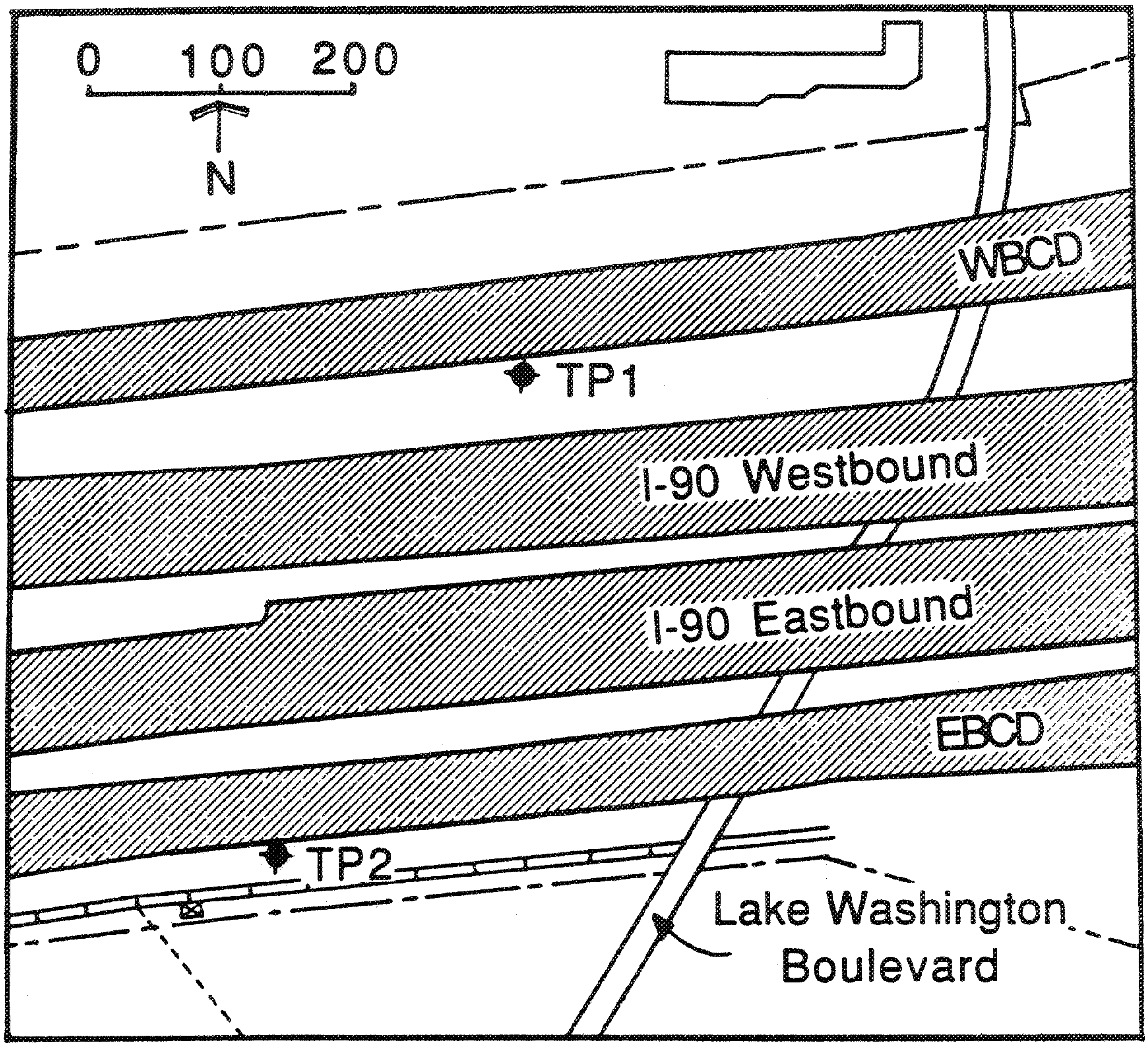

Figure 14. Mercer Slough Site Plan

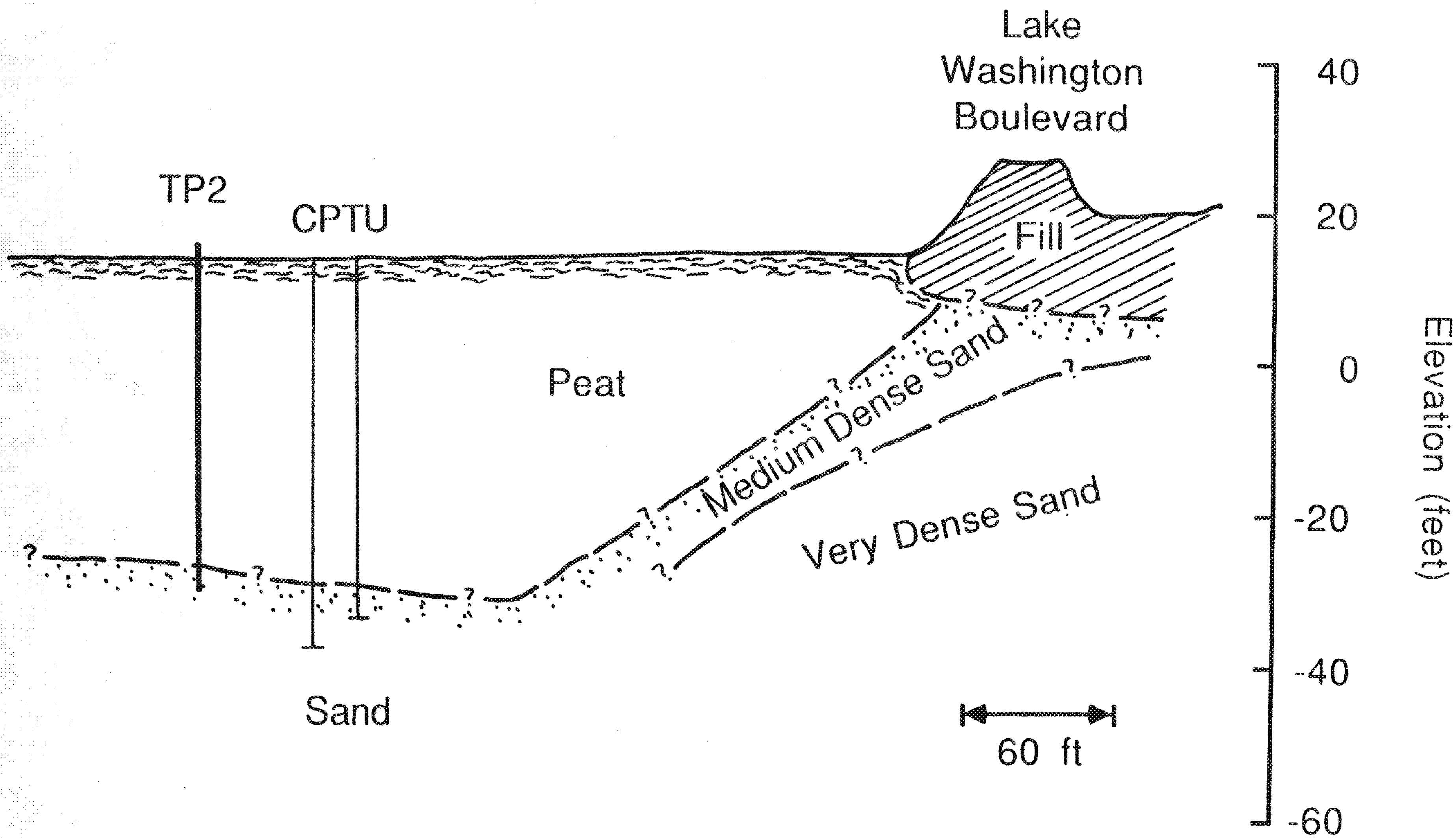

Figure 15. Mercer Slough Subsurface Profile

Lateral loads were applied to the piles by a 100-ton capacity, 9-inch throw, hydraulic jack. The deflection and slope of the piles at the point of load application were obtained from a measurement of the horizontal distance between each pile and each of three spring-tensioned horizontal wires stretched at different heights between stakes placed outside the pile's zone of influence. The subsurface deformation of each pile was monitored with both strain gauges and an inclinometer. The piles were instrumented with 11 pairs of bonded resistance strain gauges placed diametrically at distances of 5'8", 6'4", 7', 8', 9', 11', 13', 17', 23', 29', and 37' from the top of the pile. The strain gauges and associated wiring were protected by 1-1/2 inch steel angles lightly tack-welded to the outside of the piles. The strain gauges were monitored during testing by a PC-based automatic data acquisition system housed at the site in a small tent and powered by a dedicated generator. A 40-foot long slope inclinometer casing was suspended inside each of the piles and pressed against the sides of the piles by an inflatable air bag in the same manner as described for the Deep River load tests. WSDOT District 1 personnel made slope measurements at 2-foot intervals along the length of the inclinometer casing. WSDOT Materials Lab personnel reduced the slope inclinometer data.

On TP1, the loads were applied through a cabling arrangement so that the test pile was pulled toward the reacting bridge pier located about 12 feet away in the configuration shown in Figure 16. On TP2, the pile was jacked away from a nearby pile cap in the configuration shown in Figure 17. Applied loads on TP1 were measured by a GEOKON Model 3000 load cell WSDOT provided. The output from this load cell proved to be quite low for the load range used in the tests, and it was replaced by a load cell from the University of Washington structural engineering laboratory for TP2. The load cell used for TP2 was approximately seven times more sensitive than that used for TP1. Lateral loads were increased incrementally by an electrically-controlled hydraulic pump. The resistance of the peat to lateral loads was observed to be time dependent, as the pile head load decreased with time under constant pile head deflection. The top deflection and

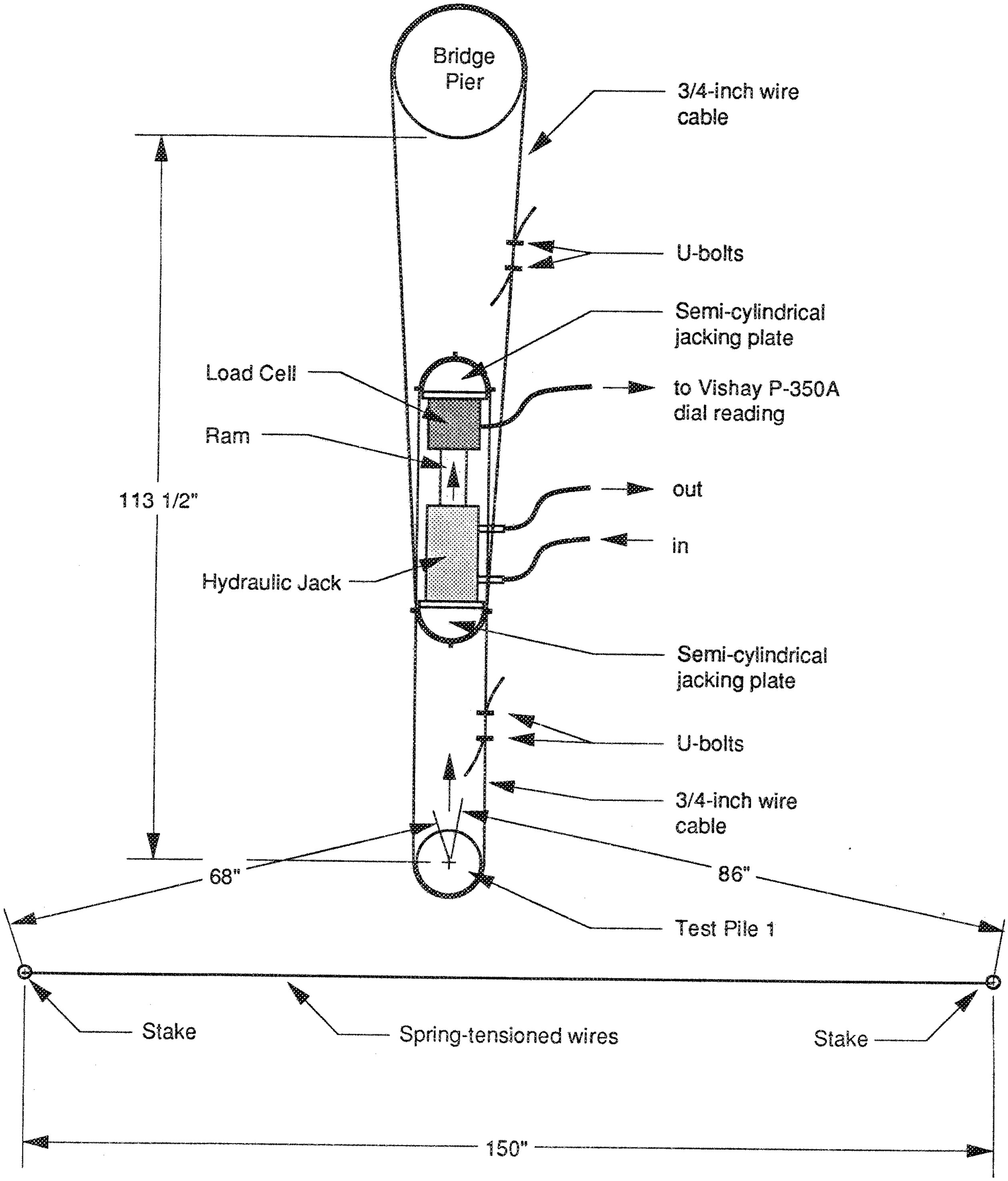

Figure 16. Plan View of Mercer Slough TP1 Test Setup

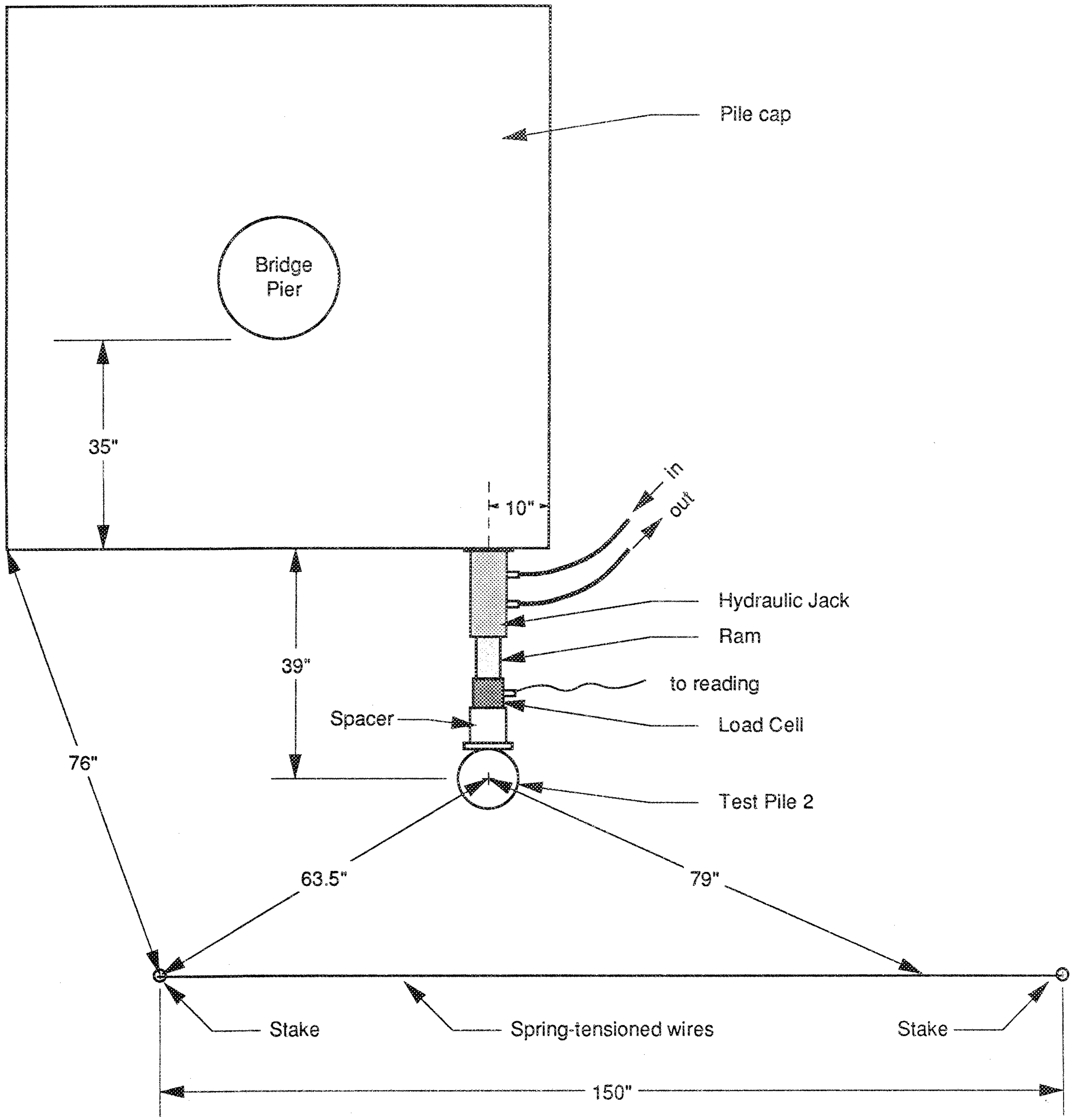

Figure 17. Plan View of Mercer Slough TP2 Test Setup

slope, and the load cell and strain gauges were read immediately after the application of each load increment and again after a period of approximately 10 to 15 minutes, at which time inclinometer readings were also taken. Intermediate load cell readings were taken on a number of occasions to study the time dependent behavior of the peat.

