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Publication of this paper sponsored by Committee on Mechanics of Earth Masses and Layered Systems.

Assessment of Hybrid Model for Pile Groups

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The effects of mathematical modeling of pile groups by representing the soil response against the piles through unit-soil-resistance relationships for isolated piles and using elasticity methods to account for group effects were studied by modeling three pile-group tests in clay reported in the literature. Emphasis is placed on the effect of varying the Young's modulus of the soil surrounding the piles and the effects of imperfect pile alignment. The errors in computed cap translation were small; and the errors in load distribution to piles and the axial load distribution along the piles were insignificantly affected by the value of the modulus and could not be eliminated by this modeling method.

Mathematical modeling of pile groups is usually conducted with one or more of the following objectives:

1. Determination of the load-settlement behavior of the group as a whole for use in a superstructure analysis,
2. Calculation of the stresses within the piles to assess their structural integrity, and
3. Determination of the ultimate capacity of the group.

This paper is concerned with deterministic mathematical models that are addressed mainly to the first two objectives.

The objectives of this paper are

1. To demonstrate a concept that permits modeling of group effects in groups that have arbitrary geometries and six degrees of freedom at the pile cap,
2. To investigate the effects of the choice of assumed elastic parameters to represent the soil between and around the piles on the response of pile groups in clay, and
3. To investigate the effects of pile alignment on computed response.

The latter effects have been studied by modeling three well-documented load tests.

MATHEMATICAL MODELS

An ideal model is one that incorporates the nonlinear behavior of piles and soil, large deformations, the three-dimensional geometry and structural flexibility of the piles, and the effects of pile installation on the properties of the soil. Such a model is beyond the current state of the art, although reasonable attempts at modeling pile groups by using finite-element representations have been made (1, 2). More practically, static equilibrium models that represent piles as linear or nonlinear springs attached to a rigid or flexible cap have been used to model groups having more than one degree of freedom (3, 4). Groups having one or more uncoupled degrees of freedom have been modeled by techniques that envision piles as discrete-element elastic bodies embedded in an elastic-solid soil mass (5-7). The principal consideration in any of these models is the means of defining pile-soil interaction (e.g., the form of the spring relationships at the cap in the cap-spring model). In the cap-spring model, the effect of pile-soil interaction can be approximated in the working-load range by representing the piles as equivalent columns and cantilevers (4). More-precise analysis requires that pile-soil interaction be modeled more fundamentally. This can be done by modeling each individual pile as a discrete-element elastic body (8, 9) that is supported by independent, nonlinear axial, lateral, and torsional unit soil-reaction springs, whose properties can be obtained from published criteria developed for single piles in various soils. Unit soil-reaction relationship criteria include axial criteria (8, 10, 11), lateral criteria (12, 13), and torsional criteria (14).

In the elastic-solid model, pile-soil interaction in axial, lateral, and torsional loading is considered by means of elasticity theory (which has obvious limitations). This approach has the advantage that it considers the effect of load transferred from one discrete pile element to the soil on the load-deformation behavior at other elements directly. This effect is considered only indirectly (through use of criteria based on load tests where the effect occurred) in models that use unit soil-reaction curves. However, it is possible to track the complete nonlinear behavior of soil reaction by using unit-soil-reaction curves, whereas the use of elasticity theory to represent pile-soil interaction (even where modified to account for slippage at the pile-soil interface) gives a less-precise response.

The cap-spring model is incapable of considering pile-soil-pile interaction (group action) except by difficult, and often empirical, manual adjustment of the pile-head-spring relationships to account for assumed softening or stiffening effects in the various modes of loading.

Focht and Koch (15) have proposed a rational, approximate model for consideration of group action in groups of laterally loaded vertical piles that allows the use of unit-soil-resistance relationships in geometrically simple groups. This model can be classified as a hybrid model, because it combines salient features of elastic-solid and of cap-spring models. In essence, the hybrid model first uses unit-soil-reaction curves to represent the behavior of individual piles and then uses elastic methods to modify those curves for group action, based on the soil reactions against the piles computed in a noninteractive analysis (without consideration of group action). This concept has been extended to three-dimensional pile groups by O'Neill and others (16) by using several important algorithmic modifications. The hybrid model is a rational and reproducible way to model significant pile-soil interaction effects (including gap zones behind laterally loaded piles, soil degradation due to the cyclic loading, and nonlinear soil response) that are difficult to model by elasticity methods alone.

Figure 1. Modification of unit-soil-resistance curve at element on generic pile.

