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Hyperbolic P-Y Model for Static and Cyclic Lateral Loading Derived from Full-Scale Lateral Load Testing in Cemented Loess Soils

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ABSTRACT

A new hyperbolic model for generating P-Y curves is developed for use in lateral analyses of cemented soils (a "C-Φ" soil with both cohesive intercept and friction angle), and includes degradation of the static loading soil response to model the effects of load cycling (repeated loading) with load cycle number. The p-y curves, for both static and cyclic loading, were derived from full-scale lateral test results conducted on a set of six drilled shafts in a loess deposit at the Kansas DOT test site. Values of the model parameters obtained from the load testing at this site are presented, as well as a discussion of the model parameters effect on the results such that the model may be rationally applied to other sites with C-Φ type soils. A step-by-step procedure is presented to illustrate the application of the model. Correlation of the model to the CPT cone tip resistance (q_c) from CPT testing is made, as this was found to be the most reliable; However, the model is applicable to other methods, both in-situ and laboratory, of assessing the soils strength.

INTRODUCTION

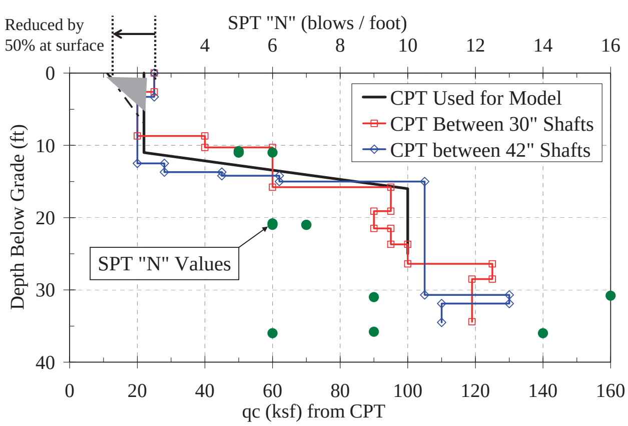
A deposit of loess, known as the Loveland Member, located on the northwest corner of I-435 and State Highway 32 in Wyandotte County, Kansas, was selected for its deep, uniform deposit of loess and deep groundwater table. The soil parameters are summarized, while a complete discussion of the origin and significant engineering properties of loessal soils, laboratory and in situ tests performed at the test site, and an evaluation of existing correlations for in-situ test data when applied to loess is contained in Parsons et al. (2009). Two pairs of 30-inch (762-mm) diameter shafts and one pair of 42-inch (1067-mm) diameter shafts (six shafts total) were tested simultaneously using a hydraulic jack acting between a shaft pair with both shafts fully instrumented. All shaft spacing was at approximately 12 ft (3.66 m) center-to-center. All shafts were approximately 47 feet (14.3 m) deep, and had 3.1 feet (0.95 m) of surface casing that extended to only a few inches below grade.

SITE INVESTIGATION AND SOIL CONDITIONS

A total of thirteen SPT borings (SPT) were made using a hollow stem auger and an automatic hammer. Undisturbed soil samples were obtained using 89 mm diameter Shelby tubes. Three cone penetration tests (CPT), two pressuremeter tests (PMT), and two continuous soil profiles were obtained (using an A.D. Bull Soil Sampler). All of these in-situ tests (except CPT-1) were performed in 2005, and were conducted within two days of the lateral load testing. Laboratory testing consisted of CU as well as UU triaxial compression tests, unconfined compression tests, direct shear tests (samples cut both parallel and perpendicular to the sampler tube axis), consolidation, collapse, and index property tests. A back-fit model of the pile behavior using the variety of soil strength data obtained (both in-situ and laboratory) to the measured pile performance led to the conclusion that the CPT testing provided the best correlation.

Laboratory testing classified the soil as low plasticity clay (CL) from the ground surface to a depth of 4.9 meters (16 feet) and from 8.5 to 9.8 meters (28 to 32 feet) with a layer of non-plastic to low plasticity silt (ML) from 4.9 to 8.5 meters (16 to 28 feet). CPT 1 and 3 indicated a clay layer from the surface to a depth of approximately 2.7 to 3.7 m (9 to 12 feet), then alternating layers of silty sand, sandy silt, and clayey silt, while CPT 2 indicated a soil profile consisting of alternating layers of sand, clayey sand, gravely sand, all with no silt. Confirmation of CPT soil classification through sampling is recommended for areas with loess.