FINDINGS

DEEP RIVER LOAD TESTS

The two Deep River lateral load tests differed in terms of the geometry of the loading setups and the nature of their surface and subsurface conditions. TPA was located on level ground underlain by the natural silty soils on the east side of the river. After the portion of the pile originally protruding above the ground surface had been cut off, the lateral loading system was modified so that the lateral load was applied essentially at the ground surface. TPB was located at the crest of the irregularly shaped levee on the west side of the river. The weight of the levee soil had consolidated the underlying natural soils to effective stresses greater than those corresponding to the same depths below the natural ground surface at the TPA site. However, since the levee soil immediately adjacent to TPB had been excavated and the load was applied essentially at the level of the top of the levee, both a lateral load and a bending moment were applied to TPB. The effect of the increased strength of the soil under the levee was expected to be offset somewhat by the eccentric application of lateral load to the pile. Therefore, the overall load-deflection behavior of TPB was not expected to be as different from that of TPA as might have been initially assumed. However, interpretation of the results of TPB was more difficult than that of TPA, since no specific strength data were available for the levee soils or the silts that underlie the levees.

Test Pile A

TPA was tested on September 22, 1988, according to the loading schedule shown in Table 1. The loading schedule was originally intended to include two unload-reload loops to determine whether slow cyclic loading would have a significant effect on the pile response. One of these unload-reload loops was expected to be necessary when the hydraulic jack had been fully extended. The original test procedure in this situation was to unload the pile when the hydraulic jack had been fully extended and place spacer

Table 1. Loading Schedule for Test Pile A at Deep River Site

Load Increment Number	Applied Load (kips)
1	0
2	10
3	0
4	10
5	20
6	0
7	20
8	28
9	0
10	30
11	40
12	50
13	25
14	0

Table 2. Loading Schedule for Test Pile B at Deep River Site

Load Increment Number	Applied Load (kips)
1	0
2	5
3	10.5
4	0
5	10.5
6	25
7	40
8	0
9	25
10	52.5
11	0
12	48
13	52
14	36.5
15	0

blocks between the jack and the pile, then to resume loading. However, because of its low lateral stiffness, the timber reaction pile system deflected nearly as much as the test pile in response to the applied loads. Consequently, approximately half of the available travel of the hydraulic jack was consumed by lateral movement of the reaction system. Therefore, unloading and reloading for spacer placement took place more times than had been originally intended. The result was the additional unload-reload loops shown in Table 1.

The load-deflection behavior observed in the test on TPA is shown in Figure 18. The initial response was relatively stiff; however, increasing nonlinearity rapidly became evident at pile head deflections greater than about 1/2 inch. The deflected shapes of TPA at lateral loads of 10, 20, 40, and 50 kips are shown in Figure 19. After TPA had been loaded to the maximum pile head deflection of 6-1/2 inches, at which point the

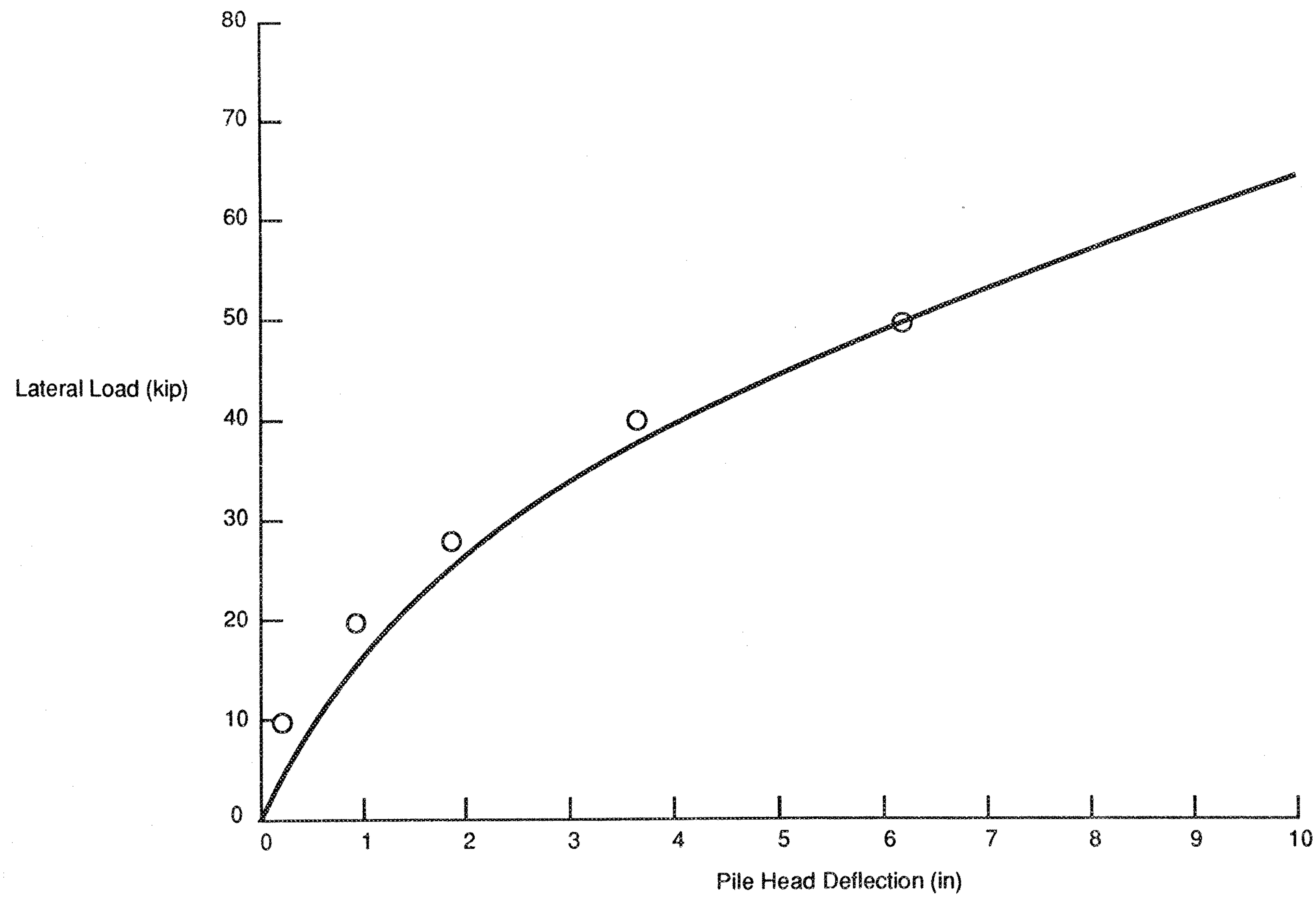

Figure 18. Load-Deflection Behavior for Deep River TPA

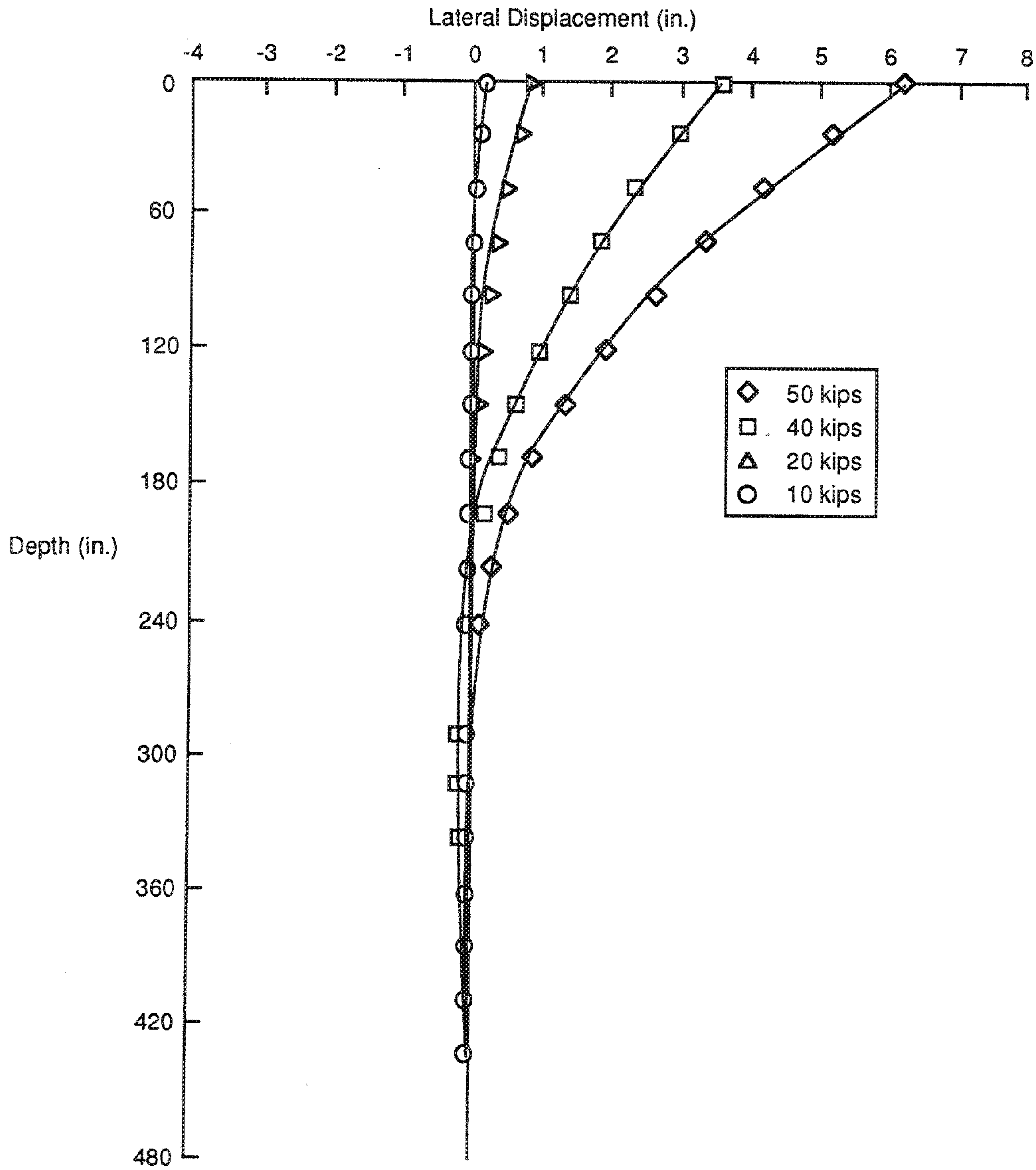

Figure 19. Deflected Shape of TPA at Lateral Loads of 10, 20, 40, and 50 kips

deflections increased so quickly that the hydraulic pump was not able to increase the lateral load any further, the lateral load was reduced to zero.

Test Pile B

TPB was tested on September 23, 1988, under conditions of light rainfall. The original loading schedule for TPB included only one unload-reload loop, which was to occur when the the hydraulic jack's end of travel was reached. However, the low lateral stiffness of the timber reaction pile system at the TPB site again necessitated a greater number of unload-reload loops than originally had been anticipated.

The load-deflection behavior observed in the test on TPB is shown in Figure 20. The response of TPB was very similar to that of TPA, providing evidence of the compensating effects of the higher soil strength under the levee and the eccentricity of load application. The deflected shapes of TPB at lateral loads of 10.5, 25, and 52 kips are shown in Figure 21. After TPB had been loaded to the maximum pile head deflection of 9.5 inches, at which point the deflections increased so rapidly that the hydraulic pump was not able to increase the lateral load any further, the lateral load was reduced to zero.

MERCER SLOUGH LOAD TESTS

Test Pile 1

TP1 was installed on April 3, 1989, and was tested on April 10, 1989. Lateral loads were increased incrementally with two unload-reload loops, and a final unloading measurement was made after the pile had reached its maximum lateral displacement of approximately 8.5 inches. The load-deflection behavior of TP1 is shown in Figure 22, and deflected shapes at lateral loads of 2.2, 4.2, and 6.2 kips are presented in Figure 23.

Test Pile 2

TP2 was installed on April 21, 1989, and tested on April 24, 1989. Lateral loads were increased incrementally with no unload-reload loop in order to simulate the monotonically increasing loads that would be caused by moving peat. The load-deflection response measured at the tops of TP1 and TP2 were similar, as shown without unload-reload loops in Figure 24. The deflected shapes of TP2 at lateral loads of 2.4, 4.8, and 6.9 kips are shown in Figure 25.

