Evaluation of In Situ Strength of a Peat Deposit from Laterally Loaded Pile Test Results

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Peat soils exhibit strength and stiffness characteristics that are strongly influenced by extension and interlocking of fibers within the peat. Consequently, the in situ strength and stiffness characteristics of peat cannot be measured accurately with existing small-scale laboratory or field testing instruments. The use of an instrumented pile lateral load test is described for evaluation of the strength and stiffness characteristics of a peat deposit that had been exerting lateral stresses on existing pile foundations. Pile load tests were performed on two relatively flexible piles installed in approximately 45 to 50 ft of fibrous peat. Each pile was instrumented with both strain gauges and inclinometers. The piles were loaded laterally at the ground surface until displacements increased so quickly that additional loads could not be placed. At this point, the strength of the peat had been mobilized in a relatively large volume of soil in the vicinity of the pile. Interpretation of the strain gauge and inclinometer readings allowed definition of the deflected shapes of the piles and of the unit soil resistance profiles. Those deflected shapes allowed evaluation of the *p-y* behavior of the soil from which the strength and stiffness could be obtained. Those strength and stiffness values were consistent with those obtained for other peats and with the available results of a previously performed laboratory testing program.

Measurement of the shear strength of peats has posed a difficult problem for geotechnical engineers for many years. Many peats exhibit a component of shear strength that results from extension and interlocking of fibrous, organic material within the peat (*1*,*2*). 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. Landva (*2*), in a comprehensive review of in situ testing of peats, states 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." In this paper, the use of a laterally loaded pile test as an in situ test to measure the strength and stiffness of peat is described. The resistance of a peat to lateral displacement of a pile requires mobilization of shear strength in a relatively large volume of soil in the vicinity of 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.

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In situ strength tests generally fall into two main categories: direct measurement tests, which directly measure the strength of the soil on a known and constrained failure surface, and indirect measurement tests, which mobilize the strength of the soil on an unknown failure surface where the geometry is obtained from an appropriate theory of soil mechanics. The vane shear test (*3*,*4*) and the Iowa borehole shear test (*5*,*6*) are examples of the former, and the cone penetration test (*7*,*8*), pressuremeter test (*9*,*10*), screw plate test (*11*,*12*), and plate load test (*13*,*14*) are well-established examples of the latter. The use of indirect measurement tests requires that the physical phenomena to be measured be understood and that a reasonable model relating the measured phenomena to the parameter(s) of interest be available. Modeling of the response of pile foundations to lateral loads has developed rapidly in recent years to the point where the *p-y* behavior of soft soils can be expressed in terms of their strength and stiffness characteristics (*15*,*16*).

BACKGROUND

The site, profiled in Figure 1, is located on the I-90 right-of-way immediately west of Lake Washington Boulevard near the eastern shore of Lake Washington in Bellevue, Washington. The site is traversed in the east-west direction by a pile-supported Seattle Water Department pipeline and four pile-supported I-90 elevated-bridge structures. The eastern edge of the site is bordered by the Lake Washington Boulevard embankment fill, which rises to an elevation about 15 ft above the remainder of the site. Figure 2 is a plan of the site. Subsurface soil movement in the area of the eastbound collector-distributor (EBCD) and westbound collector-distributor (WBCD) ramps of the I-90 Bellevue Transit Access project has been observed over a period of several years. This subsurface soil movement has resulted in movement of existing pile-supported structures in the area, including highway bridges and a water supply pipeline. The original purpose of the research described in this paper was to investigate the lateral load behavior of piles in the Mercer Slough peats for the design of new pile-supported structures and to estimate the forces exerted on the existing piles by lateral movement of the surrounding peat.