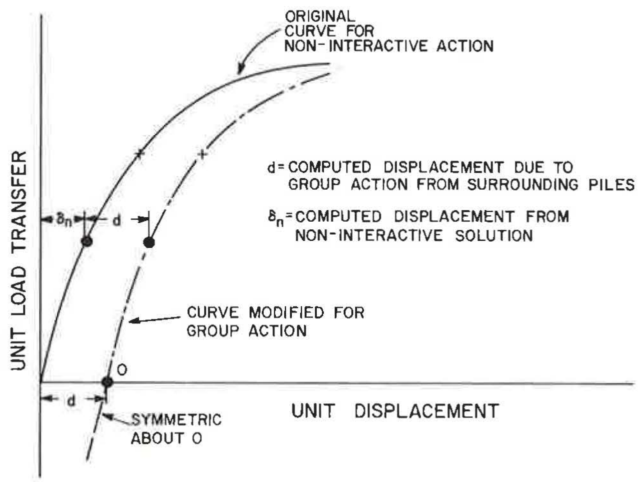
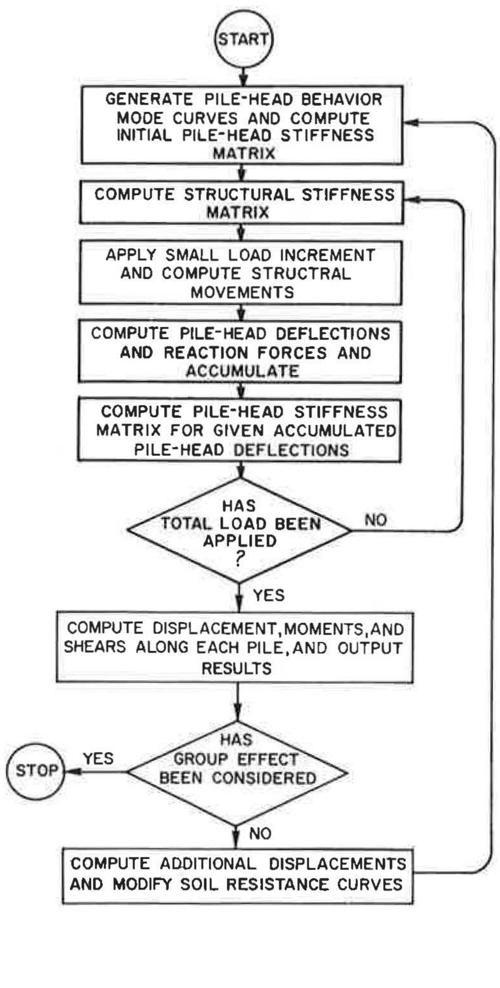


Figure 2. General flow diagram for hybrid algorithm.



MECHANICS OF HYBRID MODEL

The basic mechanics of the hybrid model are described in detail elsewhere (16) and are reviewed only briefly here. First, a noninteractive solution is made by using a nonlinear cap-spring model (3) that includes nonlinear cap-support spring relationships for the component piles obtained by modeling them as independent discrete-element bodies supported by nonlinear unit-soil-reaction springs. The soil reactions along all piles in the system are obtained from this solution. Then, pile-soil-pile interaction is considered by applying corrections to the unit-soil-reaction curves used in the noninteractive analysis (see Figure 1). This is accomplished for a generic pile by computing added displacements in three dimensions due to soil loads imposed from all other piles in the soil at the centers of the discrete elements along the generic pile. The added displacements are obtained from Mindlin's equations for displacement (7). The displacements so obtained are then transformed into displacements parallel and perpendicular (in two orthogonal directions) to the pile axis, and these are applied as offsets to the unit-soil-reaction curves. That is, group effects are accounted for by displacing the unit-soil-resistance curves for individual piles at various nodes along a pile an amount equal to the displacement that would have occurred in the soil mass at the node had the pile not been present but had the surrounding piles produced soil reactions identical to those obtained in the noninteractive analysis. It is assumed that the piles are spaced at a distance such that the added displacements are essentially elastic in nature.

The modified soil-reaction relationships are input back into the model used in the noninteractive analysis, and the entire solution is repeated, yielding a new set of displacements and loads at the pile heads and stresses and soil reactions along the piles that more nearly equal those in the real system. Progressively better solutions can be obtained through iteration. The analyses described below are those obtained after only one sequence of corrections. A general flow diagram of the computational scheme is shown in Figure 2.

The chief limitations of the current version of the hybrid model (although not limitations to the concept of hybrid analysis) are that (a) the interference of piles between an active pile (one whose soil loads are used to compute added displacements at a generic pile) and the generic pile is not directly considered; (b) if the elastic modulus of the soil mass varies with depth, pile-soil-pile interaction can be approximated only; (c) the pile cap is rigid and receives insignificant support from the subgrade; and (d) true ultimate capacity (plunging load) is constrained to be the sum of the ultimate capacities of the individual piles. The use of the model to predict pile-group response is most successful when pile installation does not produce significant changes in the in situ stress-strain properties of the soil mass, as would occur, for example, for a displacement pile group in initially loose sand.

The hybrid model is therefore nonrigorous because of the superposition of elastic displacements on nonlinear displacements obtained from unit soil-resistance relationships, but it is an efficient and practical systematic way to consider group action in three-dimensional pile groups.

EFFECT OF VARIABLES

The user selects the modulus of the soil for representation of group action independent of the soil properties implied for individual pile behavior. This permits mitigation (to some degree) of the well-known limitation of pile reinforcement of the elastic halfspace that represents the soil. The input modulus of the soil thus becomes a modulus for reinforced soil, which can best be evaluated by correlating computed results with field measurements. Such a process is not possible with the elastic-solid model.