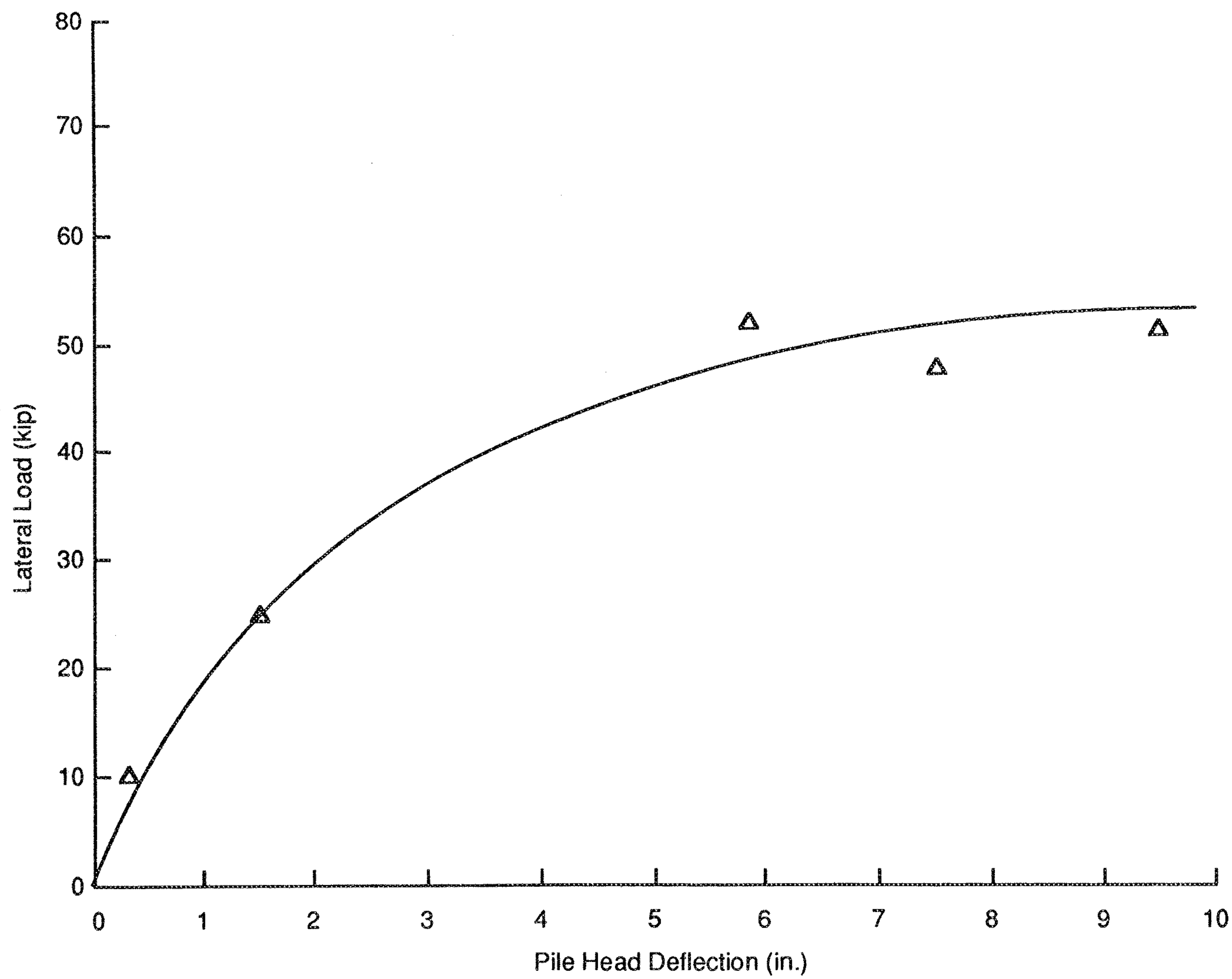

Figure 20. Load Deflection Behavior for Deep River TPB

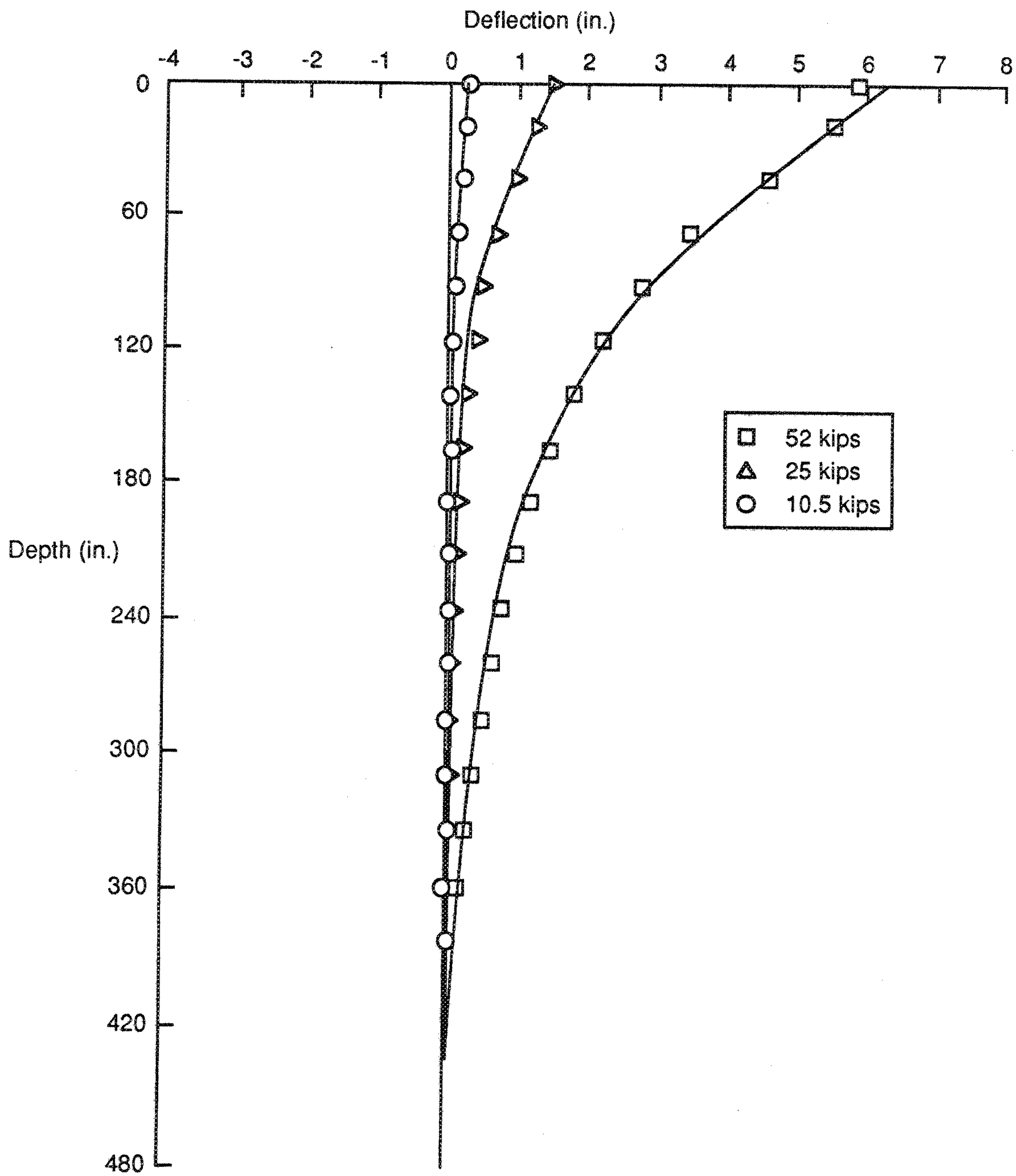

Figure 21. Deflected Shape of TPB at Lateral Loads of 10.5, 25, and 52 kips

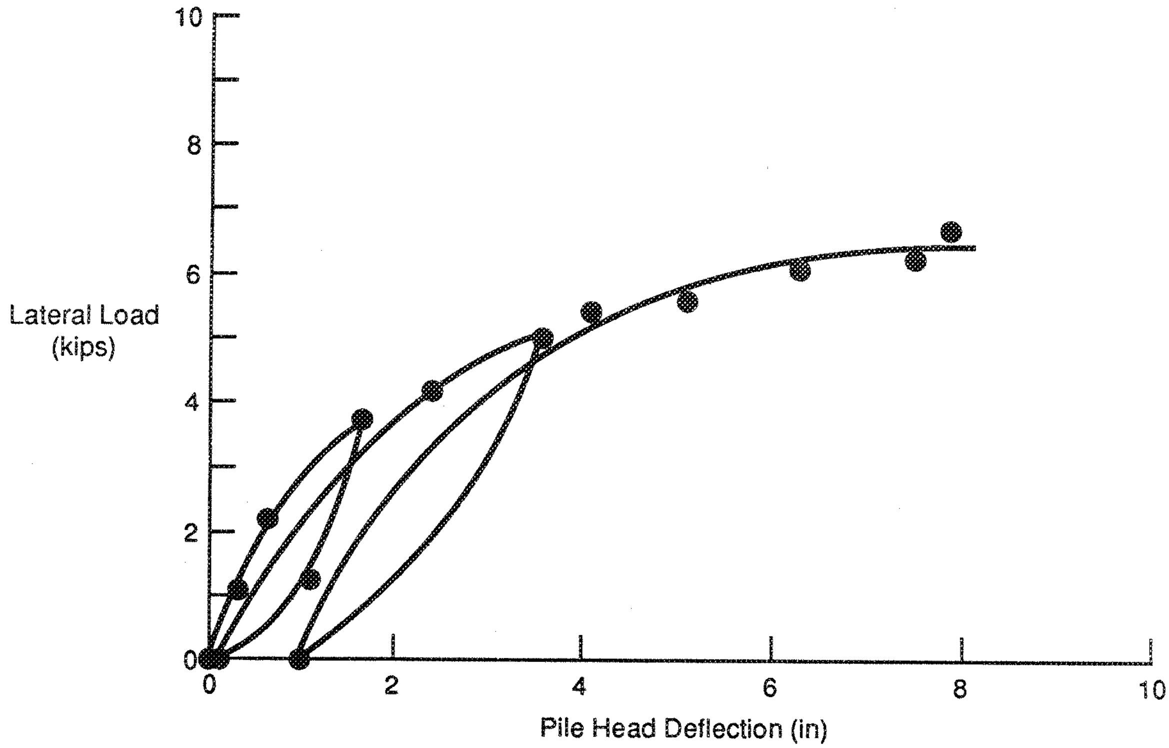

Figure 22. Load-Deflection Behavior for Mercer Slough TP1

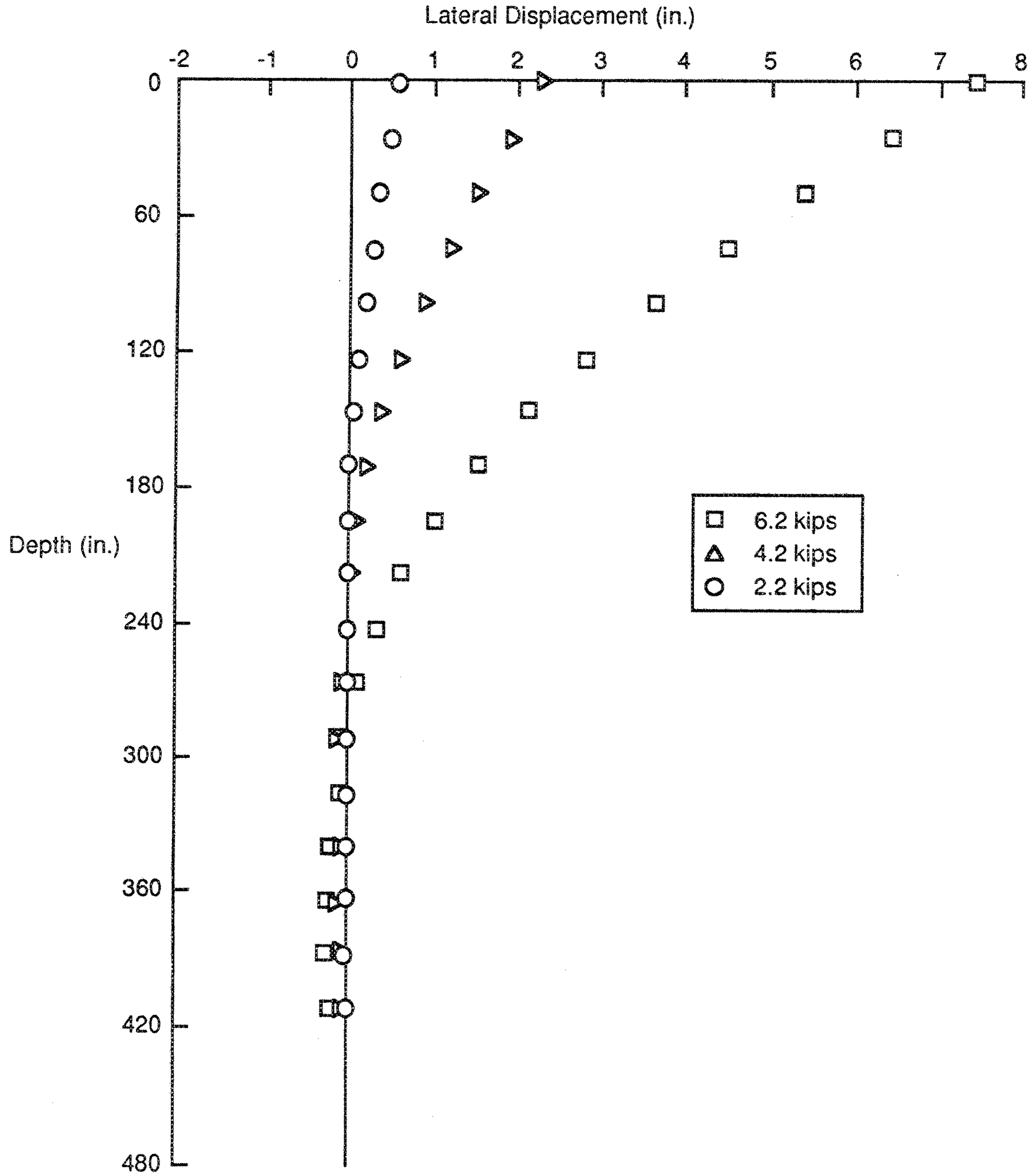

Figure 23. Deflected Shape of TP1 at Lateral Loads of 2.2, 4.2 and 6.2 kips

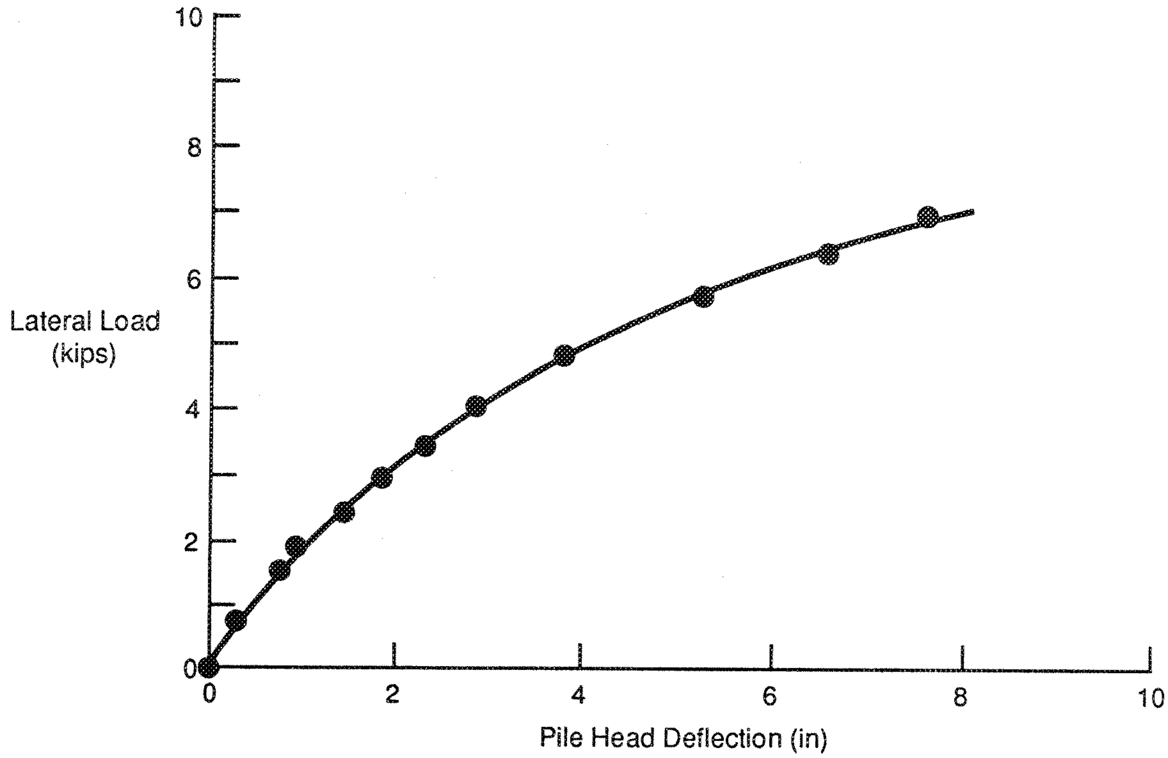

Figure 24. Load-Deflection Behavior for Mercer Slough TP2

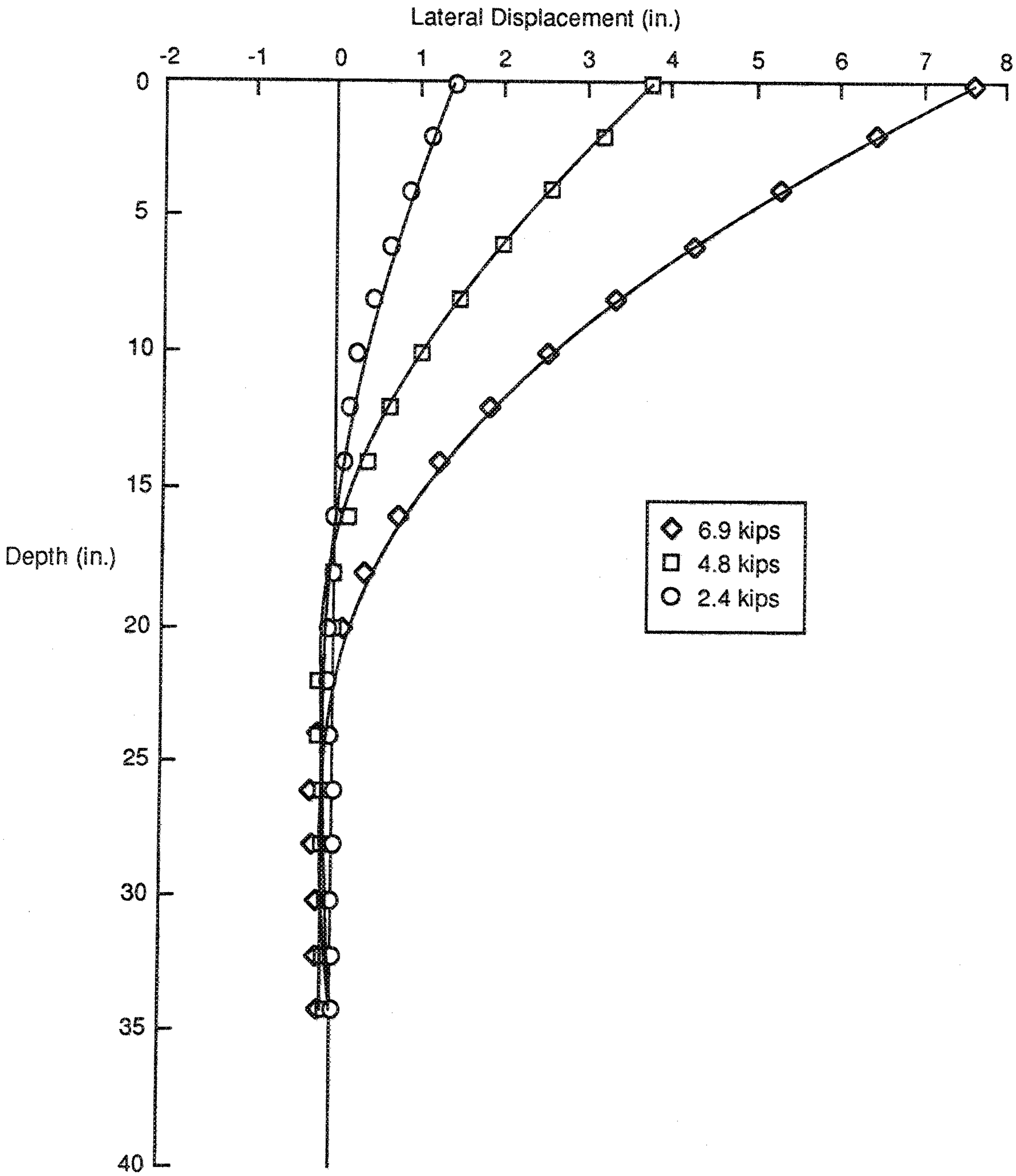

Figure 25. Deflected Shape of TP2 at Lateral Loads of 2.4, 4.8 and 6.9 kips

INTERPRETATION, APPRAISAL AND APPLICATION

INTERPRETATION OF LATERAL LOAD TESTS

The results of pile lateral load tests may be interpreted with a number of techniques that range from simple to complex. In many pile lateral load tests, measurements of pile response have been made only at the ground surface, e.g., load-deflection measurements. While such tests are relatively easy and inexpensive to instrument and conduct, the results are impossible to interpret in a fundamental way. The results of tests in which only load-deflection measurements are made may be quite useful as an indication of the lateral load response that can be anticipated at a particular site; however, they cannot be easily applied to other sites. For proper interpretation of p-y behavior, it is necessary to measure the deformed shape of the pile in response to the applied loading. In pile load tests, these deformations are usually measured in terms of pile curvatures and/or pile slopes. Under ideal conditions, the p-y behavior can be obtained directly from the instrumentation results. In practice, however, it is very difficult to accurately obtain p-y behavior directly from the instrumentation data.