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 approx-

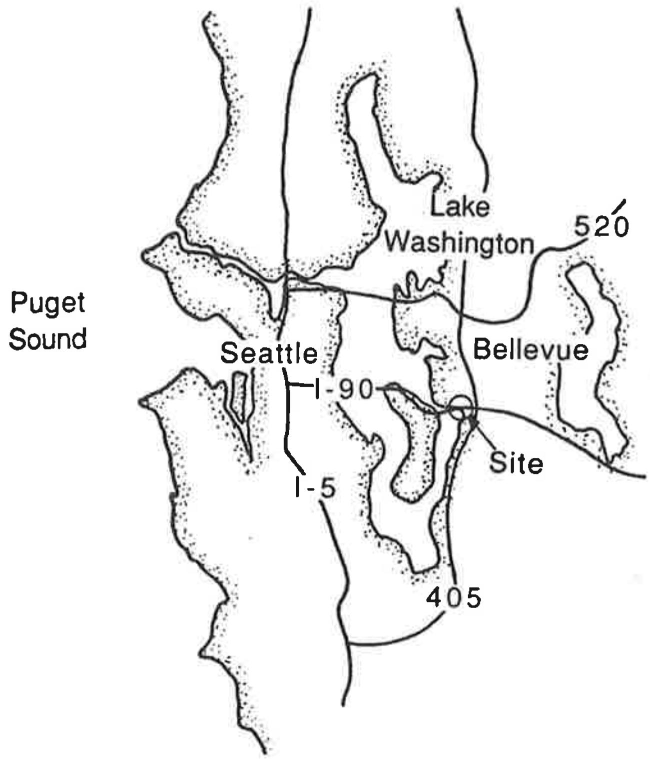

FIGURE 1 Site location.

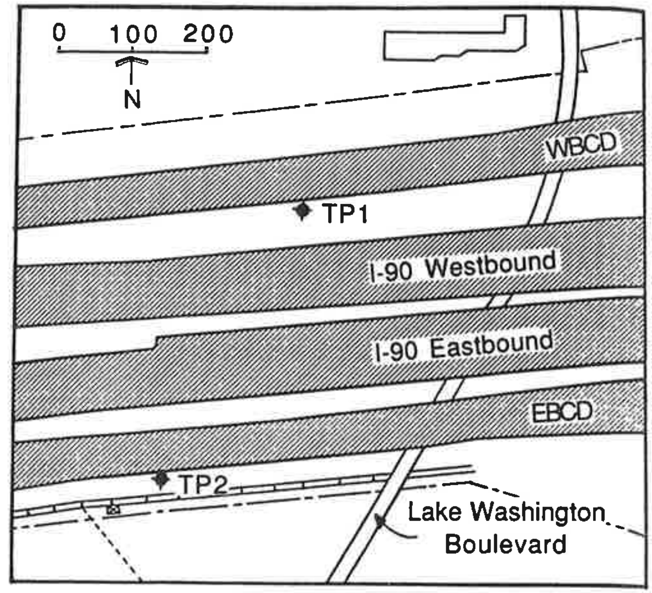

FIGURE 2 Site plan.

imately at the ground surface. Subsurface investigations indicated that the site subsurface conditions are dominated by a peat deposit of variable thickness. The peat deposit generally overlies clay and silt deposits, which overlie granular materials. Artesian pressure conditions have been observed in the soils underlying the peats. The peat was described in previous subsurface investigations performed for the Washington State Department of Transportation as a "brown, fibrous, organic material, with a low dry density and shear strength and high water content and compressibility." The Lake Washington Boulevard fill rests at least partially on the Mercer Slough peat, and its weight is considered to be the driving force causing movement of the peat. A subsurface profile along the WBCD is presented in Figure 3.

FIELD AND LABORATORY MEASUREMENT OF PEAT STRENGTH

Previously performed subsurface investigations in the vicinity of the pile load test sites consisted of conventional boring and sampling along with vane shear and cone penetration test