An idealized profile of cone tip resistance (q_c) with depth interpreted as an average from the CPT soundings performed between the static test shafts (8-9 June 2005). The cone tip resistance (q_c) was reduced by 50% at the soil surface, and allowed to return to the full value at a depth equal to two pile diameters to account for the passive wedge failure mechanism exhibited near the ground surface, as is illustrated in Fig. 1. The idealized tip resistance (q_c) values with depth were correlated to the model parameter of the Ultimate Soil resistance (P_{uo}) that can be provided by the soil at corresponding depths.



[FIG. 1] Idealized Tip Resistance (q_c) Profile form CPT Testing

Triaxial and direct shear testing showed a consistent increase in friction angle from approximately 20° near the ground surface to approximately 26° at a depth of 2.1 m (7 ft), while values calculated using correlations for CPT and SPT were consistently higher by nearly a factor of 2. Triaxial and direct shear testing also showed a decrease in cohesion from 35kPa (5 psi) near the ground surface to approximately 8 kPa (1 psi) at depth of 2.1 m (7 ft). Elastic modulus data with depth from both in-situ (SPT, CPT, and PMT) and laboratory testing (UC, UU, CU) produced a trend line of modulus E(kPa) = 5000 + 1300 · [Z – 4], where Z is the depth in meters. The pressuremeter values were high while the UC test values were low when compared with the trend of the other tests, while the values computed from CPT test results test had limited variability and reflected an intermediate value. A detailed presentation of the soil testing, both in-situ and laboratory, is presented in Parsons et al. (2009) along with an interpretation as to the accuracy and reliability of the parameters obtained.

LATERAL LOAD TESTING

Pairs of shafts were tested simultaneously to provide reaction for each other, with both shafts fully instrumented, using a hydraulic jack "pinned" between a shaft pair such that each shaft acted in a free-head condition, as seen in Fig. 2. A pair of 42-inch (1067-mm) diameter shafts and pair of 30-inch (762-mm) diameter shafts were statically tested; while a second pair of 30-inch diameter shafts was cyclically tested. A pair of LVDT's measured displacement at the top of each shaft relative to a beam that was simply supported at a distance away from the shafts being tested, and inclinometer soundings were made at select load increments to provide the deflected pile profile and depth to maximum moment. A load cell was placed inline with the hydraulic actuator, and a pressure transducer provided for a back-up to the load cell readings.

[FIG. 2] Test Set-Up

Static Tests: Load increments and decrements were sustained for duration of approximately 5 minutes, with the exception of the load increments with inclinometer soundings where the duration was approximately 20 minutes. The lateral loads were maintained near constant at load increments without inclinometer soundings; while the hydraulic pressure was locked off during an inclinometer sounding to better maintain the deflected pile shape with depth during the sounding. Displacement reported at each LVDT is the average of measured values for the duration of each load increment. Lateral loads were applied to the 42-inch and 30-inch diameter static tests as shown in Table 1.

[TABLE 1] Static Test Shaft Load Schedule

42-inch Diam.		30-inch Diam.	
Load (kips)	Inclin. Sounding	Load (kips)	Inclin. Sounding
0	Yes	0	Yes
18		10	
33		21	
49		30	
62		39	
78		51	Yes
92		59	
107	Yes	70	
123		79	Yes
137		90	
150	Yes	99	Yes
168		110	
183		120	
193	Yes	127	Yes
214		113	
219	Yes	83	
165		44	
113		0	Yes
60			
0	Yes		

Cyclic Tests: There were a total of four load increments ("A" through "D") on the cyclic test shafts, with ten load cycles performed per load increment, as shown in Table 2. The lateral load was sized for each load increment such that the pile displacements would be greater than those of the previous load increment to the extent that effects of plastic soil deformation from the previous cycle would be largely negated. The lateral load for load cycles 1 and 10 were sustained for approximately 15 to 20 minutes to allow time for inclinometer soundings to be performed. The hydraulic pressure was locked off during an inclinometer sounding to better maintain the deflected pile shape with depth. For the load cycles without inclinometer soundings (load cycles 2 through 9) the total cycle duration was approximately 1 minute, 2 minutes, 3.5 minutes, and 6.5 minutes for load increments "A" though "D", respectively. The load was reversed at the top of each load cycle to return the top of shaft to approximately the location that it was prior to testing.