It remains to evaluate the elastic modulus for use with the hybrid model. The effect of the modulus has been studied by using the following methodology.

1. Three well-documented load tests on two full-scale test groups in clay were modeled. The soils at the two test sites were similar: CL clays having a thin layer of desiccated soil overlying a softer, slightly preconsolidated soil. The pile tips floated at one site and were driven to rock at the other. The tests included a vertical load test on one group and a lateral and a combined lateral-vertical load test on the other.
2. For both test groups, single piles were tested separate from the group tests in the axial or lateral modes or both. The individual piles were modeled independently before the hybrid analysis was conducted by using unit-soil-reaction curves from published criteria and then adjusting the curves to produce computed load-deformation responses compatible with the measured responses in the single piles. The adjusted unit-soil-reaction curves were then used for each pile in the group analysis. Differences in calculated and measured group responses therefore should be strongly dependent on the choice of elastic modulus, which is the primary variable being investigated. All piles were divided into 50 discrete elements.
3. The groups were modeled by varying the Young's modulus of the soil mass while holding Poisson's ratio at a constant value representative of undrained conditions. Modeling was also conducted without considering group action. The results from the model were compared with measured values, where measured values were available, to assess the effects of varying the modulus and of neglecting pile-soil-pile interaction.
4. In each analysis, the modulus was related to the reported undrained cohesion of the soil as obtained by conducting Q-type compressive-strength tests on samples obtained by using routine sampling techniques, which result in partial disturbance of the samples.

One of the test groups was asymmetric due to pile drift that had occurred during driving. By modeling this group in its ideal geometric configuration and in its as-driven configuration, it was possible to investigate the effects of deviation from ideal geometry on the behavior of the group.

Test Group 1

Schlitt (17) has reported a vertical load test on a square group of nine friction piles in clay. Although the piles were nominally plumb and the load was applied only vertically, the test was modeled in three dimensions because one corner pile was driven to a depth considerably greater than the remaining piles, causing the pile-head surface to tilt during loading and produce lateral reactions against the piles. The group geometry and site conditions are shown in Figure 3.

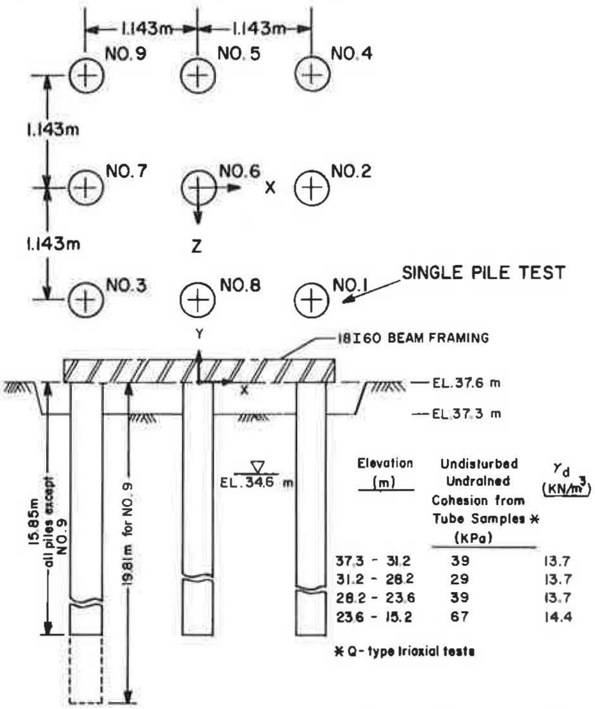
The piles were of the Monotube type, with butt diameters of 30.5 cm (12 in) and tip diameters of 20.3 cm (8 in). Load was applied to the group by jacks resting on an I-beam cap placed across the pile heads.

The profile of undrained cohesion of the clay is suggestive of slightly overconsolidated soils at a depth greater than 6 m (20 ft) below the ground surface and having more highly overconsolidated clay nearer the surface. Because in situ stress-strain data were not available for the test site, correlative methods were used to obtain the modulus (E) of the in situ soil. Ladd (18) implies that the secant modulus of undisturbed, normally consolidated clay at a stress level of 20 percent of failure in undrained triaxial shear is approximately 550-1000 times the undrained cohesion. It was assumed for purposes of modeling that the soil mass would be disturbed but would reconsolidate during the installation process to a condition near, but degraded from, the in situ condition and that the reinforcement effect of the piles would return the effective modulus to near its in situ value. Soil samples had been obtained by using tube samplers with an 18 percent area ratio and tested in Q-type triaxial compression. Because most of the soil was only slightly overconsolidated, the modulus of the soil mass was chosen within the range suggested by Ladd. One analysis was conducted at E = 23 100 kPa (3350 lbf/in^2) (570 times the average undrained cohesion of the soil from the surface to the pile tips), and one was conducted at E = 34 500 kPa (5000 lbf/in^2) (850 times the average measured undrained cohesion). A Poisson's ratio of 0.48 was used in both analyses.