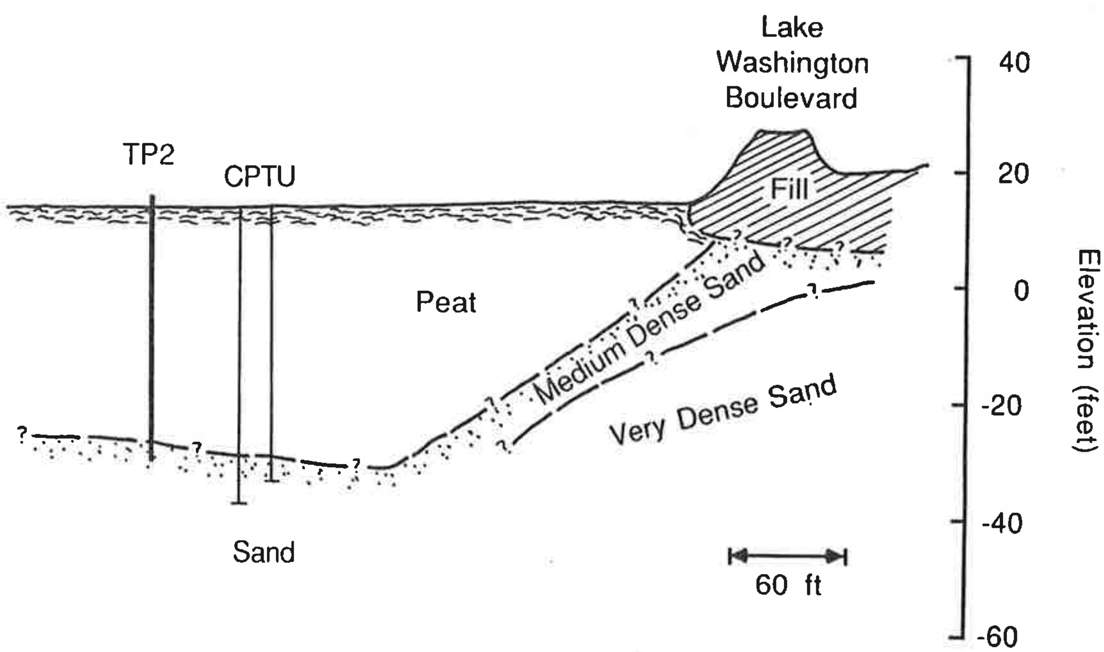

FIGURE 3 Subsurface profile.

profiling. Unconsolidated-undrained (UU) triaxial and vane shear tests indicated peak undrained shear strength ranging from values as low as 15 to 175 psf with a possible trend of modestly increasing strength with depth (see Figure 4). This large range of variability is consistent with that observed in other studies of vane shear tests in peat (*2,17*). Two piezocone penetration test profiles indicate 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 as indicated in Figure 5. Interpretation of cone penetration tests in peat has been recognized as being very difficult (*2*) owing to lateral deflection of typical size penetrometers and to the mode of deformation and failure.

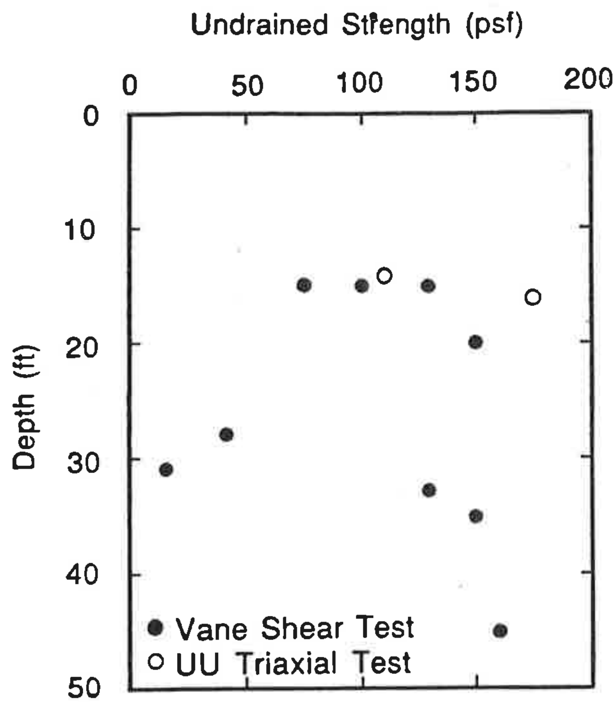

FIGURE 4 Triaxial and vane shear strength data.

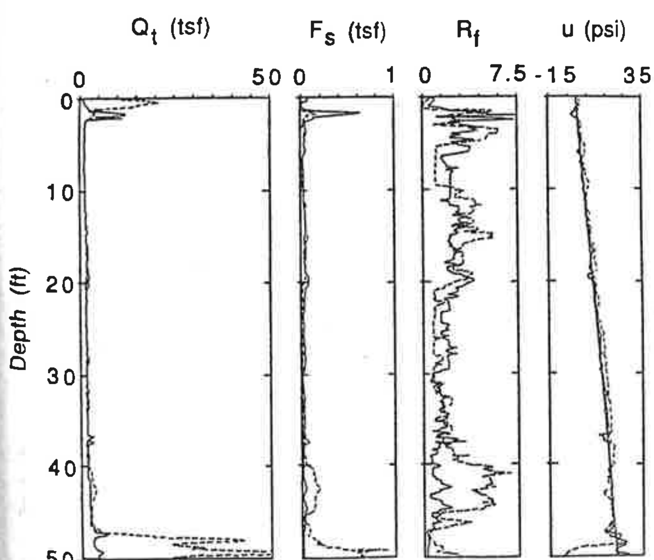

FIGURE 5 Piezocone and penetration test logs.