p-y CURVE CRITERIA

A number of methods have been proposed for the development of p-y curves that relate unit soil resistance to lateral pile deflection. Because of the complexity of the manner in which unit soil resistance is mobilized, its characteristics have generally been determined semi-empirically from the the results of full-scale and model pile load tests. The first p-y curve criteria for cohesive soils, the Soft Clay Criterion of Matlock (4) and the Stiff Clay Criterion of Reese and Welch (5), were developed in response to the needs of the offshore industry in the 1970s. These p-y curve criteria, which were based on the results of a very small number of pile load tests, required an *a priori* decision about whether the soil was "soft" or "stiff." Because these p-y curve criteria were developed before a significant number of data regarding the full-scale lateral load response of piles were available, they include a number of empirically-based elements that limit the breadth of their general applicability to a wide range of cohesive soils.

To correlate the results of this investigation's lateral load tests with the known behavior of laterally loaded piles in other types of soils, test results had to be interpreted within the framework of an existing p-y curve development procedure. The use of this approach required the availability of both an analytical model capable of describing the behavior of laterally loaded piles and p-y criteria capable of describing the general behavior of soils similar to the soils at the two test sites. The analytical model selected for test interpretation was the widely used Winkler model, which has been incorporated in the computer program COM624 currently used by WSDOT engineers. The p-y criteria selected for both the Deep River silts and the Mercer Slough peats was the Integrated Clay Criterion of O'Neill and Gazioglu. (6) The Integrated Clay Criterion is similar in some respects to the Soft Clay Criterion; however, it has been shown to represent the influence of pile diameter more accurately and to be accurate over a wider range of soil conditions than the Soft Clay Criterion. (6, 7) Because only two tests were performed at each site, these features of the Integrated Clay Criterion were considered extremely important to the interpretation of the test results.

INTEGRATED CLAY CRITERION

In an attempt to develop a p-y criterion for cohesive soils that was more broadly based on high quality field load test data and to remove the subjective distinction associated with characterization of such soils as either soft or stiff, O'Neill and Gazioglu proposed the Integrated Clay Criterion. (6) This criterion would be applicable to all cohesive soils regardless of stiffness. The Integrated Clay Criterion was based on the results of 21 full-scale, field lateral load tests on piles installed at 11 locations. Soil conditions ranged from very soft to very stiff. They developed it by making a number of reasonable assumptions regarding the influence of factors such as pile diameter, pile length, and soil stiffness and by optimizing several parameters to produce a procedure that provided the best agreement with the available data.

The Integrated Clay Criterion specifies a p-y relationship of the form

$$p = \frac{1}{2} p_{ult} \left(\frac{y}{y_c}\right)^{0.387} \leq p_{ult} \qquad \text{Equation 1}$$

where

$$y_c = 0.8 \sqrt{D}\, \varepsilon_c \left(\frac{EI}{E_s}\right)^{0.125}$$

$$p_{ult} = F N_p\, c\, D$$

$$N_p = \begin{cases} 3 + 6\,(x/x_{cr}) & \text{for } x \leq x_{cr} \\ 9 & \text{for } x > x_{cr} \end{cases}$$

$$x_{cr} = 0.25\, L_c$$

$$L_c = 3 \left(\frac{EI}{E_s \sqrt{D}}\right)^{0.286}$$

E_s = secant soil stiffness

ε_c = critical strain (at one-half $(\sigma_d)_{max}$ in UU triaxial test)

EI = flexural stiffness of pile

F = soil degradability factor

c = cohesive strength of soil

D = pile diameter

In this formulation, the unknown quantities are the soil degradability factor, F, the cohesive strength of the soil, c, the secant soil stiffness, E_s, and the critical strain, ε_c. For most cohesive soils, the ratio of secant soil stiffness to cohesive strength is constant; hence, consideration of the ratio E_s/c as an unknown is often more useful, particularly for soil conditions in which the cohesive strength varies with depth. For ductile materials, the soil degradability factor may be assumed to be unity.

INTERPRETATION OF DEEP RIVER LOAD TEST RESULTS

Interpretation of the results of the lateral load tests within the framework of an existing p-y criteria required evaluation of the cohesive strength of the soil. The rapid loading employed in the tests in the fine-grained silts at the Deep River site was unlikely to have allowed pore pressure dissipation during loading. Hence it was appropriate to base the interpretation of these tests on the undrained strength of the soil at the locations of the two tests.

Evaluation of Undrained Strength

A limited amount of information on the shear strength of the Deep River silts at shallow depths was available from the results of the previous subsurface investigations. These strength data, which were based on a small number of vane shear and triaxial compression tests, are presented in Figure 26. The CPT soundings performed in the recent supplementary subsurface investigation provided invaluable supporting strength data for the Deep River silts, without which interpretation of the results of the Deep River tests would have been very difficult.

The undrained strength of a soil deposit may be interpreted from the results of CPT tests in different ways. The undrained strength, s_u, generally obtained from some form of the deep bearing capacity equation, can be expressed as

$$s_u = \frac{q_c - \sigma_{vo}}{N_k} \qquad \text{Equation 2}$$

where q_c = cone resistance,

σ_{vo} = total vertical overburden stress, and

N_k = cone factor.

Historically, the cone factor has usually been obtained from correlations between cone resistance and the undrained strength obtained from vane shear tests. The use of vane shear tests, which require correction for anisotropy and strain rate effects, has led to considerable uncertainty in the cone factor, N_k, or the corrected cone factor, N_k^*,

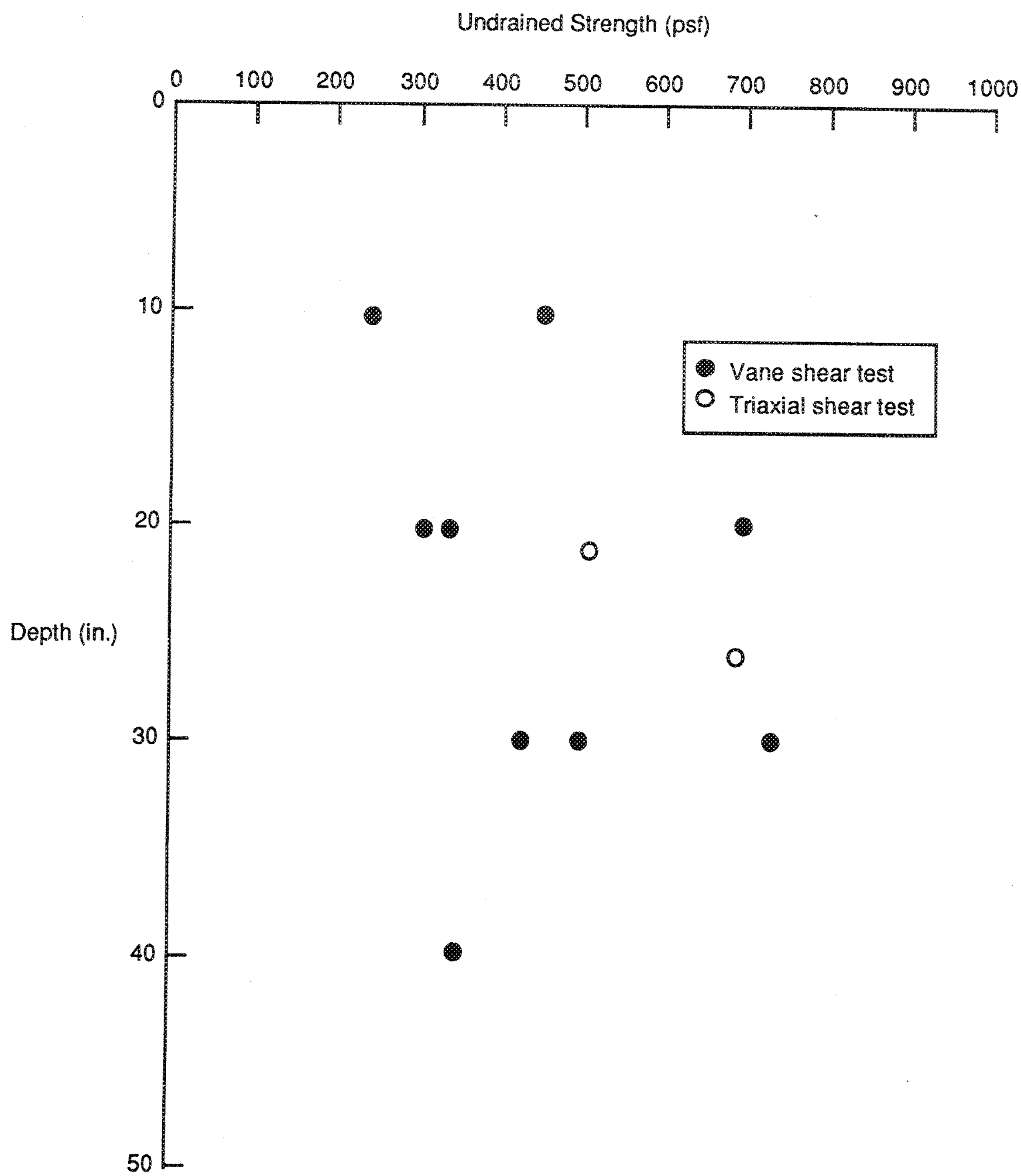

Figure 26. Deep River Strength Data (Vane and Triaxial)

which includes the vane shear strength correction of Bjerrum. (8) Interpreted with respect to vane shear strengths, the cone factor varies in a highly scattered manner with the plasticity index, as shown in Figure 27. Recently, research has been performed to relate cone resistances to undrained strengths obtained from UU triaxial tests. (9) In this framework, the UU cone factor, N_{kuu}, as shown in Figure 28, shows much less scatter and much lower sensitivity to the plasticity index than the vane shear cone factor. For most practical purposes, N_{kuu} can be assumed to equal 12.

Undrained Strength at TPA Site

The undrained strength of the Deep River silts surrounding TPA was evaluated from the results of CPT1 and CPT2, which were located near the TPA site. Interpreted on the basis of the equation

$$s_u = \frac{q_c - \sigma_{vo}}{N_{kuu}} \qquad \text{Equation 3}$$

after Stark and Delashaw (9), the undrained strength profile near TPA was as shown in Figure 29. The CPT results indicated the presence of a medium stiff, dessicated crust approximately 3 feet thick underlain by very soft silt. Below the crust, the undrained strength increased approximately linearly with depth. These results were consistent with the characteristics of the Deep River silts observed in the field, though the surface crust near TPA appeared to have been disturbed during pile driving and other construction activities. A linear undrained strength profile described quite well the characteristics of the Deep River silt at TPA for depths below the crust, i.e., between about 3 to 4 feet and 35 to 40 feet. In this region, the average undrained strength (in psf) was described reasonably well by the equation

$$s_u = 250 + 28.8x \qquad \text{Equation 4}$$

where x = depth in feet. This linear undrained strength profile is also shown in Figure 29.

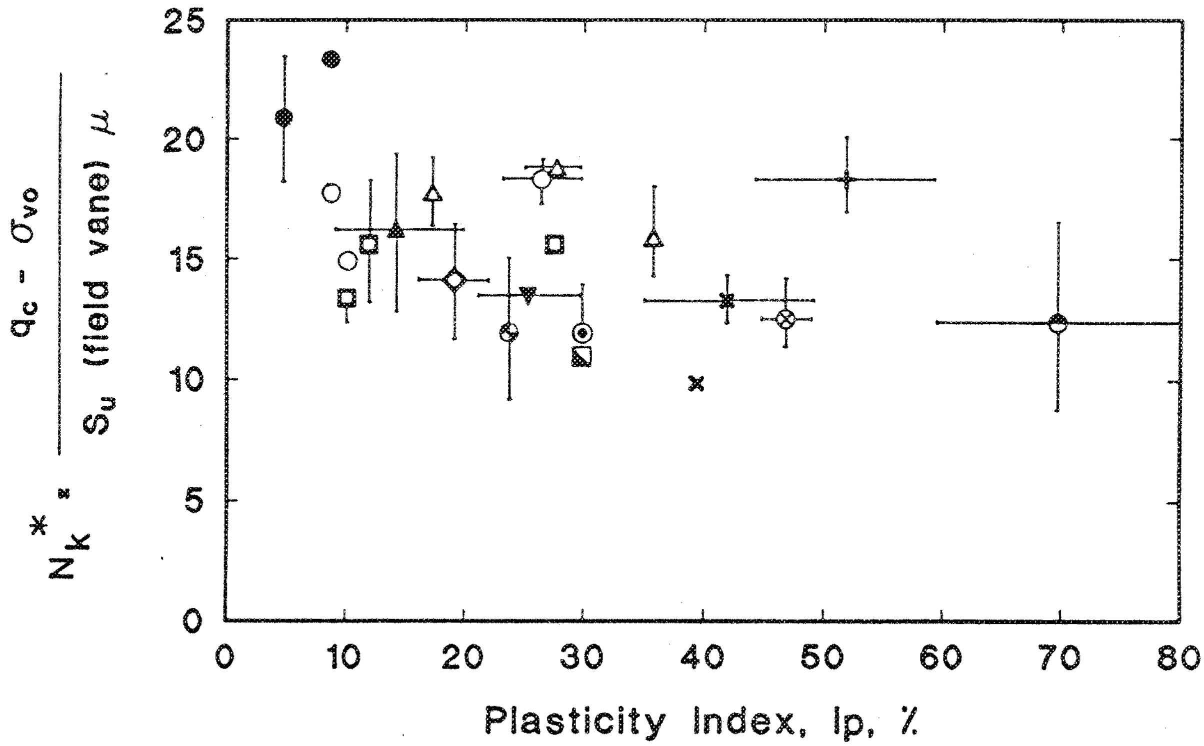

Figure 27. Cone Factor Correlation with Vane Shear Strength (after Stark and Delashaw, 1990)