The unit axial soil-resistance curves for modeling isolated pile behavior were developed by using the criteria of Vijayvergiya (11) (with lambda correlation), and the unit lateral and torsional curves were developed by using the criteria described by Reese and others (13) and by Dutt (14). Tip resistance was neglected.

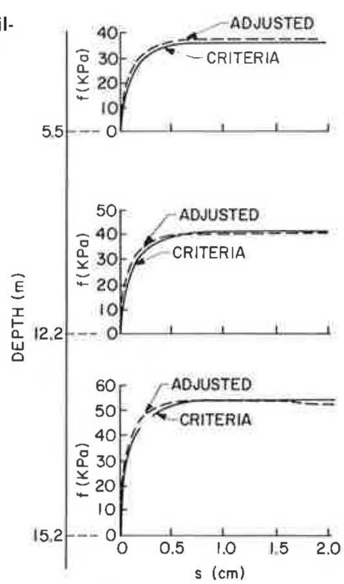
The precise form of the unit axial soil-resistance curves was varied until the computed load-settlement behavior of the isolated test pile (pile 1) reasonably matched the measured behavior under initial test loading. The unit-resistance (f versus s) curves obtained from the

Figure 3. Physical arrangement: test group 1.



NOTE: 1 m = 3.28 ft, 1 kPa = 0.145 lbf/in^2, 1 kN/m^3 = 6.37 lbf/ft^3

Figure 4. Axial unit-soil-resistance curves: test group 1.

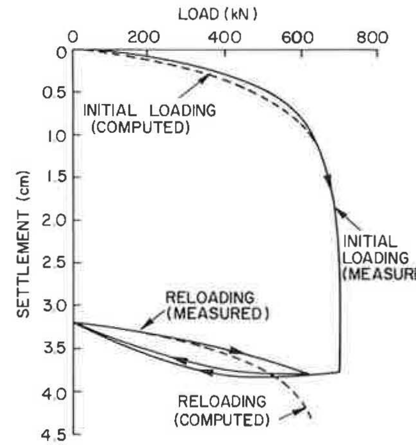


NOTE: 1 m = 3.28 ft, 1 cm = 0.394 in, 1 kPa = 0.145 lbf/in^2

criteria and those that were needed to replicate measured behavior are shown in Figure 4. The measured and computed load-settlement curves are shown in Figure 5, in which comparison to an immediate-reload test is also shown. Adjusted relationships were used. Computed settlements were somewhat too large for the immediate reload, possibly due to rebound effects in the test pile.

After pile 1 was tested, the remaining piles were driven and the group loaded to about one-half of the predicted failure load and unloaded. The group was then loaded a second time in increments to a load exceeding

Figure 5. Load-settlement curves for single pile: test group 1.



Note: 1 cm = 0.394 in; 1 kN = 225 lbf.

the required failure load. The second loading was modeled for applied loads of approximately one-third and two-thirds of predicted failure by using the adjusted unit-soil-reaction curves for initial loading, which are more appropriate than those for reloading of the single pile because the group was not failed during the first loading. Measured and computed pile-head loads and settlements are tabulated in Table 1. Measured and computed load-transfer relationships are compared for the center pile (6) and for an edge pile (7) in Figure 6 for E = 850 times undrained cohesion.

The effects of the choice of E on the computed distribution of loads to pile heads, group settlement, and validity of pattern of load transfer are described in Table 2. For distribution of load, the ratio of the SDs of the differences in computed and measured loads to mean pile-head load was selected as a measure of the accuracy of the model. For settlement, the effect of E is expressed by the percentage error of the difference between the computed and the measured mean pile-head deflections. For load-transfer pattern, the effect of E is expressed as the percentage error of the mean load transferred from pile to soil in the upper 9.2 m (30 ft) of embedment.

The variations in the measured load on the pile heads may be caused by (a) slight asymmetric positioning of load jacks coupled with the use of a flexible cap, (b) unknown variations in soil properties over the site, or (c) slight variations from the intended pile alignment that were not reported and hence are not modeled.

Pile 6, the middle pile, was the pile most affected by the flexibility of the cap, which led to a measured load greater than the modeled load. As shown in Figure 6, the greatest differences in computed and measured load transfer occurred near the top of the pile, where additional pile-head settlement due to dishing of the cap would have forced transfer of the excess load in a flexible pile.

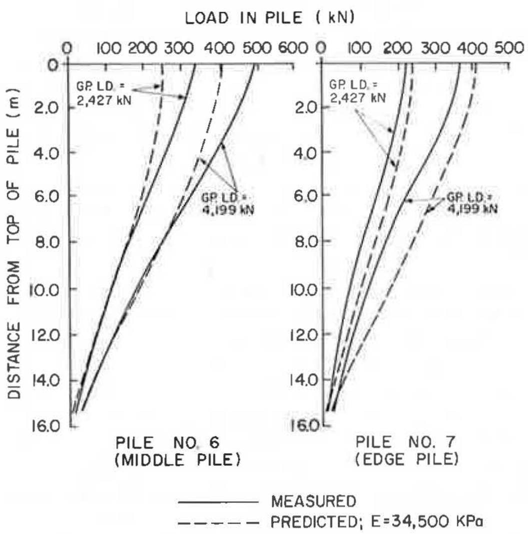
The effects of E for this test can be summarized as follows: (a) the prediction of group settlement was significantly better when pile-soil-pile interaction was considered, particularly where E = 850 times the undrained cohesion, (b) the inclusion of pile-soil-pile interaction improved the prediction of distribution of load to pile heads only slightly, and (c) the inclusion of pile-soil-pile interaction did not improve predictions of load-transfer relationships within the group. There were

Table 1. Individual pile-head loads (settlements): test group 1.