LATERAL LOAD BEHAVIOR

Two test piles were installed in the Mercer Slough peats and were subjected to lateral loads at the ground line. Flexible piles were required to develop the pile bending necessary for evaluation of unit soil resistance because of the very soft nature of the soil. Test pile diameters reasonably near those of the actual piles were required to model the actual piles accurately in the area with minimal scale effects. Those conflicting requirements led to the optimum selection of an intermediate size pile with relatively thin walls.

Test Materials, Instrumentation, and Procedures

The piles used in the lateral load tests were 8-in. diameter steel pipe piles with 0.25-in. wall thickness. The piles were nominally 60 ft long and were installed with open ends to allow penetration under their own weight as far as possible and then by additional static vertical load supplied by the boom of a boom truck. The piles were instrumented with 11 pairs of bonded resistance strain gauges placed diametrically opposite each other at distances of 5 ft 8 in., 6 ft 4 in., and 7, 8, 9, 11, 13, 17, 23, 29, and 37 ft from the top of the pile. The strain gauges and associated wiring were waterproofed and protected by 1½-in. steel angles welded to the outside of the piles. The steel angles were attached in 10-ft-long sections, each lightly welded at only their bottom ends, to provide the desired protection during pile installation without influencing the flexural resistance of the pile.

Test pile 1 (TP1) penetrated under its own weight to a depth of approximately 15 ft and was then pushed to refusal at a tip depth of about 51 ft. The pile, originally 59 ft long, was then removed and reinstalled nearby after cutting a 5-ft length off the bottom to allow the highest strain gauges to be located just below the ground surface. Test pile 2 (TP2) penetrated under its own weight to a depth of about 20 ft and was then pushed to refusal at a tip depth of approximately 43 ft. The pile, originally 60 ft long, was removed and reinstalled nearby after cutting a 14-ft length off the bottom. After installation, the tops of TP1 and TP2 were 5 and 3 ft above the ground surface, respectively.

Lateral loads were applied to the piles by a 100-ton capacity, 9-in. throw, hydraulic jack. The deflection and slope of the pile at the point of load application were obtained by measuring the horizontal distance between the pile and each of the three spring-tensioned horizontal wires stretched at different heights between stakes placed outside of the zone of influence of the pile. The subsurface deformation of each pile was monitored by both strain gauges and an inclinometer. The strain gauges were monitored during testing by a PC-based automatic data acquisition system that was housed at the site in a small tent and was powered by a dedicated generator. A 40-ft long slope inclinometer casing was suspended inside each of the piles and was pressed against the sides of the piles by an inflatable air bag. The air bag was constructed of 6-mil polyethylene sheeting with an unrestrained inflated diameter of about 24 in. When installed in the 7.5-in. I.D. pile and inflated to approximately 1.5 psi by a high-volume low-pressure compressor (ShopVac), stresses in the air bag itself were very low and leakage or rupture was not observed. Slope measurements were made at 2-ft intervals along the length of the inclinometer casing.

On TP1, the loads were applied through a cabling arrangement so that the test pile was pulled toward the reacting bridge pier, located approximately 12 ft from the test pile. On TP2, the pile was jacked away from a nearby pile cap. Applied loads on TP1 were measured by a GEOKON model 3000 load cell provided by the Washington State Department of Transportation (WSDOT). 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, which operated essentially as a displacement-controlled device. The resistance of the peat to lateral loads was observed to be time dependent, with the pile head load observed to decrease with time under constant pile head deflection. The top deflection and slope, and the load cell and strain gauges, were read immediately after application of each load increment and again after a period of approximately 10 to 15 min, 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.

Test Results

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 in. TP2 was installed on April 21, 1989, and was tested on April 24, 1989. Lateral loads were increased incrementally with no unload-reload loop in order to simulate the type of monotonically increasing loads that would be caused by moving peat. The load-deflection response measured at the tops of TP1 and TP2 were generally similar (see Figure 6).