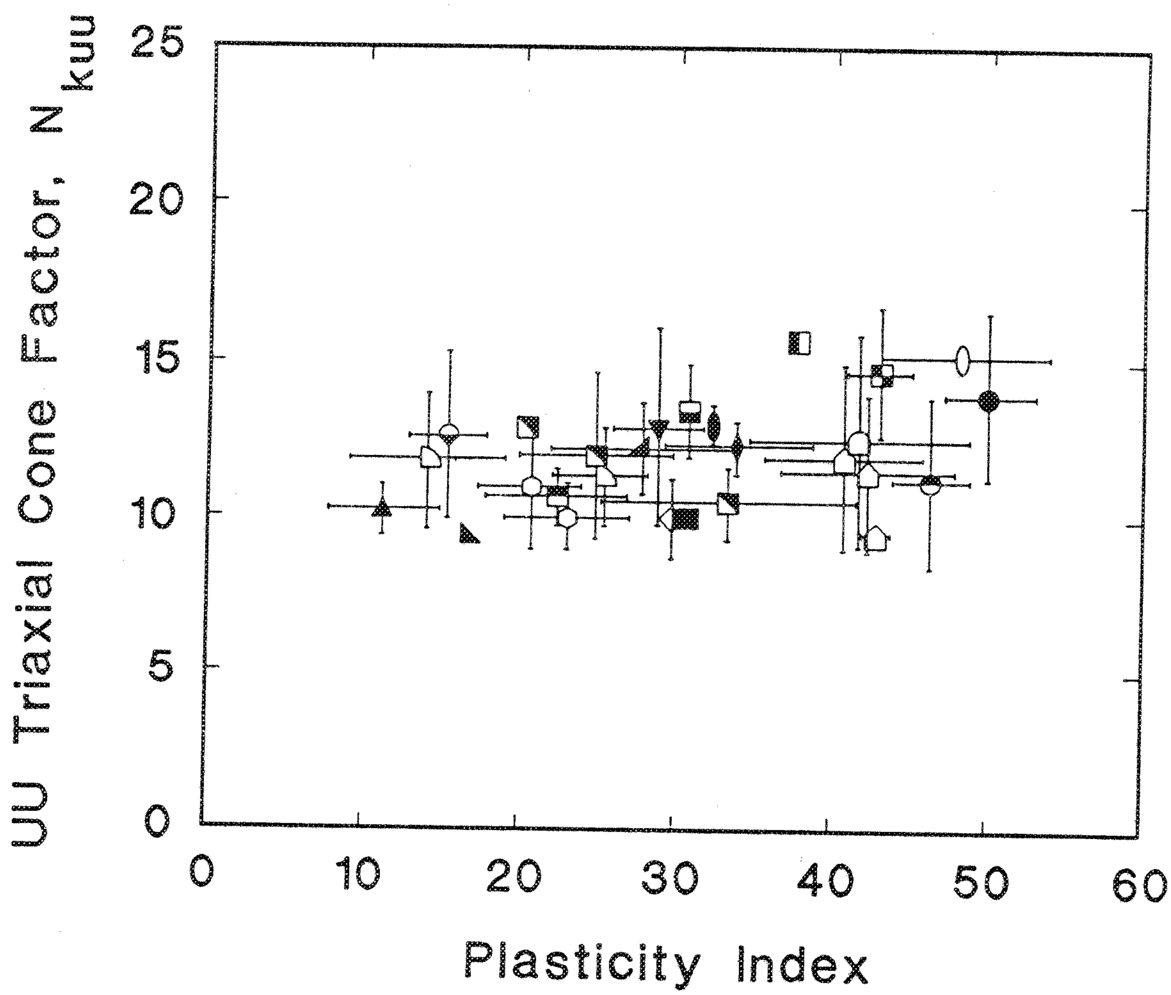

Figure 28. Cone Factor Correlation with UU Triaxial Shear Strength (after Stark and Delashaw, 1990)

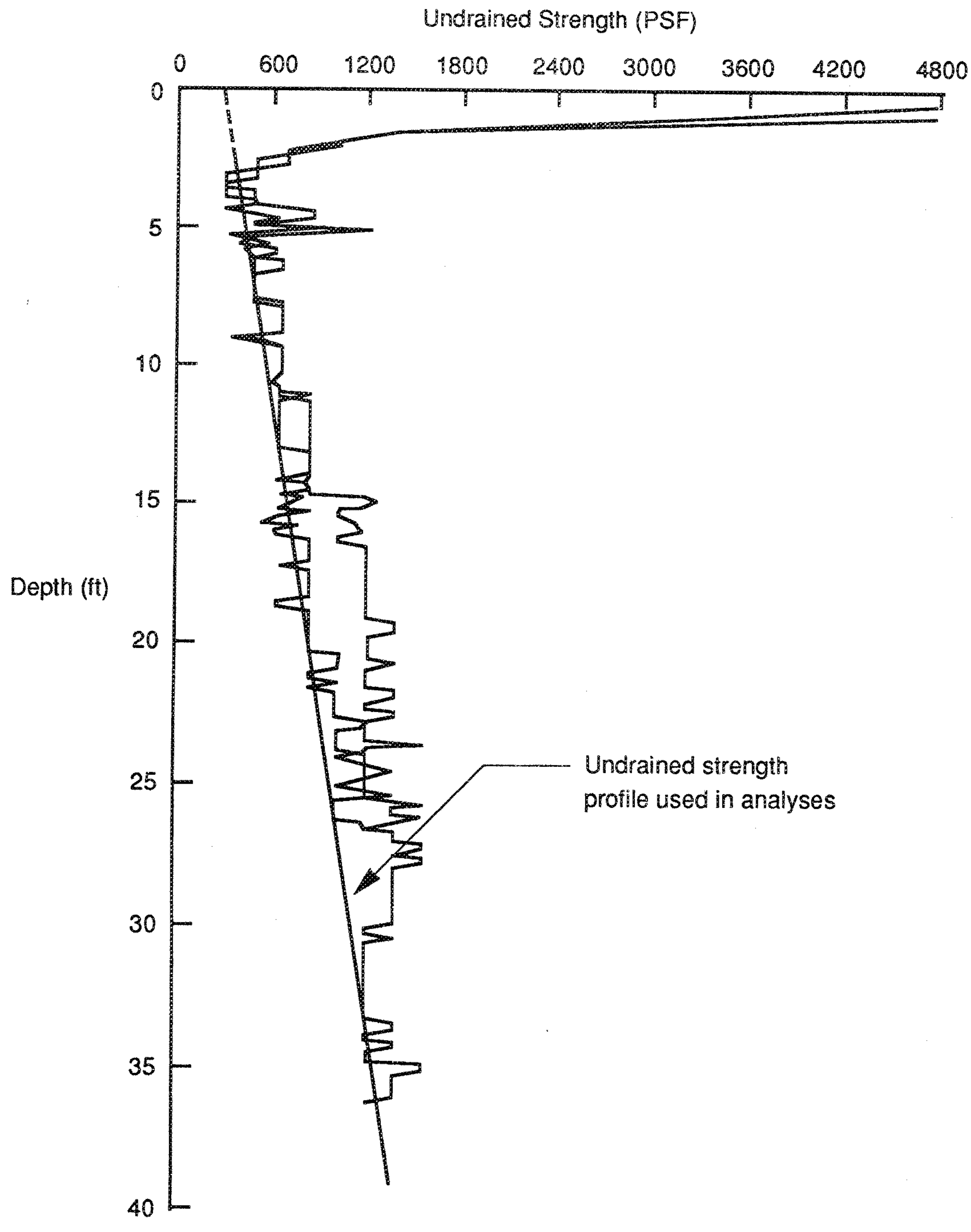

Figure 29. Undrained Strength Profile at TPA Site

Undrained Strength at TPB Site

Interpretation of the undrained strength of the Deep River silts at the TPB site was complicated by the unusual site conditions for TPB. TPB was installed in the levee on the west side of the river. Reliable comparison of the results of TPA and TPB required that the effects of the levee on the underlying soil and the excavation of the levee soil immediately adjacent to the pile be accounted for in a reasonable way. The presence of the levee caused the underlying silty soil to be consolidated to stresses greater than those which would have been present in the absence of the levee. Therefore, the silty soils near the original ground surface surrounding TPB were expected to be stiffer and stronger than the soils at the same elevation surrounding TPA. As an illustration of this effect, the results of CPT3, which was adjacent to the levee in which TPB was installed, indicated undrained strengths greater than those near TPA at depths between about 6 and 18 feet.

The undrained shear strength of normally consolidated cohesive soils is proportional to the major principal effective stress acting on the soil. The average buoyant unit weight of the Deep River silts was approximately 30 pcf, indicating that the vertical effective stress increased at an average rate of 30 psf per foot of depth. The linear function describing the undrained strength profile indicated that the undrained strength increased by an average of 28.8 psf per foot of depth. On this basis, the average incremental normalized strength ratio was

$$\frac{\Delta s_u}{\Delta \sigma'_v} = \frac{28.8}{30} = 0.96 \qquad \text{Equation 5}$$

While this ratio was considerably larger than the ratio that would have been expected for a clay of the same plasticity index (8, 10), it was based on reliable, site-specific measurements of the Deep River silts and was used for interpretation of the TPB results. The undrained strength profile of the Deep River silt at TPB differed from that at TPA because of the additional vertical stress imposed by the weight of the levee soil. Thus, the average TPB undrained strength profile was calculated as the sum of the average TPA undrained strength profile and an undrained strength increment that depended on the incremental vertical effective stress imposed on the underlying soils by the weight of the levee. At the TPB site, the distribution of vertical effective stress increment with depth was evaluated with elastic stress distribution methods. At each depth, the vertical stress increment was used to calculate an undrained strength increment, which was added to the average TPA undrained strength profile. The resulting undrained strength profile used to interpret the TPB test is shown in Figure 30.

Ultimate Unit Soil Resistance

The ultimate unit soil resistance for TPA was calculated in the conventional manner with the Integrated Clay Criterion. Since the Integrated Clay Criterion assumes flat ground conditions, procedures had to be developed to account for the irregular surface around TPB. For the relatively rapid loading applied during the tests, a wedge-type failure at depths less than the critical depth was assumed to occur on a plane inclined at 45 degrees from vertical. An equivalent depth for ultimate soil resistance computation was determined to be the depth at which the soil within a 45-degree wedge in flat ground conditions would have had the same weight as the actual weight of soil within the 45-degree wedge, considering the actual geometry of the TPB test site shown previously in Figure 9. The value of ultimate soil resistance calculated by the Integrated Clay Criterion at this equivalent depth was then applied at the actual depth.

Evaluation of p-y Curve Parameters

In the Integrated Clay Criterion formulation, the unknown parameters are the soil degradability factor, F, the cohesive strength of the soil, c, the secant soil stiffness, E_s, and the critical strain ε_c. For the Deep River tests, the cohesive strength corresponded to the undrained strength profiles shown in Figures 29 and 30. Since the undrained strength of the Deep River silts varied with depth, the secant soil stiffness was also expected to vary with depth. However, the secant soil stiffness could be reasonably assumed to be proportional to the undrained strength of the soil. Because the cohesive strength was

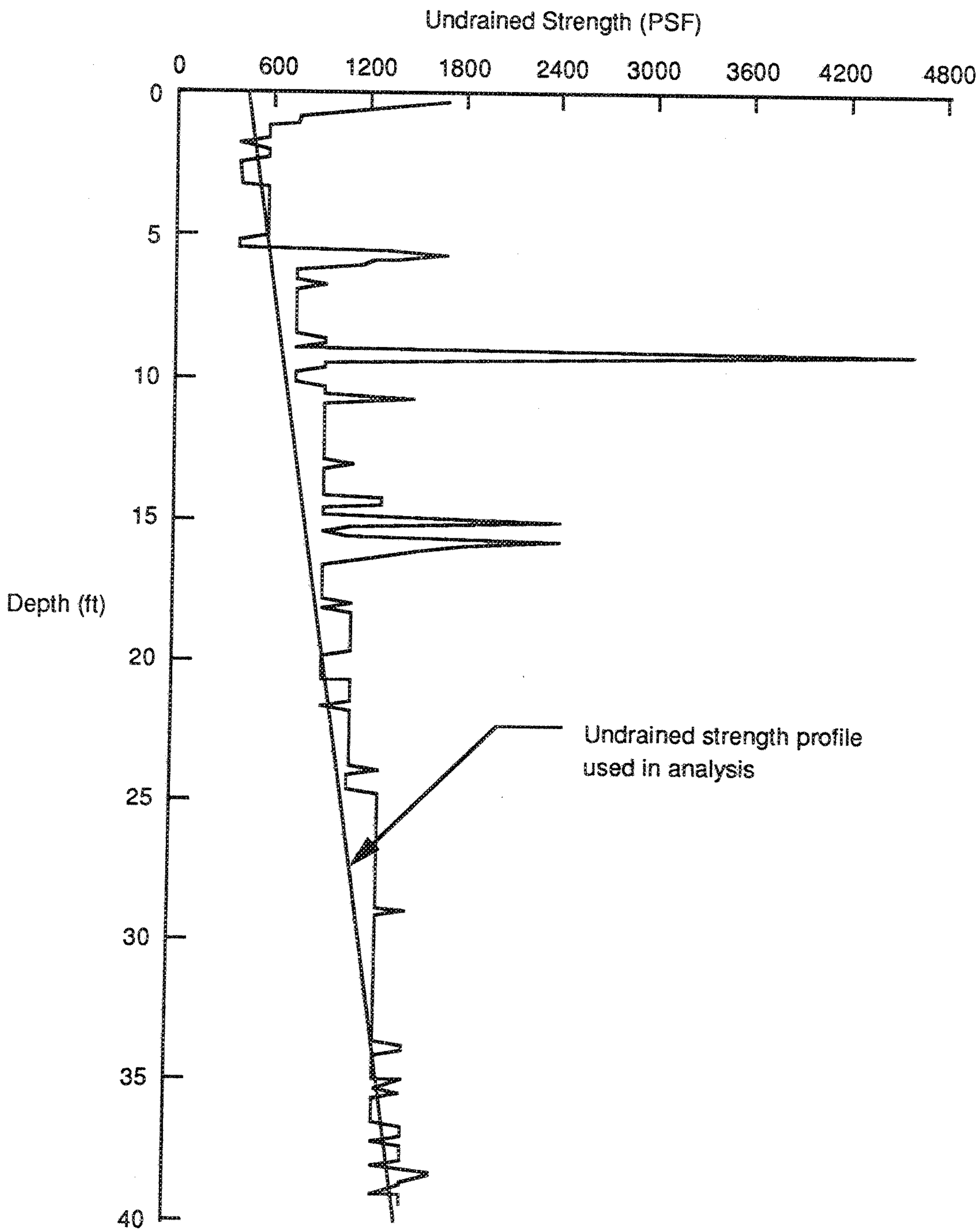

Figure 30. Undrained Strength Function for TPB Site

known and secant soil stiffness was assumed to be proportional to cohesive strength, the unknown parameters were F, E_s/c, and ε_c. These remaining three unknowns were varied in a direct search optimization procedure to find the combination of soil properties that provided the best fit with the observed results. The optimization procedure sought to minimize the weighted error between measured and predicted pile displacements along the length of the pile. To account for the fact that greater strains are induced in the soil near the ground surface, the error function gave more weight to displacement errors near the ground surface than at greater depths. For each load, the weighted error was calculated as

$$\text{Weighted Error} = \sum_{i=1}^{n} \frac{L+1}{x_i+1} \left| y_{meas} - y_{pred} \right|_i \qquad \text{Equation 6}$$

where n = number of points at which measured and predicted displacements were compared,

L = length over which measured and predicted displacements were compared, and

y_{meas} and y_{pred} = the measured and predicted displacements, respectively, at depth x_i.