	Group Load = 2427 kN								Group Load = 4199 kN							
			Model Results								Model Results					
	Measured		E = 34 500 kPa		E = 23 100 kPa		E = ∞[a]		Measured		E = 34 500 kPa		E = 23 100 kPa		E = ∞[a]	
Pile No.	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)	Load (kN)	Settle-ment (cm)
1	267.0	0.48	290.90	0.46	295.88	0.58	269.94	0.20	529.6	0.99	513.98	1.04	532.40	1.42	467.70	0.41
2	345.3	0.48	261.66	0.46	260.55	0.56	269.71	0.20	562.5	1.04	452.65	1.02	439.30	1.37	466.81	0.41
3	262.5	0.36	279.26	0.43	279.53	0.56	269.54	0.20	405.0	0.81	492.66	1.02	494.95	1.32	465.96	0.38
4	290.1	0.51	279.26	0.43	279.53	0.56	269.54	0.20	502.0	1.07	492.66	1.02	494.95	1.32	465.96	0.38
5	186.9	0.51	250.80	0.43	245.64	0.53	269.31	0.18	364.0	1.02	413.81	1.02	406.18	1.27	465.11	0.38
6	331.1	0.56	248.09	0.43	243.94	0.56	269.54	0.20	491.3	1.17	406.86	1.02	404.06	1.32	465.11	0.38
7	224.3	0.43	250.80	0.43	245.64	0.53	269.31	0.18	365.8	0.89	413.81	1.02	406.18	1.27	465.11	0.38
8	192.2	0.48	261.66	0.46	260.55	0.56	269.71	0.20	401.4	0.91	452.65	1.02	439.30	1.37	466.81	0.41
9	327.5	0.36	304.60	0.43	315.77	0.51	270.43	0.18	577.6	0.71	559.94	1.02	581.70	1.22	469.60	0.38
Total	2427		2427		2427		2427		4199		4199		4199		4199	
Avg		0.46		0.44		0.56		0.20		0.97		1.02		1.32		0.39

Note: 1 kN = 225 lbf; 1 kPa = 0.145 lbf/in²; 1 cm = 0.39 in.
[a] I.e., pile-soil-pile interaction is neglected.

Figure 6. Measured and predicted axial load distribution: test group 1.



Note: 1 m = 3.28 ft; 1 kN = 225 lbf.

minor differences in these effects when the load level was varied.

Test Group 2

A group of H-piles has been tested by Kim and others (19) by a procedure in which lateral load was first applied and then lateral and vertical loads were applied simultaneously. The group is illustrated in Figure 7, which shows the planned configuration of the piles and the actual configuration as driven. The loads L (lateral) and A (vertical) were representative of working-load values. Soil conditions at the site consisted of overconsolidated clay to clay loam to a depth of 4 m (13 ft) and slightly overconsolidated clay and clay loam with gravel layers from 4.0 to 10.7 m (13 to 35 ft) underlain by limestone. The undrained cohesion values were determined as described above for test group 1.

Unit f-versus-s curves for isolated piles were developed as described above for test group 1 by assuming fixity of the tip in the limestone. Unit lateral resistance curves (p versus y) and torsional curves were obtained from Reese and others (13) and from Dutt (14). The axial and lateral curves are shown in Figure 8, from which axial and free-head lateral load-deformation curves were developed. The curves were then adjusted to produce pile-head load-deformation curves in the axial and lateral modes identical to the curves measured from the initial loading of isolated piles in these modes. Based on the reported cohesion values and the fact that most of the soil was not heavily overconsolidated, the following ratios of E to undrained cohesion were selected for modeling purposes: 250 and 750 with respect to cohesion of the surface layer (c_{s_L}), which may be more appropriate for a lateral load, and 250 and 750 with respect to the average cohesion (c_{Av}) of the entire clay profile, which may be more appropriate for combined axial and lateral loading. Poisson's ratio was taken as 0.48, and the underlying limestone was treated as a continuation of the overburden for purposes of calculating additional displacements, an assumption that is justified in light of the fact that imposed loads produced little axial or lateral soil reaction near the pile tips.

Table 2. Statistical parameters: test group 1.