The survivability of the resistance strain gauges was lower than was expected. Whether owing to mechanical damage during installation or to ineffectiveness of waterproofing measures, only a few of the strain gauge pairs functioned properly. The inclinometer data, however, were consistent with independent measurements of top deflection and slope. The inclinometer (slope) and working strain gauge (curvature) data were combined for each pile and reduced by multivariate optimization with multiple constraints for conformance with the measured boundary conditions at the top of the pile and with the assumption of fixity at the bottom of the pile. This procedure allowed expression of the deflected shape of the piles in terms of 8th-order polynomials. The deflected shapes of TP2 obtained by this procedure for lateral loads of 2.4, 4.8, and 6.9 kips are presented in Figure 7.

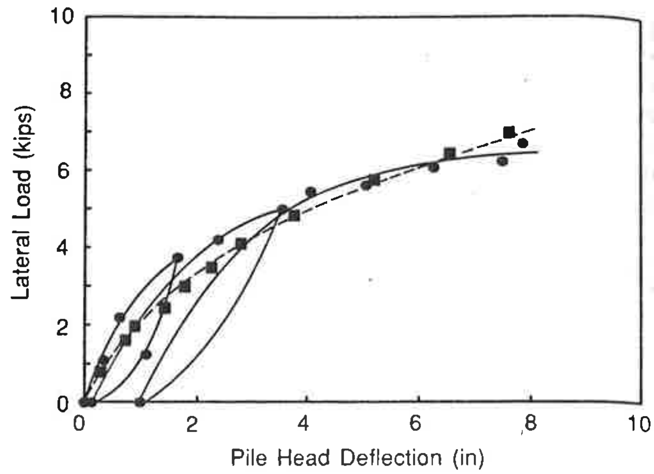

FIGURE 6 Load displacement response of TP1 and TP2.

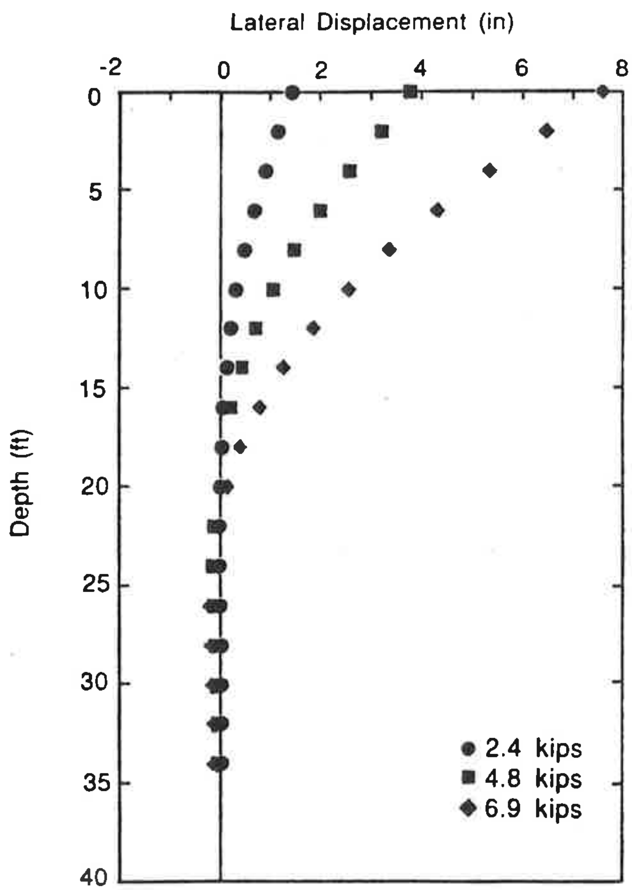

FIGURE 7 Observed deflected shapes of TP2.

INTERPRETATION OF LOAD TEST RESULTS

Evaluation of in situ shear strength from the results of an indirect in situ test requires interpretation of the test results within the framework of some applicable theory of soil mechanics. Cone penetration test interpretation, for example, often makes use of deep-bearing capacity theory, while plate load test interpretation relies on shallow-bearing capacity theory. Interpretation of the lateral load tests described in this paper was performed within the framework of *p-y* curve analysis.

