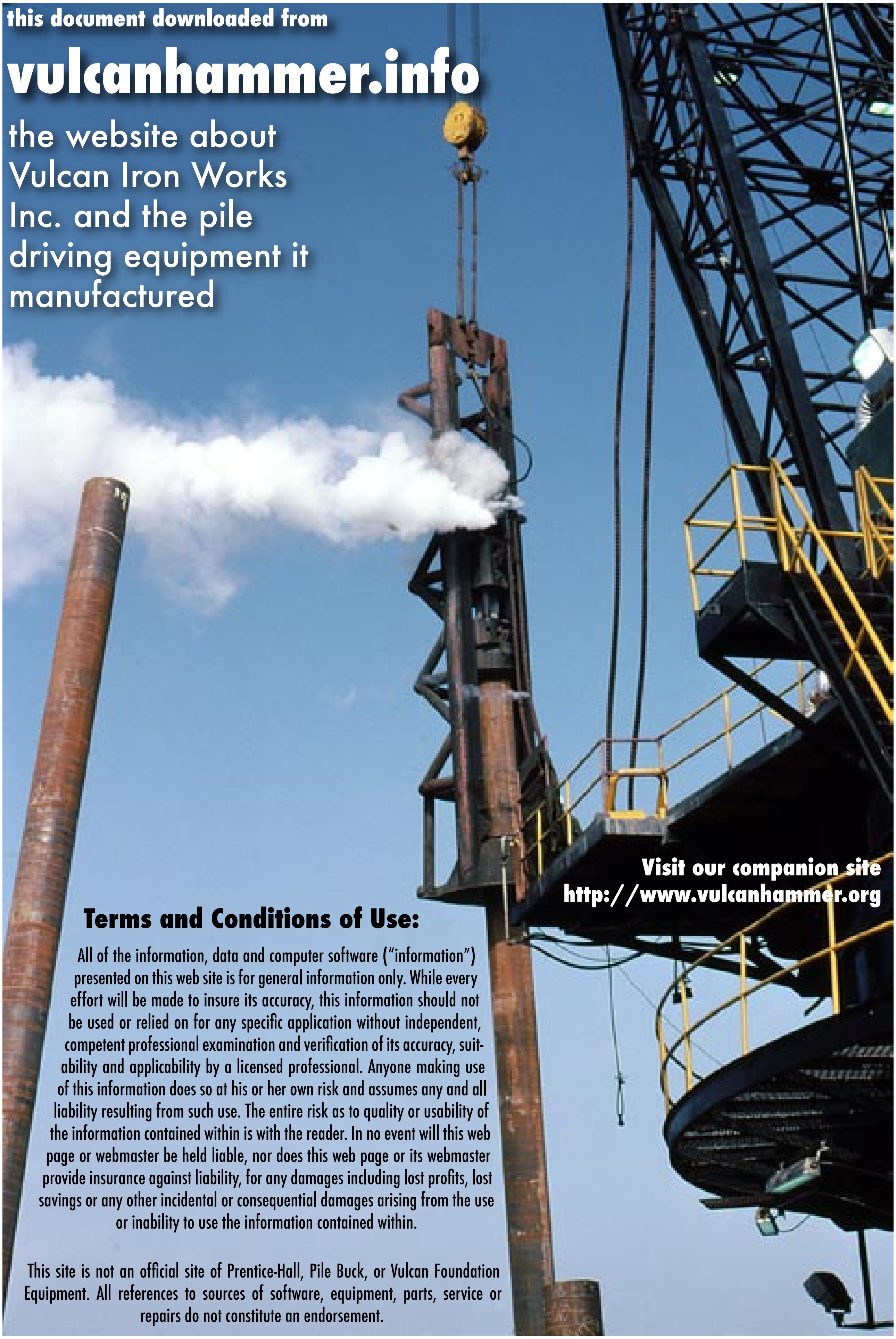

COMPARATIVE MODELING OF VERTICAL PILE GROUPS

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Abstract

The recent development of mathematical models for synthesizing load-deformation behavior of pile groups suggests the need for calibrating such models to field behavior. Two generic models described herein are used to model vertical load-deformation characteristics of five full-scale compression tests of pile groups in a variety of clay soils. Values of input parameters necessary to achieve reasonable compatibility with measurements are different in the two models. Those differences are explainable in terms of model mechanics. An extension of compression behavior to load-uplift behavior is described.