To obtain parameters that predicted the observed behavior at low loads and deflections and at high loads and deflections, the total weighted error was calculated as the sum of the individual weighted errors at low, medium, and high lateral loads for each test pile. The properties inferred by this procedure were as follows:

$E_s/c = 50$

$\varepsilon_c = 0.7\%$

$F = 1.0$

Integrated Clay Criterion p-y curves developed from these values predicted pile head load-displacement behavior that agreed well with the observed behavior, as shown in Figure 31 for TPA and Figure 32 for TPB. The relatively poor agreement between the

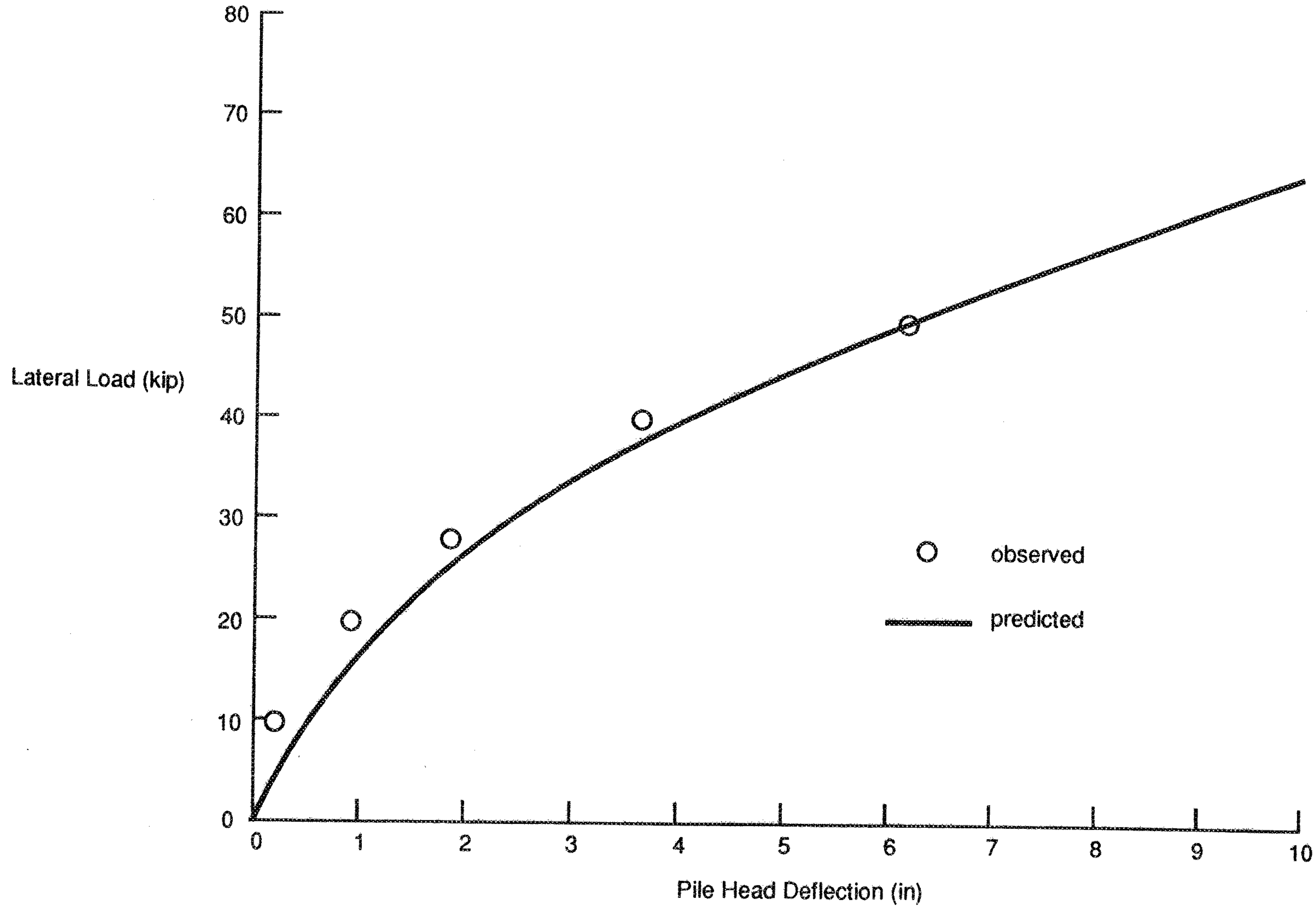

Figure 31. Predicted and Observed Load-Deflection
Behavior for TPA

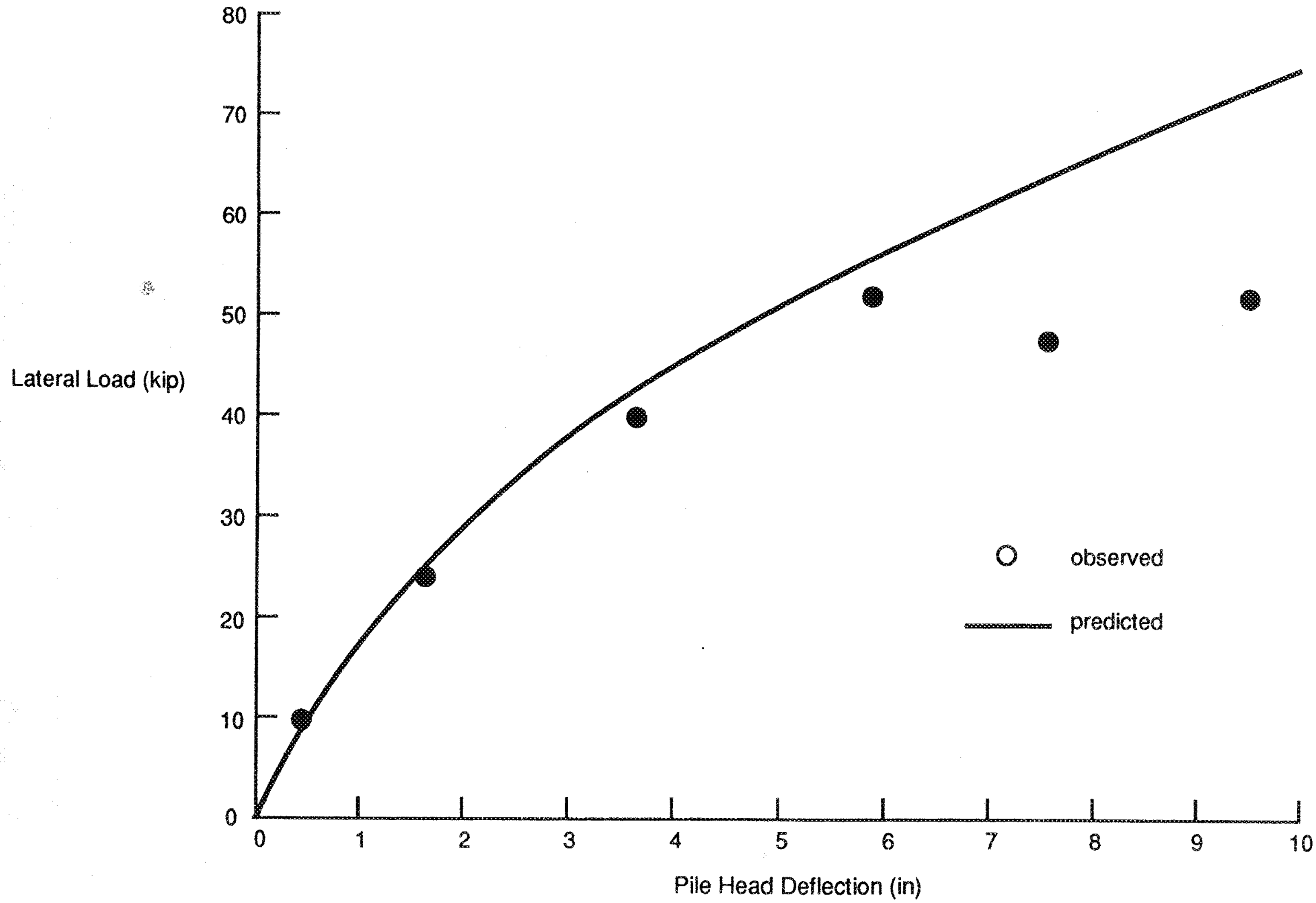

Figure 32. Predicted and Observed Load-Deflection Behavior for TPB

observed and predicted results for TPB at deflections greater than 7 inches are attributed to softening of the soil during unload-reload loops caused by low reaction system stiffness. The predicted deflected shapes of the pile also agreed reasonably well with the observed deflected shapes at low, medium, and high lateral loads,as seen in Figure 33 for TPA and Figure 34 for TPB. As expected, the agreement with respect to deflected shape was better at shallower depths where pile deflections and soil strains were largest.

INTERPRETATION OF MERCER SLOUGH LOAD TEST RESULTS

Reduction of the raw test data allowed development of p-y curve data for the peats of Mercer Slough. For this purpose, the data from both test piles were combined. This combination was possible because no significant difference in the characteristics of the peats at the two test sites was apparent from the available information and because no differences existed in loading system geometry like they had in the Deep River tests. However, because a more accurate load cell was used for TP2 and because a set of TP2 readings were made consistently 15 minutes after each load application, the TP2 data were weighted more heavily in the interpretation of results.

Evaluation of Cohesive Strength

Measurement of the shear strength of peats has posed a difficult problem to geotechnical engineers for many years. Many peats exhibit a component of shear strength that results from the extension and interlocking of fibrous, organic material within the peat. (11, 12) Conventional laboratory strength tests on relatively small-scale samples often do not reflect this fibrous component of shear strength. Most conventional in-situ strength tests also mobilize shear strength on a relatively small surface and do not capture the fibrous component of shear strength. In a comprehensive review of in-situ testing of peats, Landva stated that "cone penetration and vane testing . . . do not give meaningful results in peats and peaty organic soils," but that such results "can be obtained through large-scale or full-scale testing." (12) In this investigation, the laterally loaded

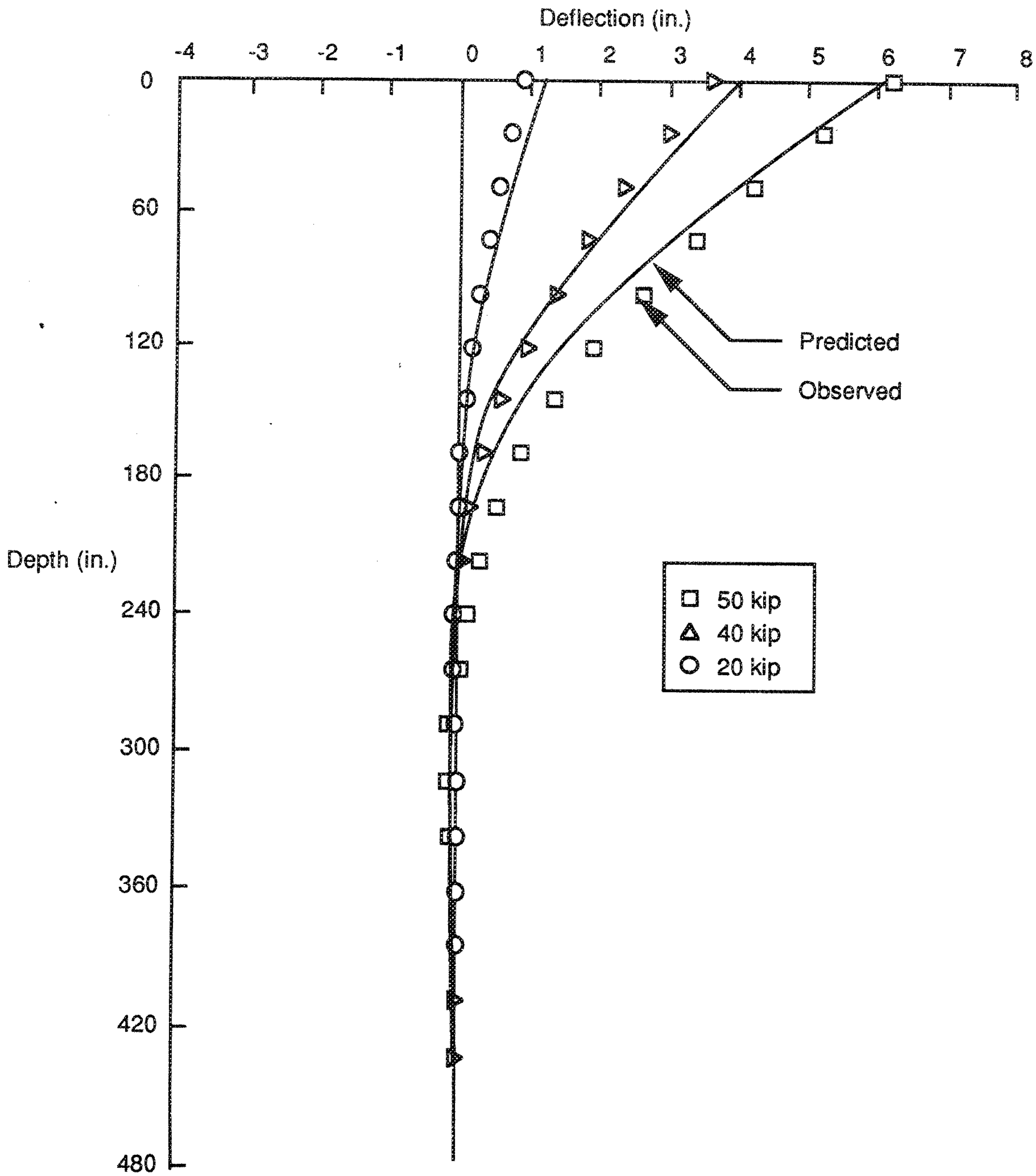

Figure 33. Predicted and Observed Deflected Shapes for TPA

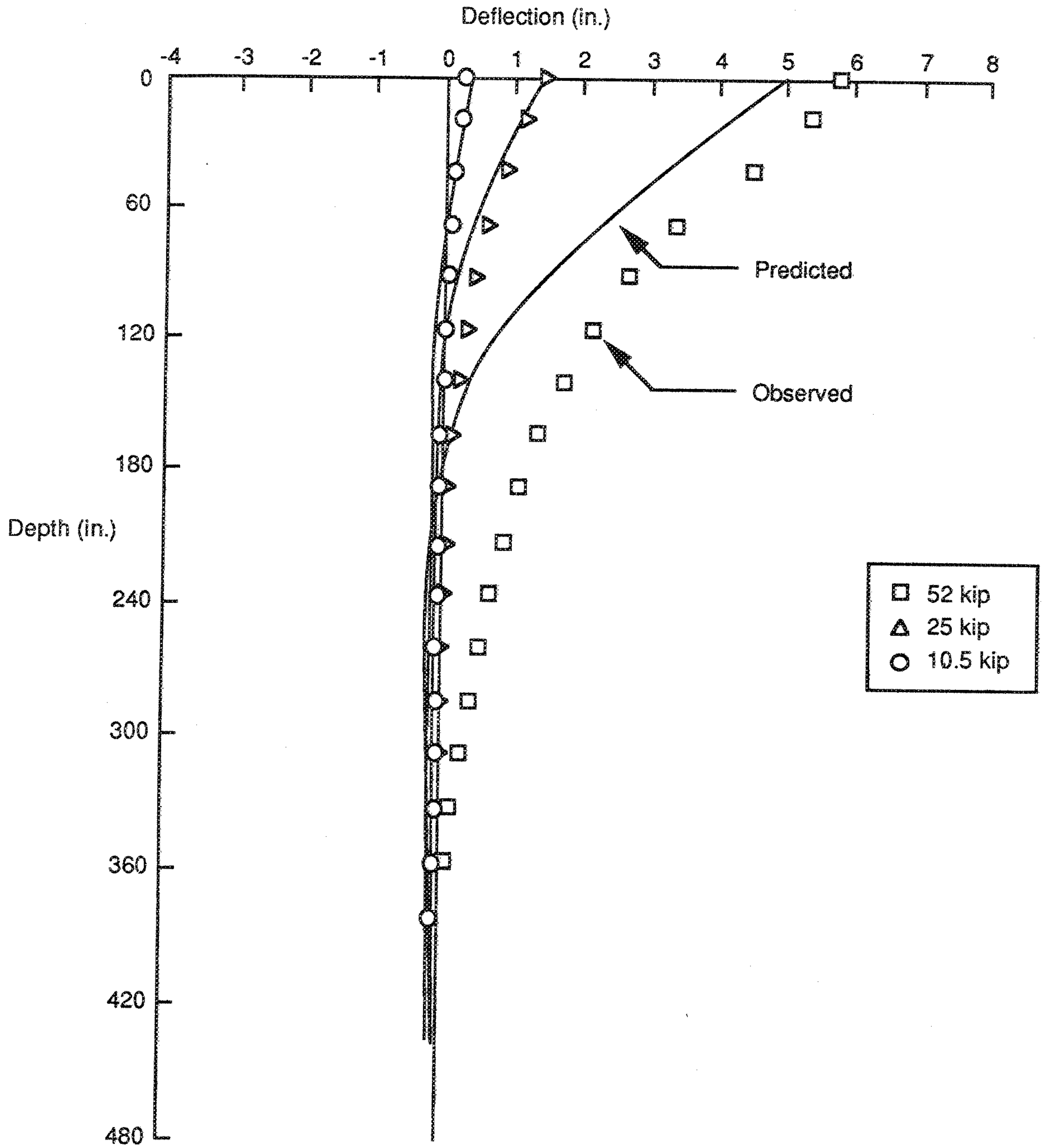

Figure 34. Predicted and Observed Deflected Shapes for TPB

pile test was interpreted as an in-situ test to evaluate the strength of the Mercer Slough peat. The resistance of a peat to lateral displacement of a pile requires mobilization of shear strength in a relatively large volume of soil surrounding the pile. This volume of peat is large enough that the fibrous component of shear strength is reflected in the soil resistance, from which an accurate estimate of the in-situ shear strength can be made.