[TABLE 2] Cyclic Test Shaft Load Schedule

Load Increm.	Load (kips)		Inclin. Sounding
	Max.	Min.	
	0	0	Baseline
A	50	-15	at Load Cycle's 1 and 10
B	79	-25	
C	99	-30	
D	127	-30	

FORMULATION OF P-Y MODEL

The measured lateral response in this series of load tests did not exhibit the severe degradation of lateral resistance at some specified displacement that is typically present in other p-y models for C-Φ soils, typically categorized as "silt" (Evans and Duncan, 1982). Additionally, this model generates a p-y response that is continuous and provides an efficient convergence with numerical programs, unlike many of the models in current use that have second order discontinuities. Additionally, the model correlates degradation of the p-y response with cycle number for cyclic loading, but still maintains a smooth hyperbolic curvature as was measured in the cyclic load tests.

The soil resistance per unit length of pile (P) is a simple definitive expression of the pile displacement (Y) and the secant modulus (E_s) at that displacement, shown in Equation 1. The initial modulus (E_i) is a simple definitive expression of the ultimate soil resistance expressed on a per unit length of pile basis (P_u) and the reference displacement (Y_i), shown in Equation 2. Both relationships are illustrated in Fig. 3.

$$P = E_s \cdot Y \qquad [1]$$

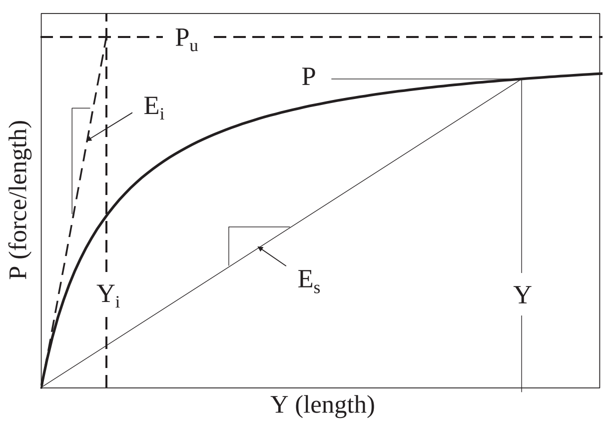
$$E_i = \frac{P_u}{Y_i} \qquad [2]$$

Where: E_s and E_i (force / length[2]),

Y and Y_i are (length), and

P and P_u are (force / length).

The magnitude of the soil resistance that will be provided at very large displacements will depend upon the value selected for use of the ultimate soil resistance expressed on a per unit length of pile basis (P_u); while the selection of the reference displacement (Y_i) will control the development of the soil resistance with displacement, and primarily affects the development at large pile displacements. Note that a correlation constant (*a*) will be subsequently presented which will allow for adjustments to the model for the development of soil resistance at small pile displacements [approximately 1 inch (25 mm) or less]. Additionally, a relationship will be presented to diminish the lateral response of the pile, in terms of both ultimate resistance as well as stiffness, by degrading value of ultimate soil resistance (Pu) for a given load cycle (N).



[FIG. 3] Hyperbolic P-Y Curve for Lateral Analyses

The secant modulus (E_s) for any given displacement (Y) is correlated by the following hyperbolic relationship involving the initial modulus (E_i) and a hyperbolic term (Y'_h) shown in Equation 3; Y'_h is in turn a function of the given displacement (Y), the reference displacement (Y_i), and a dimensionless correlation constant (a) as shown in Equation 4.

$$E_s = \frac{E_i}{1 + Y'_h} \qquad [3]$$

$$Y'_h = \left(\frac{Y}{Y_i}\right) \cdot \left[1 + a \cdot e^{-\left(\frac{Y}{Y_i}\right)}\right] \qquad [4]$$

Where: E_s and E_i are (force / length[2]),

a and Y'_h are dimensionless.

The reference displacement (Y_i) is defined as the displacement at which the tangent to the P-Y curve at zero displacement (E_i) intersects the Ultimate Soil resistance asymptote (P_u). For lateral displacement of the pile, this reference displacement (Y_i) may be considered as analogous to axial quake of a perfectly elastic-plastic bi-linear curve. Y_i is expressed in any consistent unit of (length).