Introduction

Existing and proposed concepts for offshore platforms require foundation support in the form of pile groups subjected to both compression and uplift loading. Mathematical models for synthesizing group behavior generally fall into three categories: (a) finite element models (13), (b) elastic solid models (1, 14, 16), and (c) "hybrid" models (3, 4, 10, 19). Finite element models, while providing the most rational representation of the pile-soil system, are expensive to execute, require some empirical inputs, and must still be considered research tools. The elastic solid and hybrid models are simpler, although approximate, and have found their way into design use. They are the subject of this paper.

The principal pile group effects are (a) installation effects, in which the installation of a group of piles may progressively densify or destructure the mass of soil surrounding the piles, relieve residual stresses, and/or inhibit dissipation of excess pore pressures, and (b) mechanical effects, in which the effect of loading the piles influences

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response of neighboring piles. The former effect at present can be considered only empirically in the models, while the latter effect is handled in a rational way.

The structural designer is interested in (a) the stiffness of the pile group, which is of importance in static and dynamic stress analysis, and (b) the distribution of loads to the various piles in the group in order to determine the most advantageous geometric positions for the piles. These parameters are outputs of the two subject models. A third factor of importance, group efficiency, is not directly obtainable with either of the models and so must be determined externally.

The elastic solid and hybrid models are based on somewhat different premises, so that different inputs are required to obtain optimum output accuracy. Two computer programs representative of the models, DEFPIG (14, elastic solid) and PILGP1 (12, hybrid), have been used to conduct a comparative study of the two models. The methodology followed was to model the load-settlement behavior of five well-documented monotonic compression load tests on full-sized vertical pile groups by first modeling the response characteristics of an isolated pile at each test site, inputting the resulting single pile characterizations into the programs, and then varying soil stiffness inputs to obtain the output most nearly simulating measured group behavior.

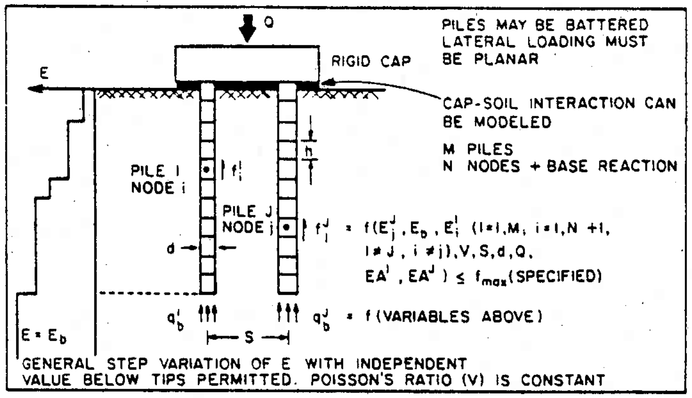
Model Description

Although the principles of both models have been described elsewhere (8, 9, 10, 11, 14), a brief comparative description using consistent notation is given below. Both models permit consideration of a depthwise variable soil modulus. Both can consider lateral pile response and multidirectional loading. DEFPIG treats cap-soil interaction; PILGP1 does not. Only those features pertaining to vertical behavior of vertical piles whose cap does not contact the soil are considered here. Schematic descriptions of the two models are shown in Fig. 1.

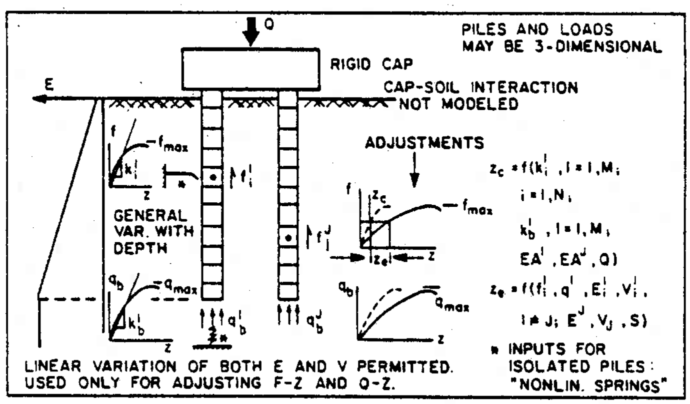
DEFPIG. In DEFPIG, a two-pile, pile-head interaction factor is defined:

$$\alpha_{IJ} = \frac{{}_{G}z_1^I}{z_1^I} \quad \ldots \quad (1)$$

where ${}_{G}z_1^I$ = settlement of the head (discrete element 1) of pile I



DEFPIG



PILGP1

Fig. 1. Comparison of Models Studied

(a generic pile) when pile I and a single neighboring pile J are loaded equally,

Z_1^I = settlement of pile I acting as an isolated pile under the same load as above.