	Effect of E		
E (kPa)	On Distribution of Load[a]	On Settlement[b]	On Load-Transfer Pattern[c]
34 500			
Group load = 2427 kN	20.7	5.6[d]	19.2[d]
Group load = 4199 kN	14.2	5.3	16.0[d]
23 100			
Group load = 2427 kN	20.6	22.2	24.4[d]
Group load = 4199 kN	14.3	36.8	22.7[d]
∞[e]			
Group load = 2427 kN	21.8	55.6[d]	15.2
Group load = 4199 kN	17.8	60.5[d]	4.8[d]

Note: 1 kPa = 0.145 lbf/in²; 1 kN = 225 lbf.
[a] Expressed as the ratio of the SDs of the differences in computed and measured loads to the mean pile-head load as percentage.
[b] Expressed as the percentage error of the difference between the computed and the measured mean pile-head deflections.
[c] Expressed as the percentage error of the mean load transferred from pile to soil in the upper 9.2 m (30 ft) of embedment.
[d] Computed value is less than measured value.
[e] I.e., pile-soil-pile interaction is neglected.

The hybrid model was used to compare the cap translations and the load distributions to piles for cases of both ideal and as-driven geometry. The results of the analyses, in which E was varied as indicated, are summarized in Tables 3, 4, and 5. Table 4 describes the effects of both E and pile geometry on the solution for three of the piles. While ideal geometry results are reported, the value of E giving the best overall results for cap translation was used in order to separate the geometric effects from the effect of modulus.

The percentage errors in the computed cap translations are summarized below (1 kN = 225 lbf).

	Lateral Load Test		Combined Load Test (L = 890 kN and A = 1922 kN)	
E	L = 890 kN	L = 445 kN	Lateral Component	Vertical Component
250 c_{SL} (as-driven)	56			
750 c_{SL} (as-driven)	0	3		
250 c_{AV} (as-driven)	91		73	64
750 c_{AV} (as-driven)	16	21	1	2
Neglected (ideal)	38		58	67
Neglected (as-driven)	31		51	67

(It should be noted that, for all cases in which the effect of E was neglected, the computed value was less than the measured value.)

1. For the lateral-load test, the best value of E in terms of correlation with cap translation for as-driven conditions was 750 c_{SL}. For the combined-load test, the best correlation was for E = 750 c_{AV}, although the errors associated with both values of E were small in the lateral-load test at 890 kN [200 000 lbf (200 kips)].

2. When the lateral load was 445 kN [100 000 lbf (100 kips)], the error in computed cap translation under as-driven conditions was approximately equal to that at 890 kN, which indicates that the choice of E is not significantly affected by the level of load within the working-load range.

Figure 7. Physical arrangement: test group 2.

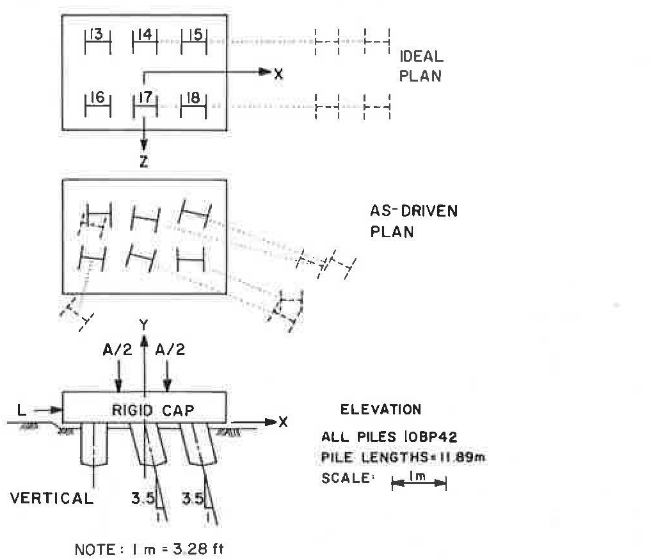


Figure 8. Axial (f versus s) and lateral (p versus y) unit-soil-resistance curves: test group 2.

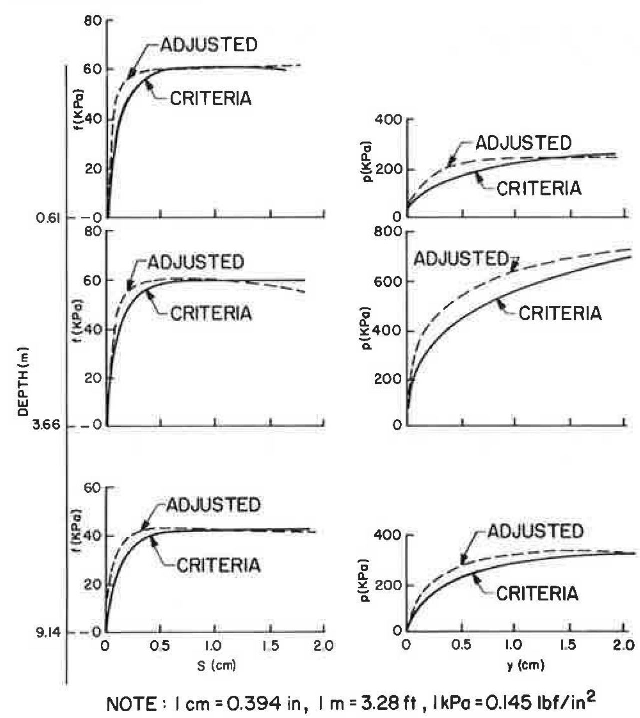


Table 3. Comparison of predicted and measured cap motion: test group 2.