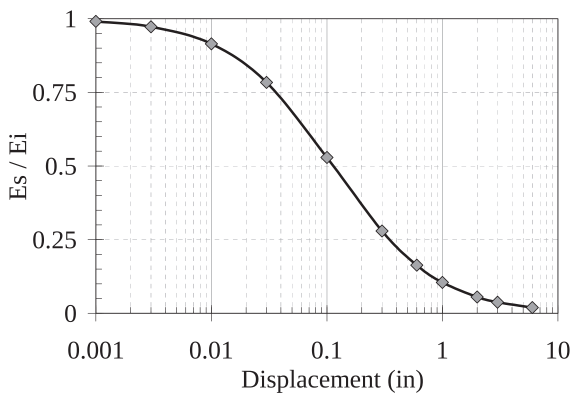
The best fit to the load test data was obtained with a reference displacement (Y_i) equal to 0.117 inches (3 mm). It is believed that the rate at which the strength develops, primarily represented by the reference displacement (Y_i), may be sensitive to soil type. Careful evaluation of the reference displacement (Y_i) parameter may be warranted when performing lateral analyses for piles within soil conditions that differ compared to those described herein, as this parameter may have substantial effect on the resulting pile deflections and stresses. The value of the reference displacement (Y_i) is inversely proportional to pile performance (i.e., a greater value of Y_i will allow for greater pile head displacements at a given lateral load).

The best fit to the load test data was obtained with the correlation constant (*a*) equal to 0.10. This value primarily affects the secant modulus (E_s) at small displacements [say within approximately 1 inch (25 mm)], and is inversely proportional to the pile performance (i.e., a larger value of *a* will curtail early development of soil resistance with displacement).

By substitution of the expression for the hyperbolic term (Y'_h) of Equation 4 into Equation 3, we note that the modulus ratio (E_S/E_i) is a function of only the hyperbolic parameters of the correlation constant (*a*) and the reference displacement (Y_i), as is shown in the Equation 5. Using the hyperbolic parameters $Y_i = 0.117$ inches (3 mm) and $a = 0.10$, this relationship of modulus ratio (E_S/E_i) vs. displacement (Y) is illustrated in Fig. 4. Note that relationship is independent of the pile diameters (b) and the values of ultimate soil resistance (P_u), and thus is representative of all the tests conducted.

The ultimate soil resistance expressed on a per unit length of pile basis (P_u) is made specific to a given pile size by multiplying the ultimate soil resistance (P_{uo}) available from the soil by

$$\frac{E_S}{E_i} = \frac{1}{1 + \left(\frac{Y}{Y_i}\right) \cdot \left[1 + a \cdot e^{-\left(\frac{Y}{Y_i}\right)}\right]} \quad [5]$$



[FIG. 4] Ratio of Secant Modulus to Initial Modulus (E_s/E_i) with Displacement (Y)

the pile diameter (b), as is shown in Equation 6. Further, it is this parameter that is degraded for a given load cycle number (N) with the correlation constant (C_N).

$$P_u = \frac{P_{uo} \cdot b}{1 + C_N \cdot \log(N)} \quad [6]$$

Where: b is (length),

C_N and N are dimensionless,

P_u results in (force / length)

The best fit of cyclic degradation to the two 30-inch (762 mm) diameter shaft cyclic load test data was obtained with correlation constant (C_N) equal to 0.24. The cyclic degradation term (denominator) reduces to 1 for N=1 (initial cycle, or static load). The value of C_N has a direct effect on the amount of cyclic degradation to the P-Y curve; a greater value of C_N will allow greater degradation of the P-Y curve. The degradation of P_u with increasing load cycle number (N) will also have the desired degradation effect built into the computation of the P-Y modulus values (E_i and E_s).

Both the initial modulus (E_i) and the secant modulus (E_s) are directly related to the pile diameter (b) as the ultimate soil resistance (P_{uo}) is multiplied by the by the pile diameter (b) to yield the soil resistance per unit length of pile (P_u), as was shown in Equation 6. It follows that the larger the pile diameter (b), the stiffer will be the lateral response. For a given ultimate soil strength expressed on a per unit area basis (P_{uo}) developing at a given rate (as defined by prescribing Y_i and *a*), a larger pile diameter (b) will engage a greater amount of soil per unit length of pile, and thus will generate more load per unit length of pile (P) at any given displacement (Y). This generation of more load per unit length of pile (P) at any given displacement (Y), by definition, results in a greater initial modulus (E_i) and secant modulus (E_s).