In principle, for a soil characterized by a uniform Young's modulus:

$$\{Z^I\} = \frac{d}{E_s}\,[{}_sI^I]\cdot[[I] - \frac{n^2}{4(L/d)^2}K_a[{}_pI^I][{}_sI^I]]^{-1}\{Y\} \; . \; . \; . \; . \; . \; . \quad (2)$$

and $$\{{}_GZ^I\} = \frac{d}{E_s}\,[{}_sI_I^I + {}_sI_J^I]\cdot[[I] - \frac{n^2}{4(L/d)^2}K_a[{}_pI^I][{}_sI_I^I + {}_sI_J^I]]^{-1}\{Y\} \quad (3)$$

In Eqs. 2 and 3

d = pile diameter, L = pile length,

E_s = Young's modulus of soil,

n = number of discrete increments in a pile,

$[_pI^1]$ = $(n+1)^2$ pile compression stiffness coefficient matrix (8),

K_a = axial relative stiffness = $(E_p/E_s)\cdot(4A_p/\Pi d^2)$, where E_p = Young's modulus of pile material and A_p = cross-sectional area of pile material,

$[_sI^I]$ = $(n+1)^2$ matrix of interaction terms describing effect of soil stiffness on influence of load transferred from each element along pile I to soil displacements at all elements along same pile. Mindlin's equations of elasticity are used to derive terms (8),

$[_sI^J]$ = same as above, except describing the influence of load transferred from elements along neighboring pile J,

$[I]$ = $(n+1)^2$ identity matrix,

$\{Y\}$ = $(n+1)$ null vector, except $y_1 = Pn/\Pi d$, where P is load at head of pile I (may be unity for elastic analysis),

$\{Z^I\}$ = $(n+1)$ vector of displacements along pile I acting as isolated pile,

$\{_GZ^I\}$ = $(n+1)$ vector of displacements along pile I acting in two-pile group.

For a group with a rigid cap,

$$[A]\,\{P\} = \{Q\} \quad \ldots\ldots\ldots\ldots \quad (4)$$

where $[A]$ = $(m+1)^2$ matrix, where m = number of piles in group and

$a_{ii} = 1\ (i \neq m)$; $a_{mm} = 0$; $a_{ij}\ (i \neq j;\ i, j \neq m) = \alpha_{IJ}\ (= \alpha_{JI})$,

$a_{mj} = 1\ (m \neq j)$; $a_{jm} = -1\ (j \neq m)$,

$\{P\}$ = $(m+1)$ vector; first m terms are pile-head loads; $(m+1)$st term is group settlement ratio (settlement of group/settlement of isolated pile under group load Q/m),

$\{Q\}$ = $(m+1)$ load vector, where $q_i = 0$, $i \neq m$, and $q_m = Q$, the applied group load.

The pile-head settlements are computed from the settlement ratio.

The terms to the right of the dots in Eqs. 2 and 3 represent the vector of side shear, f, and end bearing, q_b, stresses for the various elements. When any term exceeds a preset limiting value, such as c_a

(adhesive strength), the term may be fixed at the limit and the calculations repeated until no computed pile-soil transfer stress exceeds the limit or until mathematical instability (indicating physical failure) is reached. In this way, DEFPIG can produce a nonlinear load-settlement curve, although nonlinearity does not begin until adhesive failure begins to progress along one or more piles.

PILGP1. PILGP1 assumes that isolated pile behavior is defined completely by the pile flexibility and by uncoupled unit load transfer curves, termed f-z and q_b-z curves, which are generally non-linear. Such curves must be obtained from load tests on isolated piles or from appropriate criteria. See Refs. 2, 6, 17, 20.