For this purpose, the data from both test piles were initially combined because no significant difference in the characteristics of the peats at the two test sites was apparent from the available information. However, because a more accurate load cell was used for TP2 and because TP2 inclinometer readings were each made at a consistent time of 15 min after the time of load application, the final interpretation of results was based solely on the TP2 data.

To correlate the results of those lateral load tests with the known behavior of laterally loaded piles in other types of soils, the test results were interpreted within the framework of an existing *p-y* curve development procedure. The procedure selected was the integrated clay criteria of O'Neill and Gazioglu (*16*). The integrated clay criteria are similar in many respects to the soft clay criteria of Matlock (*15*), but they have been shown to represent the influence of pile diameter more accurately and to be accurate over a much wider range of soil conditions than the soft clay criteria. Those features of the integrated clay criteria were considered important in the interpretation of the results of only two tests on small-diameter piles in the unusual soil conditions of Mercer Slough.

The integrated clay criteria specify a relationship between unit soil resistance p and lateral pile deflection y at depth x to be of the form

$$p = \frac{1}{2} p_{ult} \left(\frac{y}{y_c}\right)^{0.37} \leq p_{ult}$$

where

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

$$p_{ult} = F\, N_p\, cD$$

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

$$x_{cr} = 0.25 L_c$$

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

and

E_s = secant soil stiffness,
ε_c = critical strain [at one-half $(\sigma_d)_{max}$ in UU triaxial test],
F = soil degradability factor describing brittleness of soil,
c = cohesive strength of soil, and
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 the ductile peat material, the soil degradability factor was assumed to be equal to 1 (*16*).

The remaining three unknowns were varied in a direct search optimization procedure to find the combination of soil properties providing the best fit with the observed results. The soil properties were assumed to be constant throughout the peat because from the information entropy standpoint (*17*) there were no compelling reasons 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 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 taken as

$$\text{weighted error} = \sum_{i=1}^{n} \frac{L + 1}{x_i + 1} \left| y_{meas} - y_{pred} \right|_i$$

where n is the number of points at which measured and predicted displacements were compared, L is the length over which measured and predicted displacements were compared and y_{meas} and y_{pred} are the measured and predicted displacements, respectively, at depth x_i. To obtain parameters that predicted the observed behavior at both low loads and deflections and at high loads and deflections, the total weighted error was taken 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 c = 200 psf, E_s = 1,000 psf, and ε_c = 3.6 percent.

Integrated clay criteria *p-y* curves developed from those values predicted pile head load-displacement behavior, which agreed well with the observed behavior (see Figure 8). The predicted deflected shapes of the pile also agreed reasonably well with the observed deflected shapes (see Figure 9). As was 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 10, where the shaded bands along each abcissa represent ±10 percent deviation from the inferred parameter value.

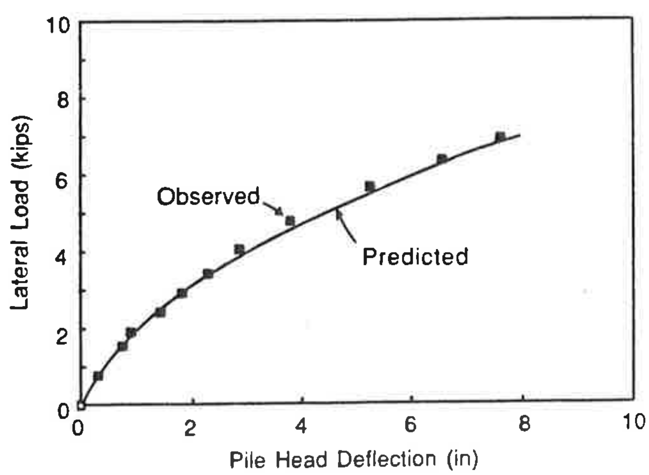

FIGURE 8 Observed and predicted load-displacement behavior.