The Ultimate Soil resistance (P_{uo}) is the maximum amount of horizontal pressure that the soil can apply to the pile undergoing lateral deflections. In order to account for the passive soil wedge failure mechanism occurring near the ground surface of the laterally loaded pile, It is recommended herein that the value of the ultimate soil resistance (P_{uo}) be reduced by 50% at the ground surface and return to the full value at a depth of 2 pile diameters below the ground surface.

CPT (cone penetration testing) results were determined to be the most reliable means by which to determine the ultimate soil resistance (P_{uo}), and is made proportional to the cone tip resistance (q_c) herein by the CPT strength correlation constant (N_{CPT}), as is shown in Equation 7. Note that the parameter of ultimate soil resistance (P_{uo}) is dependent only upon soil strength.

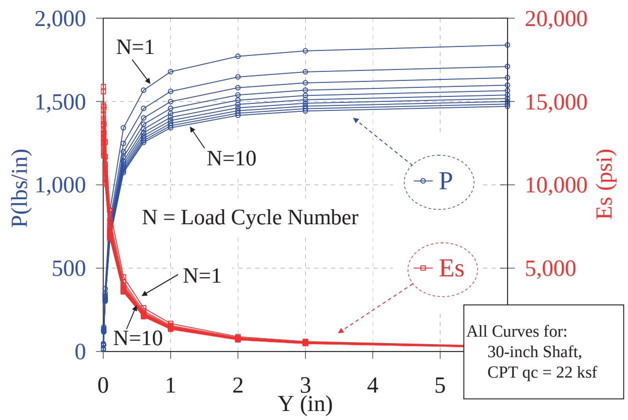
$$P_{uo} = N_{CPT} \cdot q_c \quad [7]$$

Where: N_{CPT} is dimensionless,

P_{uo}, and q_c are (force / length^2)

It was determined that the best fit of the model to the load test data was made using the cone tip resistance (q_c) profile that was previously shown in Fig. 1, and a CPT strength correlation constant (N_{CPT}) equal to 0.409. Correlations with other types of in situ testing may be developed similarly based on a cross correlation with cone tip resistance (q_c).

The static P-Y curve and the relationship of Secant modulus (E_s) with displacement (Y) for the 30-inch (762-mm) diameter shafts corresponding to the CPT tip resistance value (q_c) of 22 ksf (1053 kN/m^2) is shown in Fig. 5 as N=1, and is degraded with load cycle numbers (N = 2 through 10) for use in the cyclic load analyses. Families of curves generated for the other values of CPT tip resistance and for the 42-inch (1067 mm) diameter shafts were similar.



[FIG. 5] P-Y Curves and Secant Modulus (E_s), Static and Cyclic

STEP-BY-STEP PROCEDURES FOR GENERATING P-Y CURVES

(1) Develop an idealized profile of ultimate soil resistance (P_{uo}) with depth that is representative of the strata at the pile location. The idealized profile of ultimate soil resistance (P_{uo}) should be reduced by 50% at the soil surface, and allowed to return to full value at a depth equal to two pile diameters. While this ultimate soil resistance (P_{uo}) profile with depth can be determined by any representative strength testing (in-situ or lab), CPT testing is recommended using the cone tip resistance (q_c) in accordance with Equation 7. Note that it is most useful to break the idealized soil profile into layers wherein the cone tip resistance is either constant with depth or linearly varies with depth as these two conditions are easily accommodated by most lateral pile analyses software.

Conduct Steps 2 through 7 for the top and bottom of each idealized soil layer, noting that a spreadsheet is an ideal tool for these computationally intensive steps.

(2) Multiply the ultimate soil resistance (P_{uo}) by the pile diameter to obtain the ultimate soil resistance per unit length of shaft (P_u) in accordance with Equation 6. For cyclic analyses, this parameter (P_u) may be degraded for a given load cycle (N) with the correlation constant (C_N). Do NOT select the use of a P-Y multiplier in subsequent lateral analyses (Step 8) for purposes of cyclic degradation, as the cyclic degradation of the resulting P-Y curve is already taken into account by use of Equation 6.

(3) Select a reference displacement (Y_i) representative of the rate at which the resistance will develop. A recommended value was presented previously along with descriptions of its effect on the resulting P-Y curve. Alternatively, a value of the initial modulus (E_i) may be selected, and then value of the reference displacement (Y_i) made in Step (4) below.

(4) Determine the initial modulus (E_i) in accordance with Equation 2.