PILGP1 employs the following basic flexibility equation:

$$\{Z^I\} = [{}_{ps}I^I]^{-1}\{Y\} \quad \ldots \quad (5)$$

where $\{Z^I\}$ and $\{Y\}$ are as defined before and $[{}_{ps}I^I]$ is a banded pile-soil interaction matrix for an isolated pile I of the form

$$\begin{matrix} (k_1^I + s_1) & -s_1 & & 0 \cdots\cdots & 0 \\ s_2 & -({}^{\ell}k_2^I + 2s_2) & s_2 & 0 \cdots\cdots & 0 \\ 0 & s_3 & -({}^{\ell}k_3^I + 2s_3) & s_3 \cdots\cdots & 0 \\ \vdots & \vdots & \vdots & & \vdots \end{matrix}$$

In the above equation

${}^{\ell}k_i^I$ = secant to f-z or q_b-z curve for element i on Pile I for increment ℓ,

$$s_i = \left(\frac{E_p A_p}{\Pi d}\right)_i^I h^{-2}$$

For an arbitrary initial load increment, each k value is assumed to be equal to the initial slope of the f-z or q_b-z curve for element i, and the displacements are computed at all elements along the pile. Adjustments are made to k if displacement incompatability is found, and the solution is repeated until closure is achieved.

The load increment is then increased, the process is repeated, and a nonlinear load-settlement curve is generated. A load-settlement curve is synthesized for every structurally dissimilar pile in the group and/or for every pile having dissimilar f-z and q_b-z curves. Then, for an applied group load Q, the pile-head load distribution is obtained from

$$\{P\} = \sum_{\ell=1}^{\lambda} \{^{\ell}P\} \quad (6)$$

where $\{^{\ell}P\} = [B]^{-1} \{^{\ell}Q\}$ . . . . (7)

In the above equation λ is the number of load increments for Q (1 for a linear solution). Also, [B] is an $(m+1)^2$ flexibility matrix in which

$$b_{ii} = 1/^{\ell}C^{I} \ (i \neq m); \quad b_{mm} = 0$$

$$b_{ij} = 0 \ (i \neq j), \text{ except } b_{mj} = 1 \ (m \neq j); \quad b_{jm} = -1 \ (j \neq m)$$

where $^{\ell}C^{I}$ is the secant modulus to the first quadrant load-settlement curve for pile I for load increment ℓ. $\{^{\ell}P\}$ is the vector of incremental pile head forces for load increment ℓ. $\{^{\ell}Q\}$ is the incremental load vector, where $q_i = 0$ except for $i = m+1$, where $q_m = Q/\lambda$.

Equation 6 yields the distribution of loads to the pile heads, assuming no mechanical pile-soil-pile interaction. For each pile I in the system, Eq. 5 is then solved with the appropriate load boundary conditions, and deflections along each pile are computed. From the element deflections the f and q_b values along the piles are obtained. Mindlin's equations of elasticity are then used to compute the soil displacement at each element along every pile J in the group due to the distributed forces along every pile I (J ≠ I). The entire set of f-z and q_b-z curves for all piles is then reformulated such that at all values of stress

$$z = (z_c + z_e) \frac{z_o}{z_e} \quad (8)$$

where c refers to the value computed from Eq. 6, e refers to the elastic displacements computed from Mindlin's equations, and o refers to the original value for the isolated pile.

The entire process, beginning with Eq. 5, is repeated three times in PILGP1, although more repetitions may improve the solution, particularly where z_c lies in highly nonlinear portions of the unit load transfer curves. The resulting loads and displacements constitute the group solution, equivalent to Eq. 4 for DEFPIG.

Principal Differences

The principal differences in the models, in terms of their practical effects on output, can be summarized as follows:

1. In DEFPIG the elastic modulus controls both the single pile load-settlement behavior and mechanical pile-soil-pile interaction. In PILGP1 the elastic modulus controls only the latter effect, so that the modulus can be adjusted to account for pile reinforcement of the soil, provided proper "calibration" tests are available. The same adjustments can be made in DEFPIG, but doing so has a more significant effect on distribution of loads to the piles than in PILGP1.

2. At relatively low loads (before the first pile-soil slip occurs), PILGP1 should give a more uniform distribution of loads among pile loads than DEFPIG because the general nonlinear form of the unit load transfer curves permits the piles which attract the most load in an otherwise linear elastic analysis (e.g., corner piles in a square group) to relax and shed load back to the other piles. At loads approaching failure both models may underpredict settlement because the implied premise that the soil transmitting stresses between piles is elastic may no longer be valid.