Previously performed subsurface investigations near the Mercer Slough pile load test sites had consisted of conventional boring and sampling, along with vane shear and cone penetration test profiling. Unconsolidated-undrained triaxial and vane shear tests had indicated peak undrained shear strength values ranging from as low as 15 psf to 175 psf, with a possible trend of modestly increasing strength with depth, as shown in Figure 36. This large range of variability was consistent with that observed in other studies of vane shear tests in peat. (12) Two piezocone penetration test profiles indicated a uniform tip resistance of approximately 340 psf with a friction ratio of 1.5 percent to 6 percent. Measured pore pressures were essentially hydrostatic. The cone penetration logs are shown in Figure 36. However, because of lateral deflection of typical size penetrometers and the mode of deformation and failure, researchers have recognized that the interpretation of cone penetration tests in peat is very difficult. (12)

Evaluation of p-y Curve Parameters

In the Integrated Clay Criterion formulation, the unknown quantities are the soil degradability factor, F, the cohesive strength of the soil, c, the secant soil stiffness, E_S, and the critical strain, ε_C. For the ductile peat material, the soil degradability factor was assumed to be equal to 1. (6) The remaining three unknowns were varied in a direct search optimization procedure to find the combination of soil properties that provided the best fit with the observed results. The soil properties were assumed to be constant throughout the peat, since no data provided sufficient reason to assume otherwise. The optimization procedure sought to minimize the weighted error between measured and predicted pile displacements along the length of the pile. To account for the fact that

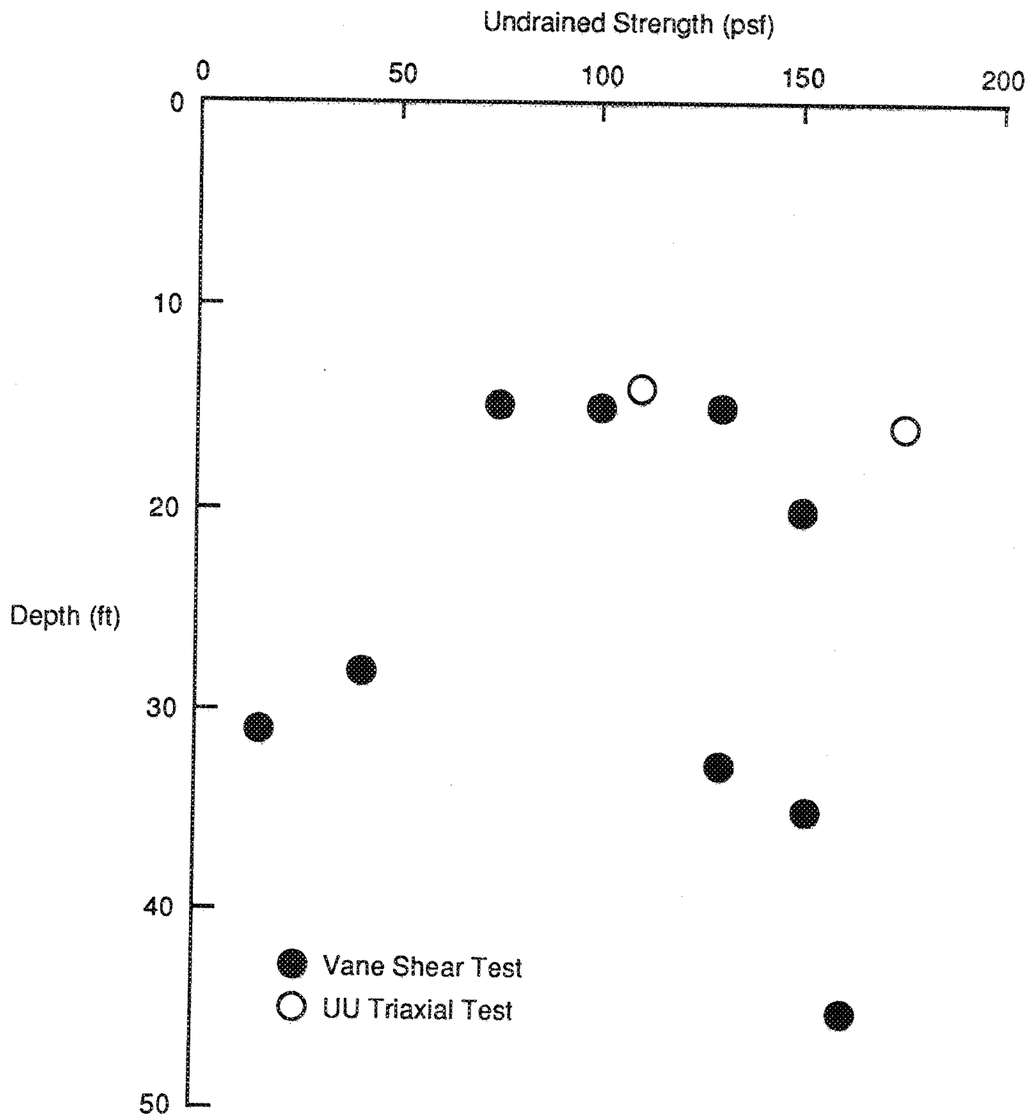

Figure 35. Mercer Slough Strength Data (Vane and Triaxial)

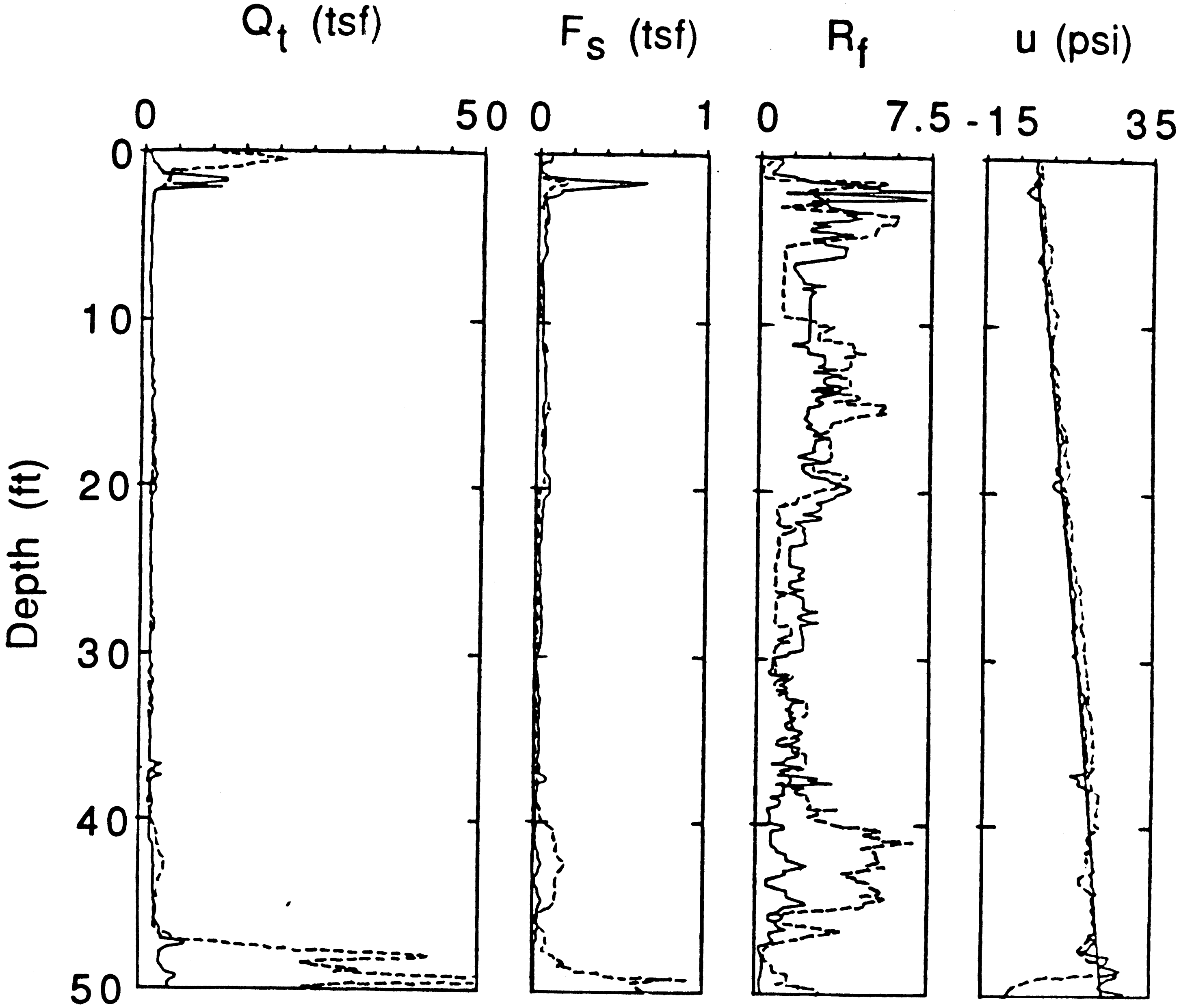

Figure 36. Mercer Slough CPT Soundings

greater strains are induced in the soil near the ground surface, the error function was based on weighted deviations in the same manner as described for the Deep River Tests. To obtain parameters that predicted the observed behavior at low loads and deflections and at high loads and deflections, the total weighted error was calculated as the sum of the individual weighted errors at lateral loads of 2.4, 4.8, and 6.9 kips. The properties inferred by this procedure were as follows:

$c = 200$ psf

$E_s = 1{,}000$ psf

$\varepsilon_c = 3.6\%$.

Integrated Clay Criterion p-y curves developed from these values predicted pile head load-displacement behavior that agreed well with the observed behavior of TP2, as shown in Figure 37. The predicted deflected shapes of the pile also agreed reasonably well with the observed deflected shapes, as seen in Figure 38. As expected, the agreement was better at shallower depths where pile deflections and soil strains were largest.

For a model to be useful in the prediction of a particular parameter value, the prediction error should be sensitive to values of that parameter. A sensitivity analysis indicated that the total weighted error was sensitive to the cohesive strength, c, but not to the critical strain, ε_c, or to the secant soil stiffness, E_s. This sensitivity is illustrated in Figure 39, in which the shaded bands along each abcissa represents ±10 percent deviation from the inferred parameter value. The sensitivity should be well noted; it emphasizes the necessity of obtaining complete and reliable strength data for prediction of lateral load response.

The inferred cohesive strength of 200 psf was somewhat higher than the strengths obtained from the field and laboratory tests that had been previously performed at the site but was within the range of shear strengths reported for other peats. The difference is likely attributable to the fibrous component of strength in the peat, which is lost during sampling and not mobilized during relatively small-scale vane shear and cone penetration