(5) Select a number of displacements (Y) for which a representative P-Y curve is to be generated. The largest of these values must be in excess of the displacements anticipated for that layer. Concentrate the data towards the smaller displacements to better define the P-Y curve where the secant modulus values (E_s) are changing quickly [for example this study used displacements of 0, 0.001, 0.003, 0.01, 0.03, 0.1, 0.3, 0.6, 1, 2, 3, and 6 inches (or 0, 0.025, 0.076, 0.25, 0.76, 2.5, 7.6, 15.2, 30.5, 76 and 152 mm)].

(6) Determine the secant modulus (E_s) in accordance with Equations 3 and 4 for each of the displacements in Step 5.

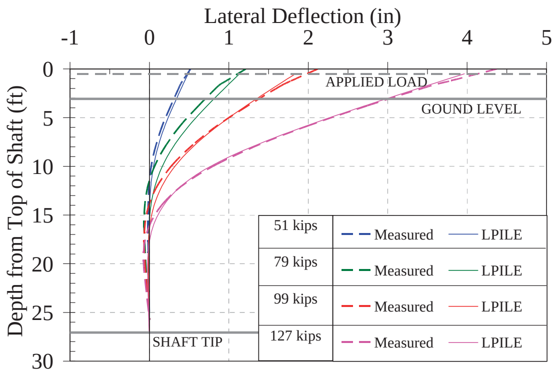
(7) Determine the soil resistance per unit length of pile (P) in accordance with Equation 1 for each of the displacements selected in Step 5.

(8) Using commercially available software, run a lateral load analyses using the "user specified P-Y curves" option. The P-Y curves are to be input, at every idealized soil layer, are the values of soil resistance per unit length of pile (P) obtained in Step 7 that correspond to the given displacement (Y) obtained in Step 5.

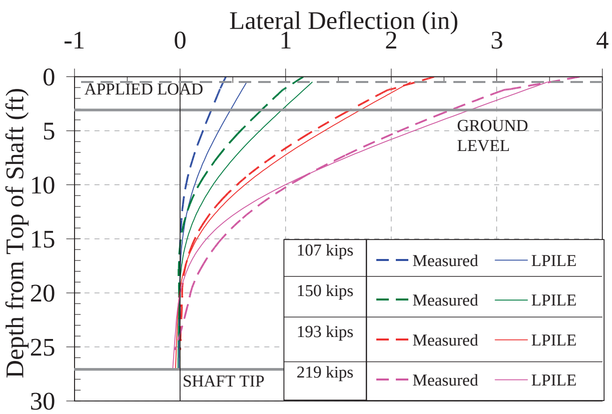
COMPARISON OF MEASURED RESULTS TO P-Y MODEL

The LPILE (v5.0) analyses of the test shafts were complicated by the nonlinear behavior of the reinforced concrete member which tended to crack and change in effective stiffness (EI) as bending occurred. This model included nonlinear EI as a function of bending by selection of LPILE "Analyses Type 3".

Static Results: The LPILE results are compared to the measured pile displacements with depth (shape) for the load increments where inclinometer soundings were performed in Figs. 6 and 7 using the average of the 30-inch and 42-inch (762-mm and 1067-mm) diameter pairs of static test shafts, respectively. These comparisons show good general agreement of pile displacement profile with depth over the wide rage of lateral loads tested (to geotechnical failure).

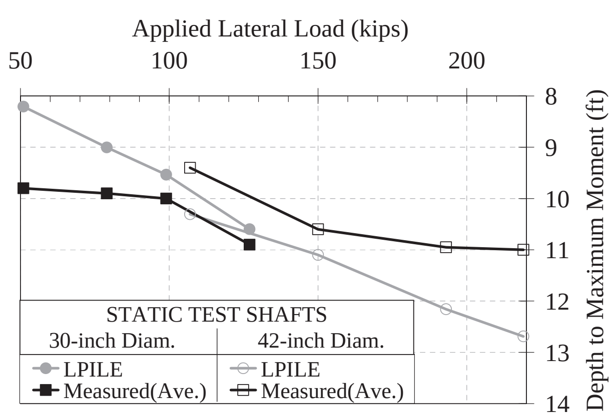


[FIG. 6] Average Pile Displacement Profiles for 30-Inch Diameter Static Test Shafts



[FIG. 7] Average Pile Displacement Profiles for 42-Inch Diameter Static Test Shafts