3. Intuitively, load transfer curves (PILGP1) should represent soil response better than elastic-plastic relationships (DEFPIG), since they are predicated on a data base of load tests on instrumented piles and thereby include such effects as soil disturbance in the near field around a pile, residual driving stresses, effects of driving through stratified soils on the smear zone around piles, and soil compaction. The complex nature of these curves, however, restricts the advantage of PILGP1 and, in instances where such curves are improperly formulated, PILGP1 can produce misleading results.

4. Both models may be less effective in soils where installation of group piles produces general density changes or destructuring in the far field soil mass between and surrounding the group piles (as for pile groups in loose sands) than in soils where such changes do not occur.

Neither model is inherently superior to the other. Both have features which can be manipulated by the user to defeat some of the problems described. For example, the soil below the pile tips can be assigned a soil modulus independent of the modulus within the pile zone in DEFPIG. Increasing that modulus results in an increasing uniformity of pile-head loads. PILGP1 permits the same type of manipulation by varying the f-z and q_b-z curves. In the calibrations described below, however, a protocol was established whereby, for friction pile groups in clay, only uniform or uniformly varying moduli were permitted for both models and whereby only the unit load transfer curves measured on reference piles at the various test sites were used in PILGP1.

Model Comparisons

Traditional physical model tests do not simulate the effective stress states of soil in situ or at the soil-pile interface. Therefore, full-scale tests were chosen for the calibration process. Calibrations are limited to monotonic loading of pile groups in clay. Of the several sets of existing data for full-scale group tests in clay, five were chosen for this study. The five sets represent groupings of 3 to 9 floating piles, spacings of 3 to 4.5 diameters, and soils ranging from insensitive to sensitive and from lightly to heavily overconsolidated. Further, each group test had an associated instrumented isolated pile test and reliable data on distribution of loads among piles in the group. The various tests are depicted schematically on Fig. 2 (5, 12, 15, 18). In the University of Houston (UH) test series, two tests were conducted; first, on the entire 9-pile group and, second, on the 4-pile group indicated within the 9-pile group. Load test procedures varied among the tests, but in each case the isolated reference piles were tested according to the scheme used to test the group.

Each of the tests was modeled with DEFPIG and PILGP1 using the following methodology. Unit load transfer curves and uniform soil moduli appropriate for modeling single piles were obtained from tests on the isolated piles. See Fig. 3 . The site-specific curves were used as inputs for PILGP1 in lieu of curves obtained from published criteria so as to provide a positive check on the model. Trial soil moduli inputs for DEFPIG were estimated from hand solutions. For each test two modulus variations were investigated with both models (1) uniform and (2) linearly increasing from zero at the soil surface with depth. Two or more numerical values were then modeled for each pattern of variation. Poisson's ratio was assumed to be 0.5 throughout.

With respect to the linearly increasing modulus pattern, the two models assume different conditions beneath the pile tips. DEFPIG assumes that the soil modulus beneath the tips (base modulus) is constant and independent of the modulus along the shafts. Side friction inter-