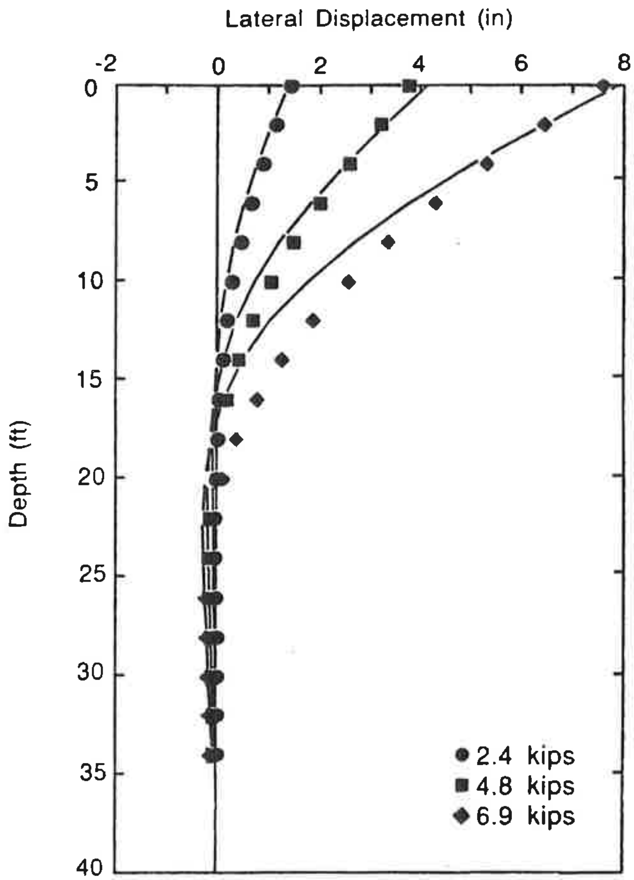

FIGURE 9 Observed and predicted deflected shapes.

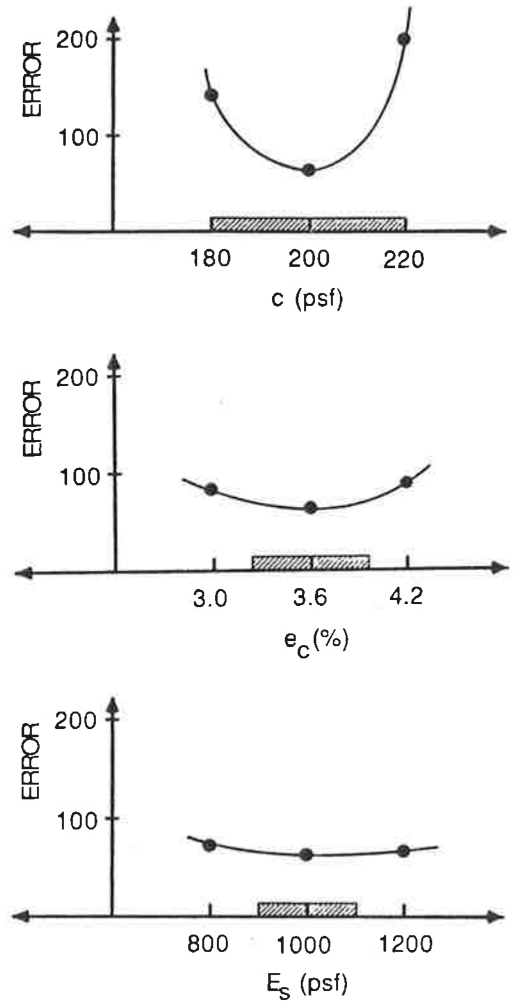

FIGURE 10 Results of sensitivity analysis.

The inferred cohesive strength of 200 psf is somewhat higher than the strengths obtained from the field and laboratory tests previously performed at the site but is within the range of shear strengths reported for other peats. The difference is likely attributable to the fibrous component of strength in the peat that is lost during sampling and not mobilized during relatively small-scale vane shear and cone-penetration testing. Poulos (*18*), on the basis of lateral load tests in cohesive soils reported by Broms (*19*), computed secant soil moduli ranging from 15 (low cohesive strength) to 95 (high cohesive strength) times the cohesive strength of the soil. The inferred secant soil modulus of 1,000 psf then appears reasonable for the Mercer Slough peat tests because the fibrous component of shear resistance provided by fiber tension in peat will require significantly more strain to be mobilized than would be required in the nonpeaty cohesive soil considered by Broms (*19*). The inferred critical strain of 3 percent is generally consistent with that observed in the UU triaxial tests on the Mercer Slough peats and for other very soft soils and is consistent with the assumption that the soil degradability factor F was equal to 1 (*16*).