Loading	E: Value (kPa)	E: With Respect to Cohesion	Translation (cm): X	Translation (cm): Y	Translation (cm): Z	Rotation (radians × 10^{-4}): X-axis	Rotation: Y-axis	Rotation: Z-axis
Lateral (L = 890 kN)								
Ideal geometry	1.55×10^6	250 c_{SL}	1.17	0.20	0.0	0.0	0.0	6.21
	4.65×10^6	750 c_{SL}	0.74	0.13	0.0	0.0	0.0	2.87
	1.03×10^6	250 c_{AV}	1.40	0.25	0.0	0.0	0.0	8.07
	3.10×10^6	750 c_{AV}	0.86	0.15	0.0	0.0	0.0	3.80
	∞^a		0.51	0.10	0.0	0.0	0.0	1.62
As-driven geometry		250 c_{SL}	1.27	0.18	0.005	1.88	0.76	3.18
		750 c_{SL}	0.81	0.10	0.005	0.02	0.28	0.72
		250 c_{AV}	1.55	0.20	0.003	1.65	0.83	4.63
		750 c_{AV}	0.94	0.13	0.005	1.91	0.43	1.35
	∞^a		0.56	0.08	0.008	2.10	0.10	-0.25
Measured			0.81					
Combined (L = 890 kN and A = 1922 kN)								
Ideal geometry		250 c_{AV}	0.79	-0.23	0.0	0.0	0.0	-4.31
		750 c_{AV}	0.57	-0.13	0.0	0.0	0.0	-2.79
	∞^a		0.28	-0.05	0.0	0.0	0.0	0.24
As-driven geometry		250 c_{AV}	1.17	-0.25	-0.13	-0.93	4.78	-12.85
		750 c_{AV}	0.69	-0.16	-0.10	-1.53	2.68	-6.29
	∞		0.33	-0.05	-0.08	-0.78	1.32	-1.05
Measured			0.68	-0.15				

Note: 1 kPa = 0.145 lbf/in^2; 1 cm = 0.39 in; 1 kN = 225 lbf.

[a] I.e., pile-soil-pile interaction is neglected.

Table 4. Comparison of modeled pile-head loads for three piles: test group 2 (L = 890 kN and A = 0).

Pile No.	E (with respect to cohesion)	Force (kN) U	Force (kN) V	Force (kN) W	Moment (kN·cm) U-Axis[a]	Moment (kN·cm) V-Axis[b]	Moment (kN·cm) W-Axis[c]
16	250 c_{SL}	-352.00	-1.78	115.26	-0.3	-13 179	54.3
	750 c_{SL}	-295.93	-0.89	121.04	-0.2	-11 563	28.3
	250 c_{AV}	-377.36	-2.23	112.59	-0.3	-13 948	-79.1
	750 c_{AV}	-313.28	-0.89	119.71	-0.2	-12 038	0.0
	750 c_{AV}[d]	-306.61	0.0	116.59	0.0	-11 789	0.0
17	250 c_{SL}	351.55	2.67	107.25	-0.1	-12 603	265.6
	750 c_{SL}	270.12	2.23	115.70	0.1	-11 201	226.1
	250 c_{AV}	388.04	3.12	105.02	-0.1	-13 360	276.9
	750 c_{AV}	296.82	2.67	112.14	0.1	-11 529	235.1
	750 c_{AV}[d]	265.67	0.0	109.92	0.0	-11 405	0.0
18	250 c_{SL}	24.92	2.67	115.26	-0.2	-13 179	262.2
	750 c_{SL}	55.18	2.23	121.04	0.0	-11 586	210.2
	250 c_{AV}	13.36	3.12	112.59	-0.2	-13 959	272.4
	750 c_{AV}	44.95	2.23	119.71	0.0	-12 049	240.8
	750 c_{AV}[d]	119.71	0.0	121.93	0.0	-12 286	0.0

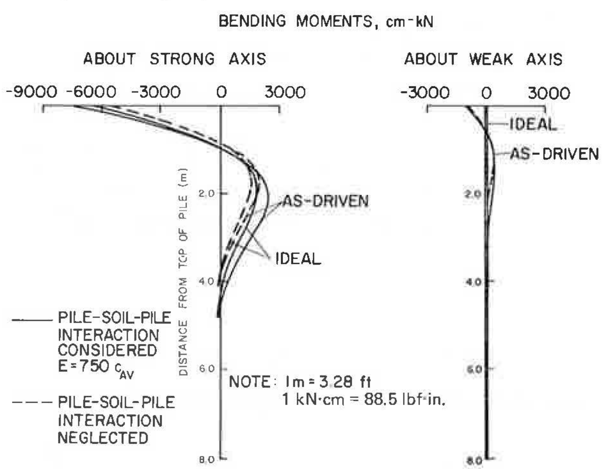
Note: 1 kN = 225 lbf; 1 cm·kN = 88.5 lbf·in.
[a] Coincides with pile axis, positive downwards.
[b] Normal to U-axis, parallel to strong axis of bending, positive in general negative Z-direction.
[c] Normal to U-axis and to strong axis of bending, positive in general positive X-direction.
[d] Ideal geometry.