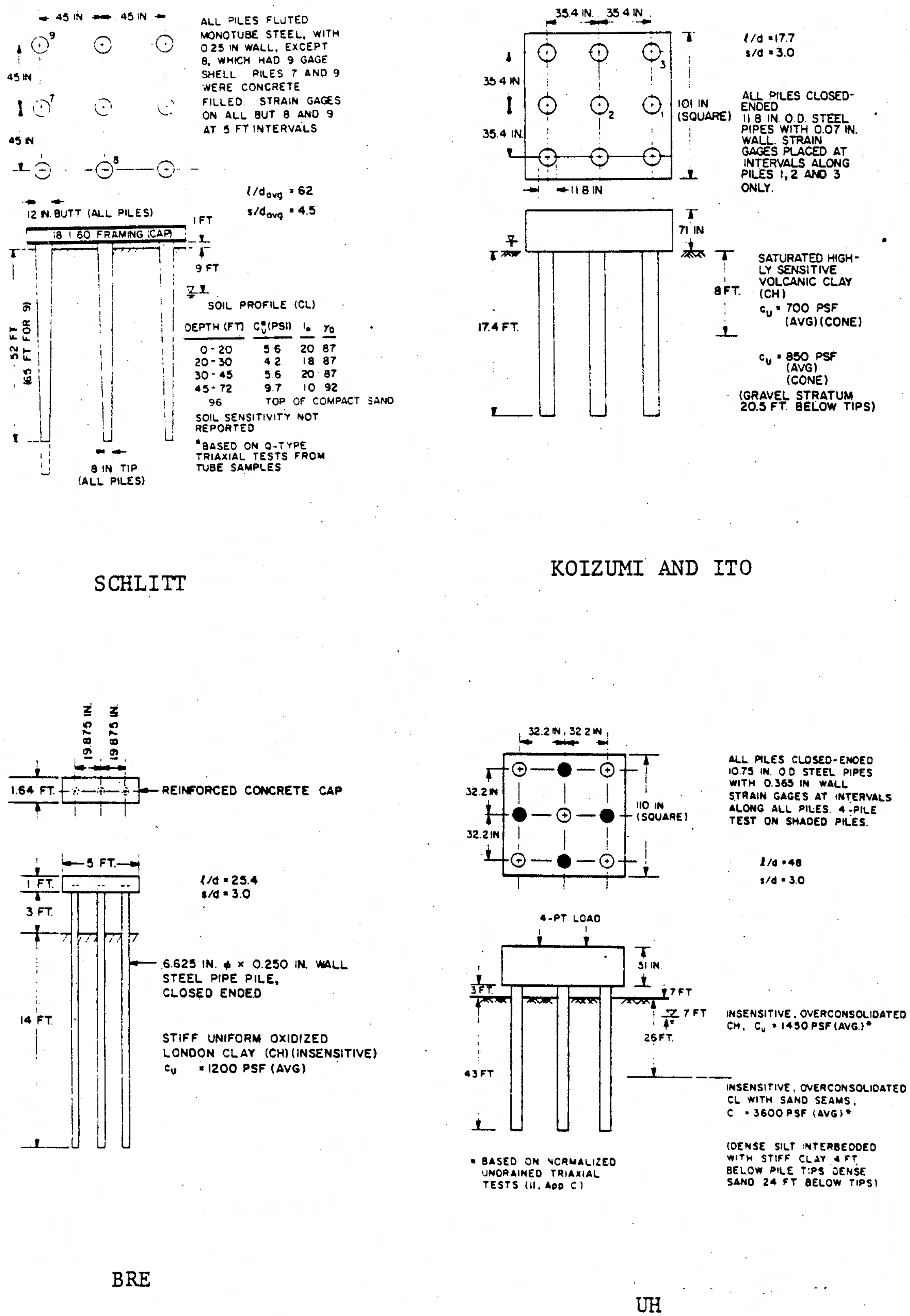


Fig. 2. Summary Data for Full-Scale Tests Modeled (1 in. = 25.4 mm; 1 ft = 0.305 m; 1 psi = 6.89 kPa; 1 psf = 48 Pa)