SUMMARY AND CONCLUSIONS

In a case where the shear strength of a fibrous material such as peat or other soil with strong secondary structure is to be measured, an in situ test that mobilizes shear strength in a relatively large volume of soil is desirable. In a peat deposit in Washington state, a laterally loaded pile test was successfully used to measure in situ shear strength. It must be recognized, as with all in situ strength tests, that the measured strength corresponds to the failure mechanism induced in the soil by the test and should therefore be applied with care to problems involving significantly different failure mechanisms. Also, by the nature of the laterally loaded pile problem, the measured in situ strength is expected to be more representative of the strength near the ground surface than at greater depths.

The resistance to two laterally loaded test piles offered by the Mercer Slough peats was reasonably described by use of the integrated clay criteria with a cohesive soil strength of 200 psf, a soil secant modulus of 1,000 psf, and a critical strain of 3 percent. Those strength and stiffness parameters are consistent with those observed for other peats and with the results of laboratory tests of the peats at the site. Thus, they are considered to provide an indication of the strength and stiffness of the Mercer Slough peats that is improved over that obtained by small-scale field and laboratory tests.

REFERENCES

1. A. Landva and P. LaRochelle. *Compressibility and Shear Characteristics of Radforth Peats.* STP 820. ASTM, Philadelphia, Pa., 1983, pp. 141–156.
2. A. Landva. In-Situ Testing of Peat. *Proc., ASCE Conference on Use of In-Situ Tests in Geotechnical Engineering*, Blacksburg, Virginia, 1986, pp. 191–205.
3. L. Cadling and S. Odenstad. The Vane Borer. *Proceedings No. 2*, Royal Swedish Geotechnical Institute, 1950, pp. 1–88.
4. L. Bjerrum. Embankments on Soft Ground. In *Proc., ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures*, Purdue University, Vol. 2, 1972, pp. 1–54.
5. J. D. Wineland. Borehole Shear Device. In *Proc., ASCE Specialty Conference on In-Situ Measurement of Soil Properties.* Raleigh, N.C., Vol. 1, 1975, pp. 511–522.
6. J. H. Schmertmann. Measurement of In-Situ Shear Strength. State-of-the Art Report. In *Proc., ASCE Specialty Conference on In-Situ Measurements of Soil Properties*, Raleigh, N.C., Vol. 2, 1975, pp. 57–138.
7. G. Sanglerat. *The Penetrometer and Soil Exploration.* Elsevier, New York, 1972.
8. *Proceedings of the European Symposium on Penetration Testing*, Stockholm, Swedish Council for Building Research, 1974.
9. L. Menard. The Menard Pressuremeter. *Les Editions Sols-Soils*, No. 26, 1975, pp. 7–43.
10. R. J. Mair and D. M. Wood. *Pressuremeter Testing: Methods and Interpretation.* Butterworths, London, 1987.
11. N. Janbu and K. Senneset. Field Compressometer: Principles and Applications, *Proc., Eighth International Conference on Soil Mechanics and Foundation Engineering*, Moscow, Vol. 1.1, 1973, pp. 191–198.
12. T. K. Mitchell and W. S. Gardner. In-Situ Measurement of Volume Change Characteristics, State-of-the-Art Report, *Proc., ASCE Specialty Conference on In-Situ Measurement of Soil Properties*, Raleigh, N.C., Vol. 2, 1975.
13. K. Terzaghi and R. B. Peck. *Soil Mechanics in Engineering Practices*, John Wiley and Sons, New York, 1948.
14. R. B. Peck, W. E. Hanson, and T. H. Thornburn. *Foundation Engineering*, John Wiley and Sons, New York, 1953.
15. H. Matlock. Correlations for Design of Laterally Loaded Piles in Soft Clays. Presented at 2nd Annual Offshore Technology Conference, 1970.
16. M. W. O'Neill and S. M. Gazioglu. *An Evaluation of p-y Relationships in Clays.* Research Report UHCE-84-3. University of Houston, April 1984.
17. M. Tribus. *Rational Descriptions, Decisions and Designs*, Pergamon, New York, 1969.
18. H. G. Poulos. Behavior of Laterally Loaded Piles: I, Single Piles. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 92, No. SM5, 1971, pp. 711–751.
19. B. B. Broms. Lateral Resistance of Piles in Cohesive Soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 91, No. SM2, 1964, pp. 3825–3863.

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