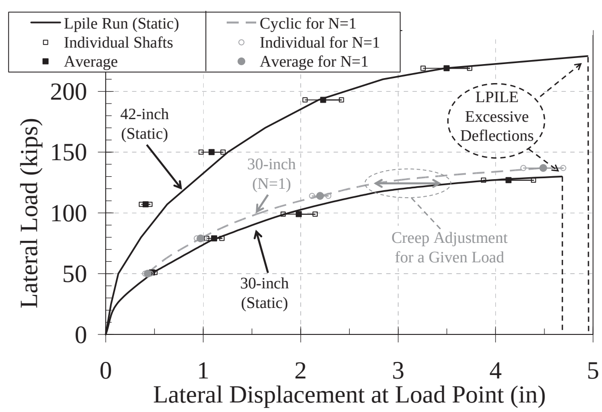
Comparisons of depth to maximum moment to that obtained from corresponding LPILE analyses in shown in Fig. 8 for the load increments where inclinometer soundings were performed. The depth to maximum moment of the test piles was determined as the point of maximum curvature. Although not directly



[FIG. 8] Depth to Maximum Moments

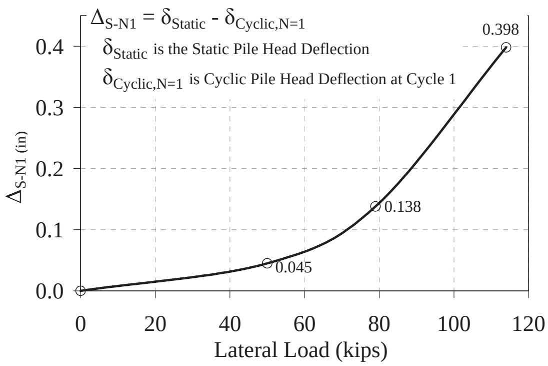
measured, the good agreement of depth to maximum moment between the measured data and the LPILE runs obtained indicated a good agreement in magnitude of the maximum moment.

The pile head lateral displacements (at the point of load application) are presented in Fig. 9. The Figure illustrates the good overall accuracy of the formulated P-Y response model used for the LPILE runs when compared to the measured static test data. The model appears to be slightly conservative for the larger 42-inch diameter shafts for predictions at pile head deflections under approximately 1 inch. The pile head displacements of the two 30-inch diameter cyclic test shafts, measured at the onset of the initial load cycle (N=1) for each of the four load increments, are also included in Fig. 9.



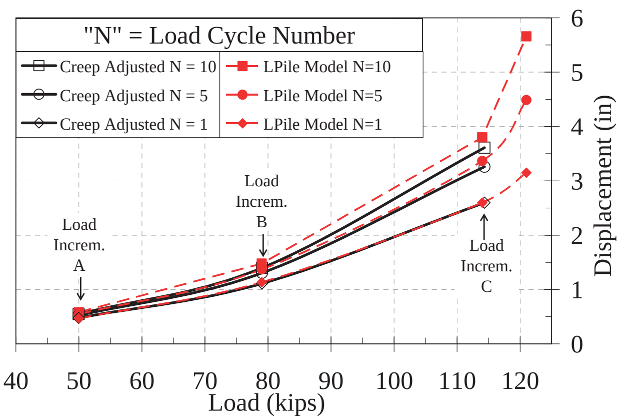
[FIG. 9] Top of Shaft Displacements

At any given lateral load, the displacement of the cyclic tests shafts (at the initial load cycle of N=1) indicate a smaller displacement than that of the static test shafts due to the load duration of the static test shafts being many times greater, resulting in a greater amount of soil creep. This difference between the statically tested 30-ich diameter shafts and the cyclically tested 30-inch (762-mm) diameter shafts (at the initial load cycle of N=1) is annotated as “Creep Adjustment for a Given Load” in Fig. 9. Further, the differences between the displacements become progressively greater with increasing lateral load. A relationship between the static and the cyclic (at the initial load cycle N=1) pile head displacements is obtained as the difference between these two curves varying with lateral load, and is shown as Fig. 10. This relationship will be utilized in subsequent presentation of the cyclic degradation results.