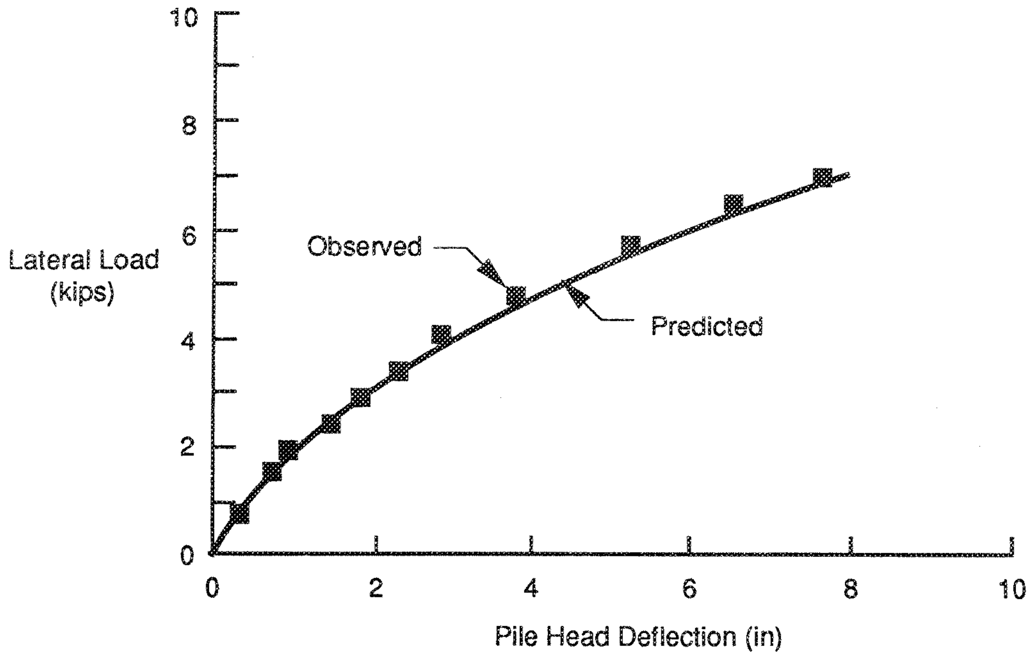

Figure 37. Predicted and Observed Load-Deflection Behavior for TP2

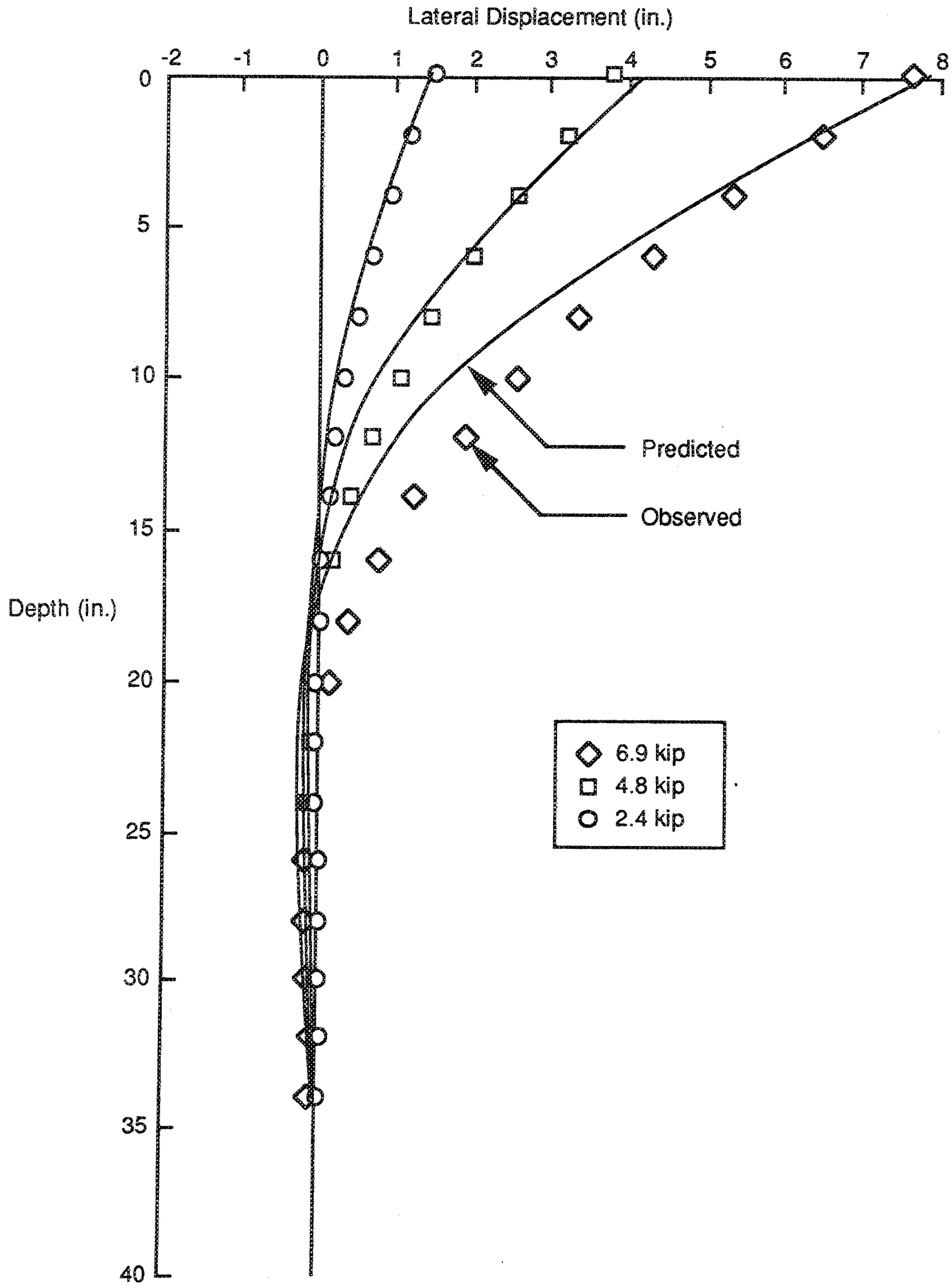

Figure 38. Predicted and Observed Deflected Shapes for TP2

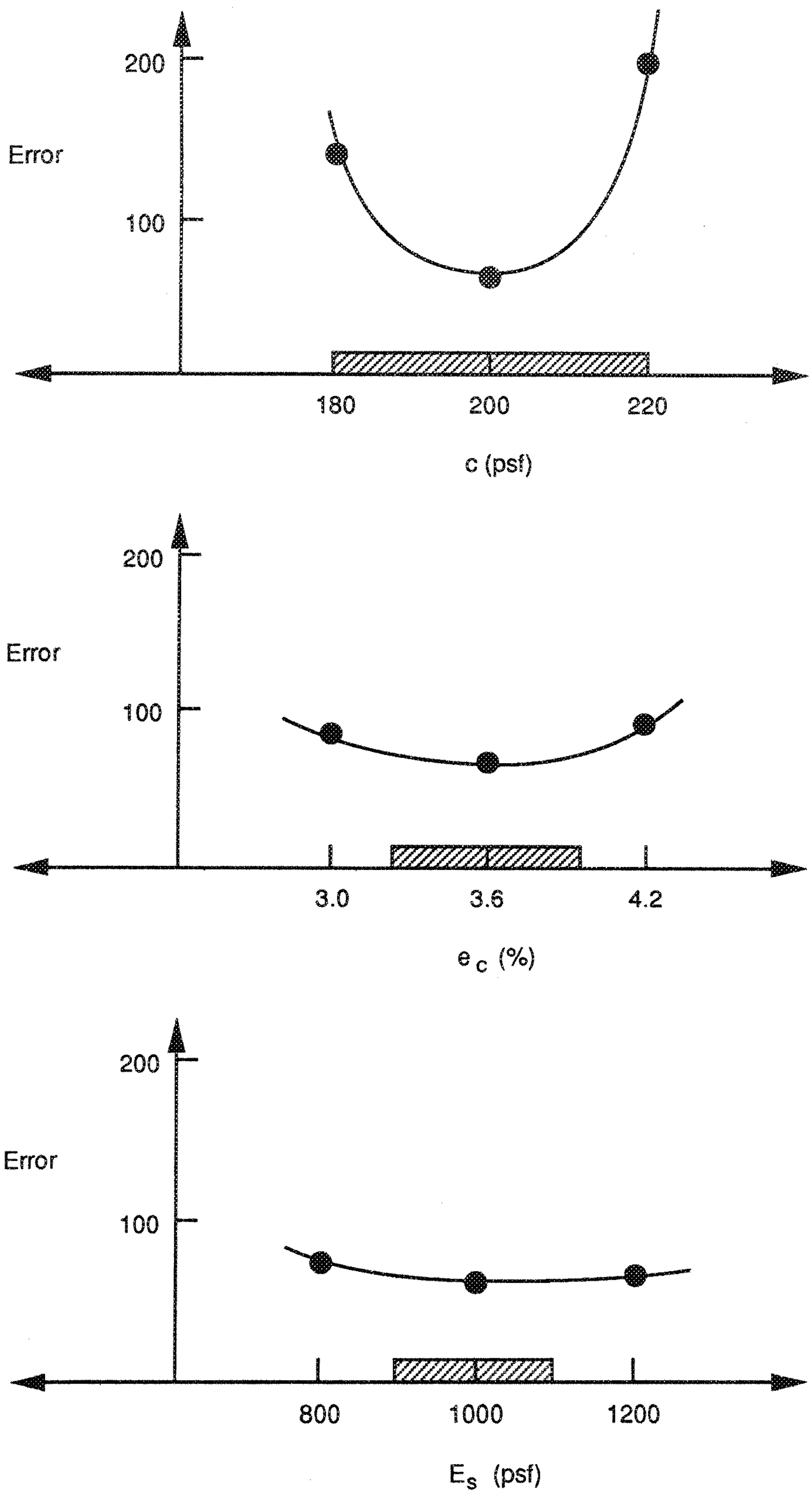

Figure 39. Results of Parameter Sensitivity Analysis

testing. Poulos (13), on the basis of lateral load tests in cohesive soils reported by Broms (14), computed secant soil moduli ranging from 15 (low cohesive strength) to 95 (high cohesive strength) times the cohesive strength of the soil. In comparison the inferred secant soil modulus of 1,000 psf appeared reasonable for the Mercer Slough peat tests, since the fibrous component of shear resistance provided by fiber tension in peat requires significantly more strain to be mobilized than is required in the non-peaty cohesive soil considered by Broms (14). The inferred critical strain of 3 percent was generally consistent with that observed in the UU triaxial tests on the Mercer Slough peats and for other very soft soils and was consistent with the assumption that the soil degradability factor, F, was equal to 1. (6)

These inferred soil parameters corresponded to the soil response at 15 minutes after the load application. As previously mentioned, stress relaxation of the peat was observed in the form of decreasing pile head loads with time under constant pile head deflection. If constant pile head load had been maintained, creep of the peat would have been expected to result in increasing deflections with time. The rate of change of the pile head load, which appeared relatively insensitive to load amplitude, is shown in terms of the ratio of the pile head load at time, t, to the pile head load immediately after load application for TP2 in Figure 40.

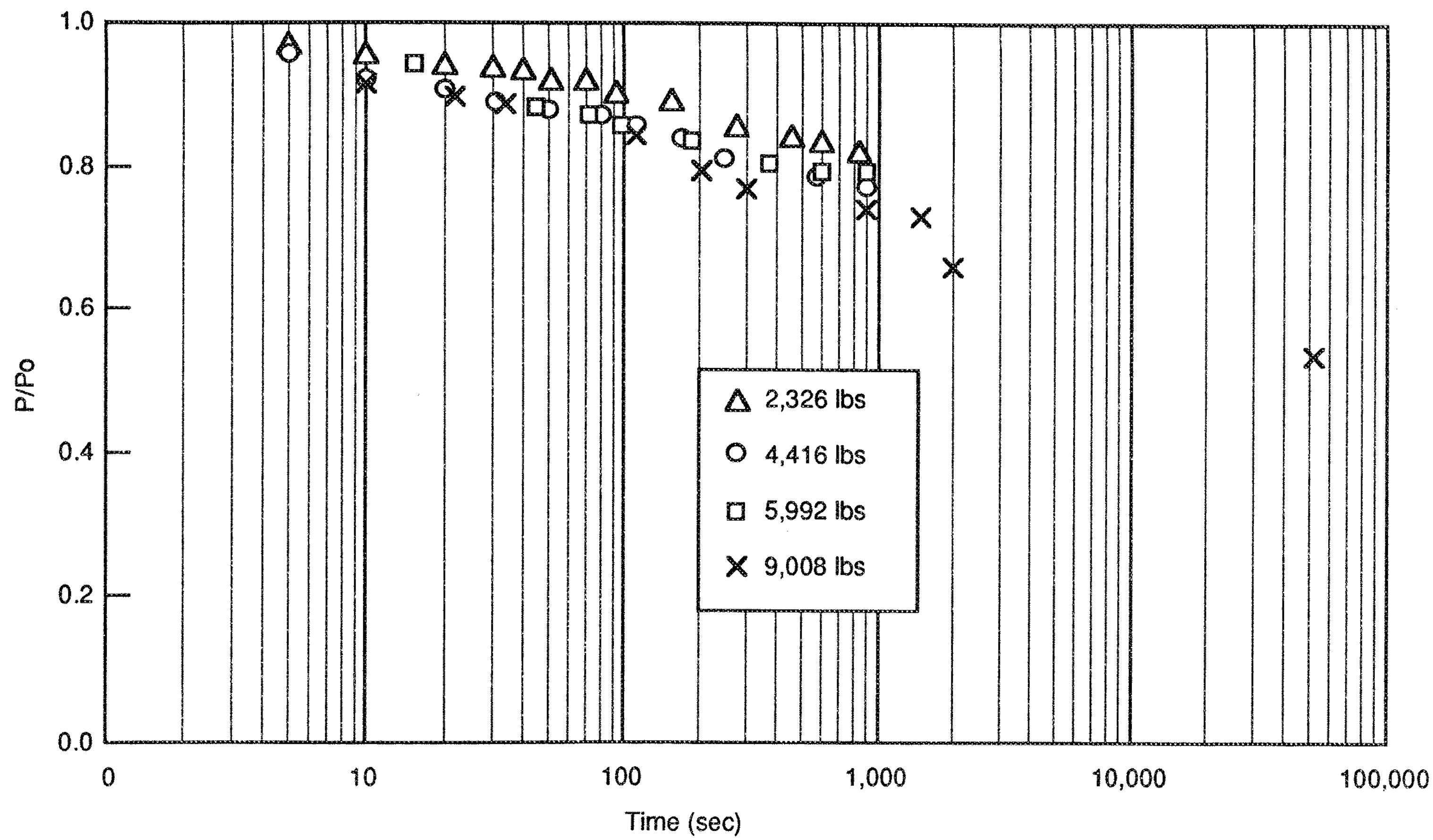

Figure 40. Rate Dependence of Pile Head Load

CONCLUSIONS AND RECOMMENDATIONS

The resistance of soil to the lateral movement of piles can be represented by p-y curves. When p-y curves are incorporated into an appropriate analysis, the response of a pile to lateral loads can be evaluated. Knowledge of this response, which includes pile deflection, rotation, bending moment, and shear, is necessary for the proper design of pile-supported highway structures. This type of analysis has been heavily used in the design of bridge foundations for many years.

The accuracy of the p-y method of analysis depends directly on the accuracy with which the p-y curves represent the ability of the soil to resist lateral pile deflections. In certain types of soil, methods for the development of p-y curves (known as p-y curve criteria) have been developed from the results of many lateral load tests on piles embedded in those types of soil. For soils classified as clays, p-y behavior may be accurately characterized by use of the Integrated Clay Criterion. (6) For soils classified as sands, p-y behavior may be obtained with the Extended Hyperbolic Criterion. (15) However, little information on the response of laterally loaded piles in soils other than clays or sands is available. In Washington, bridge foundations are commonly constructed in silts, gravels, and peats, whose laterally loaded pile behavior has not been previously studied.

The performance of full-scale lateral load tests on instrumented piles represents a valuable technique for evaluation of soil resistance to lateral pile deflection. At a given site, lateral load tests will provide the best possible indication of the p-y behavior of the soil at that site. *In the absence of well-developed p-y curve criteria for a particular soil, p-y characteristics of the soil are best obtained from full-scale lateral load tests.*

The results of full-scale lateral load tests on piles in the silts of Deep River and the peats of Mercer Slough provided valuable information for the development of p-y curves for those soils and soils similar to them. However, this information was still based on a very limited data base that should be expanded by the inclusion of future test results in similar soils. Consequently, the recommendations that follow should be used with sound engineering judgment. Recommendations for the development of p-y curves in these soils are presented in the following sections.

DEEP RIVER SILTS

The response to laterally loaded piles in the Deep River silts and similar soft silts can be evaluated with the Integrated Clay Criterion. Input parameters can be determined as follows.

Cohesive strength, c

For relatively rapid loading of the type imposed on the test piles in the Deep River tests, the cohesive strength, c, may be taken as the undrained strength, s_u, of the soil. The variation of undrained strength with depth may be determined in the following ways:

a. Drilling/Sampling/Laboratory Testing — the undrained strength profile may be determined through undrained testing of undisturbed samples in the laboratory. Samples should be tested under UU conditions if no consolidation is anticipated and under CU conditions if the effective stresses are expected to increase by the time lateral loads are applied.

b. Cone Penetration Testing — the undrained strength profile can be obtained from the results of cone penetration tests. The cone penetration tests should be interpreted in terms of UU strength using Equation 3.

c. Vane Shear Testing — the undrained strength profile of a soil can be obtained from vane shear testing. Vane shear tests must be corrected for plasticity index and often exhibit considerable scatter and uncertainty.

The undrained strength profile in soft silts of the type encountered at Deep River will generally be linear at depths below a shallow surface crust. While the undisturbed surface crust is stiffer and stronger than the immediately underlying soils, the effects of disturbance during pile installation often soften and weaken the crust. Therefore the undrained strength profile below the surface crust should be extrapolated smoothly to the ground surface in order to develop a design undrained strength profile for use with the Integrated Clay Criterion. Consideration can be given to the use of higher undrained strengths for the crust in cases where it can be demonstrated that the crust disturbance is substantially lower than that which occurred at the Deep River site.

Stiffness/Strength Ratio, E_S/c

The secant soil stiffness can be considered to be proportional to the cohesive strength of the soil. For the Deep River silts and similar soft silts, the response of laterally loaded piles can be evaluated with the relationship $E_S/c = 50$.

Critical Strain, ε_c

The concept of a critical strain is useful as a reference strain with which the p-y curve shape can be described. The critical strain is defined in the Integrated Clay Criterion as the axial strain at a deviator stress of one-half the maximum deviator stress in a triaxial compression test, even though there is no fundamental reason for doing so. Evaluation of the response of the Deep River load tests indicated that the use of a critical strain of

$$\varepsilon_c = 0.35 \frac{c}{E_s}$$

provides good prediction of the observed pile response. This critical strain will be much smaller than the strain at one-half the maximum deviator stress, and should be considered as a reference strain.

Soil Degradability Factor, F

The soil degradability factor is a measure of the ductility of the soil in the development of its resistance to lateral pile movement. Since the Deep River silts and other soft silts can be expected to exhibit ductile stress-strain behavior, the soil degradability factor should be assumed to be unity.

MERCER SLOUGH PEATS

The response of laterally loaded piles in the Mercer Slough peats was evaluated with the Integrated Clay Criterion. Evaluation of Integrated Clay Criterion parameters for peats of the type encountered in Mercer Slough is complicated by the difficulty associated with evaluation of the cohesive strength of the peat.

While procedures for estimating Integrated Clay Criterion parameters are recommended in this report, they should be considered appropriate for the peats of Mercer Slough but only approximate for other peats. When other peat deposits are encountered, *additional field load tests should be performed whenever possible to supplement these recommendations*. Even field load tests on small diameter piles with only pile head load-deflection measurements would be of use.

Input parameters for use in the Integrated Clay Criterion for peats similar to those encountered at Mercer Slough can be estimated as follows.

Cohesive strength, c

Mobilization of the cohesive strength of a fibrous peat adjacent to a laterally loaded pile involves mobilization of the tensile strength of the fibers. This tensile strength is generally not reflected in the results of small scale tests in peat; consequently, the strength determined by such tests may underestimate the available cohesive strength of the peat. Even though small scale in situ tests are of limited reliability in peats, they often represent the only feasible method of obtaining subsurface strength information of any kind. Given this undesirable situation, the following empirical methods for estimating of Integrated Clay Criterion cohesive strength are suggested:

a. **Drilling/Sampling/Laboratory Testing** — the available cohesive strength may be estimated as 1.4 times the average shear strength obtained from laboratory UU triaxial tests.

b. <u>Cone Penetration Testing</u> — the available cohesive strength may be estimated as twice the average undrained strength of the entire peat deposit, as obtained from Equation 3.

c. <u>Vane Shear Testing</u> — the available cohesive strength may be estimated as 1.55 times the average vane shear strength. Vane shear strength values of less than 50 psf at depths greater than 10 feet may be disregarded unless they are considered representative of the actual vane shear strength at those depths.

Since the composition and strength of peat deposits vary greatly over relatively short distances in both the horizontal and vertical directions, a single average value of cohesive strength should be assumed for the entire peat deposit, unless evidence to the contrary exists. Use of this approach with any of the three methods of cohesive strength estimation described above provides good agreement with the results of the pile load tests conducted at Mercer Slough.

These recommendations are rather specific, and are intended to apply to the soil encountered at Mercer Slough. Additional data are badly needed, and further full-scale load tests would be very valuable in confirming and extending the recommendations made herein.

<u>Stiffness/Strength Ratio, E_s/c</u>

The secant soil stiffness can be reasonably assumed to be proportional to the cohesive strength of peat. For peats similar to those of Mercer Slough, the response of laterally loaded piles can be evaluated using the relationship $E_s/c = 5$.

<u>Critical Strain, ε_c</u>

The resistance of peats similar to those of Mercer Slough to lateral pile deflection can be evaluated using $\varepsilon_c = 3.6\%$.

Soil Degradability Factor, F

For peats similar to those of Mercer Slough, the soil degradability factor can be assumed to be unity.

Again, it is imperative to realize that, while the above recommendations represent an improvement over previously available methods for development of p-y curves in silts and peats, they are based on a very limited amount of testing at a limited number of sites. The performance of further full-scale lateral load tests on piles in similar soils, whether conducted as part of a future research investigation or as "proof" tests on individual projects, is highly recommended to broaden the state of knowledge regarding the response of laterally loaded piles in these types of soils and to provide site-specific results for design purposes. Until such additional data are available, the results of these tests may be used to provide estimates of anticipated pile response but should not be considered likely to produce exact predictions.

IMPLEMENTATION

WSDOT may implement the results of this research project by following the recommendations for p-y curve development described in the section titled "Conclusions and Recommendations." The recommended procedures for development of p-y curves for silts and peats can be directly implemented to develop input parameters for analysis of laterally loaded piles because they are compatible with computer programs already being used for that purpose. Implementation of the research results, therefore, should be easy and rapid.

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