Table 5. Comparison of modeled pile-head loads: test group 2 (L = 890 kN and A = 1922 kN).

Pile No.	E (with respect to cohesion)	Force (kN) U	Force (kN) V	Force (kN) W	Moment (kN·cm) U-Axis[a]	Moment (kN·cm) V-Axis[b]	Moment (kN·cm) W-Axis[c]
16	250 c_{AV}	117.48	8.46	79.21	-3.8	-8506.6	612.6
	750 c_{AV}	119.26	10.68	83.66	-2.0	-7701.9	669.1
	750 c_{AV}[d]	198.92	0.0	68.53	0.0	-6308.2	0.0
17	250 c_{AV}	380.48	15.58	90.78	-3.8	-9210.8	1199.2
	750 c_{AV}	389.82	16.02	90.34	-2.3	-7957.3	1085.1
	750 c_{AV}[d]	386.26	0.0	73.87	0.0	-6411.1	0.0
18	250 c_{AV}	404.51	23.59	89.89	-3.6	-9052.6	1703.4
	750 c_{AV}	400.95	21.36	91.23	-2.1	-8000.3	1382.4
	750 c_{AV}[d]	449.90	0.0	78.77	0.0	-6688.0	0.0

Note: 1 kN = 225 lbf; 1 cm·kN = 88.5 lbf·in.
[a] Coincides with pile axis, positive downwards.
[b] Normal to U-axis, parallel to strong axis of bending, positive in general negative Z-direction.
[c] Normal to U-axis and to strong axis of bending, positive in general positive X-direction.
[d] Ideal geometry.

Figure 9. Computed bending moments for pile 14: combined-loading case—test group 2.



The effect of failure to include as-driven geometry was measured as the ratio of the standard deviation of the difference between the pile-head loads computed from the ideal case (E = 750 c_{AV}) and the loads computed from the as-driven case to the mean absolute value of the loads computed for the as-driven case (expressed as a percentage).

Item	Lateral Load Test (L = 890 kN)	Combined Load Test (L = 890 kN and A = 1922 kN)
Axial load	16.2	17.6
Strong-axis shear	3.8	18.2
Strong-axis moment	4.0	18.3

[It should be noted that these values were computed for the following conditions: (a) the as-driven (best computed) value of E that gave the best agreement with the cap deflections (i.e., E = 750 c_{SL} for lateral and E = 750 c_{AV} for combined loads) and (b) the ideal value based on ideal geometry and E = 750 c_{AV}.] This effect is seen to be the strongest for axial pile-head loads in the lateral load tests and to be equally strong for axial loads, moments, and shears at the pile head for the combined load test. The probable error in computing these pile-head loads for any pile, using ideal geometry and combined loading, is about 12 percent.

The effects of pile-soil-pile interaction and of group geometry on the computed bending in a typical pile (pile 14) for the combined-load test is shown in Figure 9. Maximum moments about the weak axis were computed to be about 17 percent of those about the strong axis when as-driven geometry was considered. Furthermore, the inclusion of group action produced a computed maximum moment whose value was 14 percent higher and whose location was 33 percent farther down the pile than that calculated when group action was neglected in the as-driven case. Torsional moments were insignificant for all cases considered.

CONCLUSIONS

1. The hybrid model appears to be a useful tool for predicting load-deformation response of complex pile groups in normally to slightly overconsolidated clay that are loaded vertically, laterally, or by a combination of vertical and lateral loading.
2. The hybrid model can be employed by using published criteria for unit-soil-resistance curves, but the results are improved when the unit-soil-resistance curves for individual piles are adjusted by using the results of

load tests conducted on isolated piles near the group to be modeled that are loaded in a manner representative of group loading.

3. For both vertical and lateral loading in the cases studied, the best correlation between measured cap movement and equivalent soil modulus (E) occurred for E = 750 to 850 times the undrained cohesion of the soil (as indicated by Q-type laboratory tests on samples recovered by methods that do not attempt to minimize soil disturbance during sampling). This value represents, to some extent, the effect of pile reinforcement of the soil and may therefore be affected by pile spacing and total number of piles in the group. Use of E-values one-third too low in the axial tests increased the error in computed settlement by 17-32 percent, and use of E-values two-thirds too low in the lateral test increased the error in computed translation by 56 percent. The error in computed cap movement when the best value of E was used was relatively independent of the load direction and magnitude within the working-load range, even though the load-movement response of the cap was nonlinear for the axial, lateral, and combined load tests.

4. Use of the hybrid model did not significantly improve the computed axial load transfer along the piles or the distribution of loads to pile heads in comparison with the results obtained when group effects were neglected.

5. For test group 2, a probable error of 12 percent was inferred in computed pile-head thrusts and strong axis shears and moments when ideal group geometry, rather than as-driven geometry, was included under conditions of combined loading. Conclusions 4 and 5 have important implications with respect to the need for probabilistic modeling of pile groups.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.