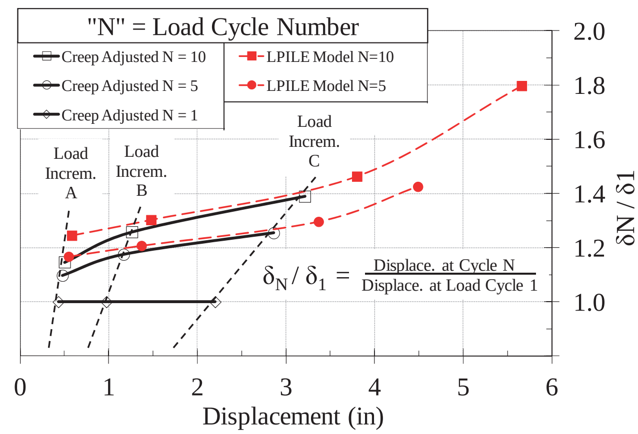
[FIG. 10] Pile Head Creep Relationship between the Static and the Initial Cyclic Load Cycle (N=1)

Cyclic Degradation Results: LPILE analyses results were compared to pile head displacement measurements (reported at the point of load application) made for all load cycles (N = 1 though 10) within all four load increments (A through D). The pile head displacements obtained with LPILE were consistently in excess of that which was measured during the cyclic testing. The LPILE model developed was based upon the static test shafts with longer hold times than was maintained for cyclic tests. When adjustments were made for creep, previously shown in Fig. 10, the pile head displacements obtained with LPILE are in good agreement as shown in Fig. 11.

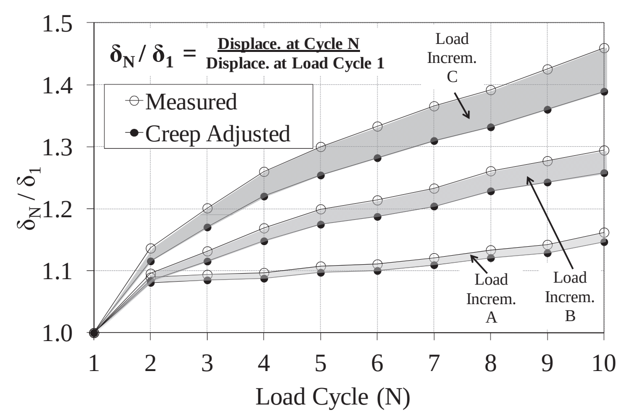


[FIG. 11] Creep Adjusted Pile Head Displacements vs. Lateral Load for 30-inch Diameter Cyclic Test Shafts

Comparisons of the ratio of the top of shaft deflection at displacements at the Nth cycle to the first cycle (δ_N / δ_1) are also shown vs. the top of shaft deflection at Load Cycle (N) in Fig. 12 for the creep adjusted deflections. Note that the line of data points at a given load increment are skewed at an angle to the ordinate, and further, the amount of skew increases with increasing load increments. This demonstrates that the increase in pile head deflection with load Cycle number (N) to have a more pronounced effect as the load is increased. This is best shown in Fig. 13 where the top of shaft deflection ratio (δ_N / δ_1) is shown vs. the load cycle number (N), noting the increase in angle relative to the ordinate with increasing load increment.



[FIG. 12] Creep Adjusted Pile Head Displacement Ratio (δ_N / δ_1) vs. Top of Shaft Displacement for 30-inch Diameter Cyclic Test Shafts



[FIG. 13] Pile Head Displacement Ratio (™$_N$ / ™$_1$) vs. Load Cycle Number (N) for 30-inch Diameter Cyclic Test Shafts

CONCLUSION

The method presented for generating hyperbolic P-Y curves are well suited for modeling both static and cyclic lateral loading in cemented soil (a "C-Φ" soil with both cohesive intercept and friction angle). The model makes a break from conventional p-y curve generation methodology in that for a given soil strength, a larger the pile diameter will result in a stiffer lateral response. This better matches the observed lateral response behavior, as well as making sense intuitively. Also, the smoothed curve that is produced will be algorithm friendly as there are no second order discontinuities, common to many other methods, that have the potential to produce convergence problems.

The relationships presented easily accommodate degradation of the static curve with load cycle number (up to N = 10). The degradation of the ultimate soil resistance per unit length of shaft will also have the desired degradation effect built into the computation of the P-Y modulus values.

The suggested values for the model parameters (including the strength parameter used correlated to the cone tip resistance from CPT testing) are well calibrated to the Loess soils at the test site, as evidenced by the good fit of the model analyses compared to the measured results for both the static and cyclic tests. A step-by-step procedure is presented for the use of the model to generate user specified P-Y curves for lateral analyses, and each parameters effect on the resulting p-y curve is discussed.

ACKNOWLEDGEMENTS

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