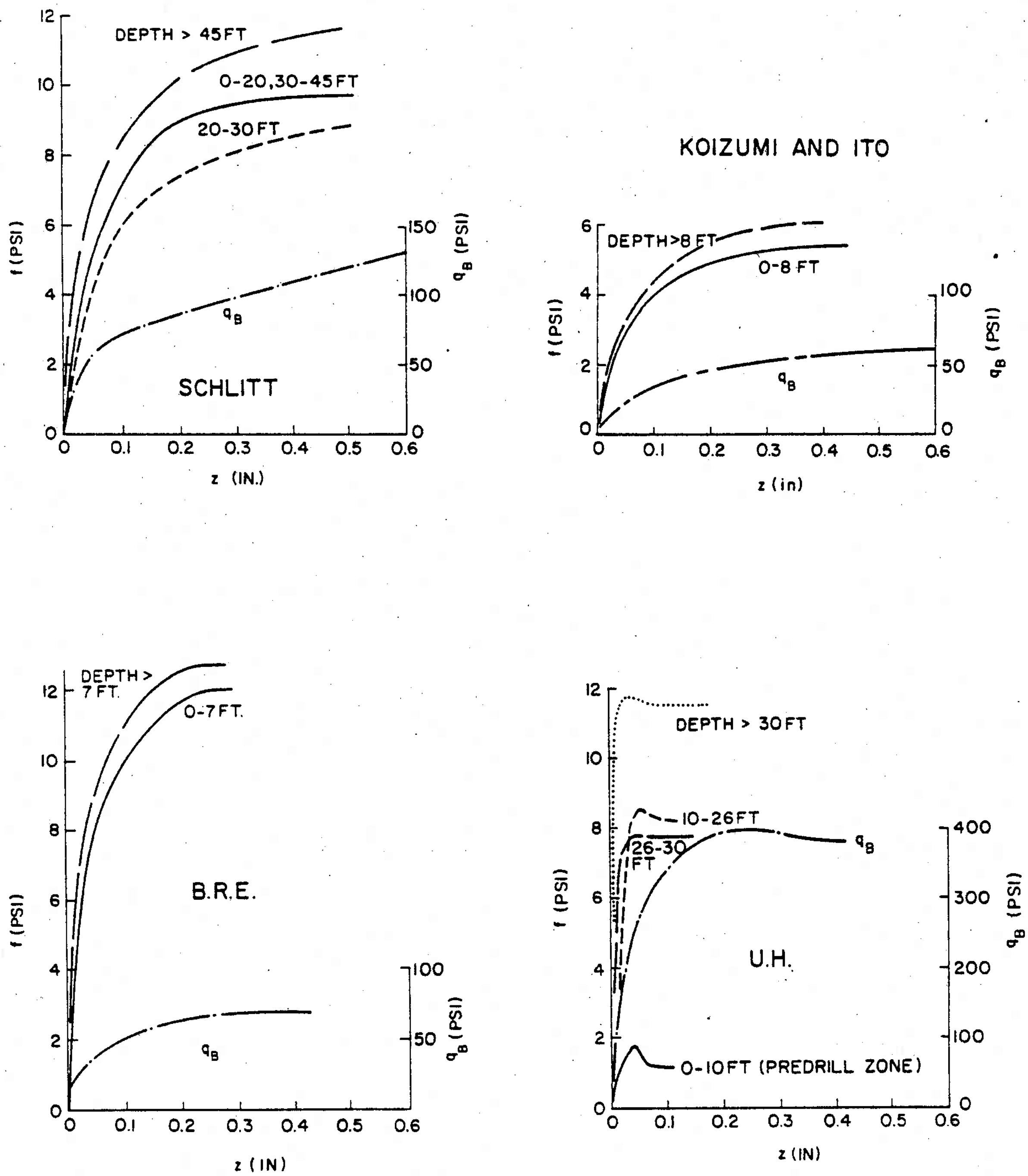


Fig. 3. Unit Load Transfer Curves from Reference Pile Tests
(1 in. = 25.4 mm; 1 ft = 0.305 m; 1 psi = 6.89 kPa)

action is influenced by the value of base modulus as well as by the variation of modulus along the shafts. For purposes of this study, which modeled floating pile groups underlain at some depth by stiffer soils, the soil modulus was varied from zero at the surface linearly to some value at the pile tips. Below that depth, the base modulus was assumed to equal the extrapolated linear modulus at a depth of 1.25 pile lengths. Below 1.5 lengths the soil was modeled as rigid. PILGP1 on the other hand, computes modifications of unit load transfer curves by using directly the average modulus between points Ii and Jj (Fig. 1), so that the pattern of modulus variation beneath the pile tips has no physical meaning and does not influence the solution.

The number of discrete elements was kept equal for both DEFPIG and PILGP1. Depending on pile length, 10 to 18 elements were used in every case. From 3 to 5 load values were applied to produce each load-settlement curve and related outputs.

The numerical results of the study are presented synoptically in Figs. 4 and 5 and in Table 1. Where measured settlement ratios fell between computed values, results were interpolated for purposes of making entries in Table 1.

The sum of CPU and I/O times for PILGP1 was about half of that for DEFPIG. However, preliminary runs had to be made with PILGP1 in this study to calibrate the unit load transfer curves given in Fig. 3 for single pile behavior, so that the overall computational effort with PILGP1 exceeded that for DEFPIG.

DEFPIG does not output distribution of loads along piles; PILGP1 does. Figure 6 compares distribution of load along piles in the UH 9-pile test computed from PILGP1 with measured values for a load approximating a static working load, using E values from Table 1. The comparison shown in Fig. 6 is typical of all tests modeled with the exception of the Koizumi and Ito test, where PILGP1 overpredicted rate of load transfer, especially in the center pile. The simultaneous close comparison of settlement ratio, distribution of load to pile heads, and distribution of load along piles suggests that PILGP1 is modeling the physical problem approximately correctly in the class of soil profiles considered here.

In the Koizumi and Ito tests, in which a completely rigid pile cap was used and where the piles were driven into a very sensitive clay,

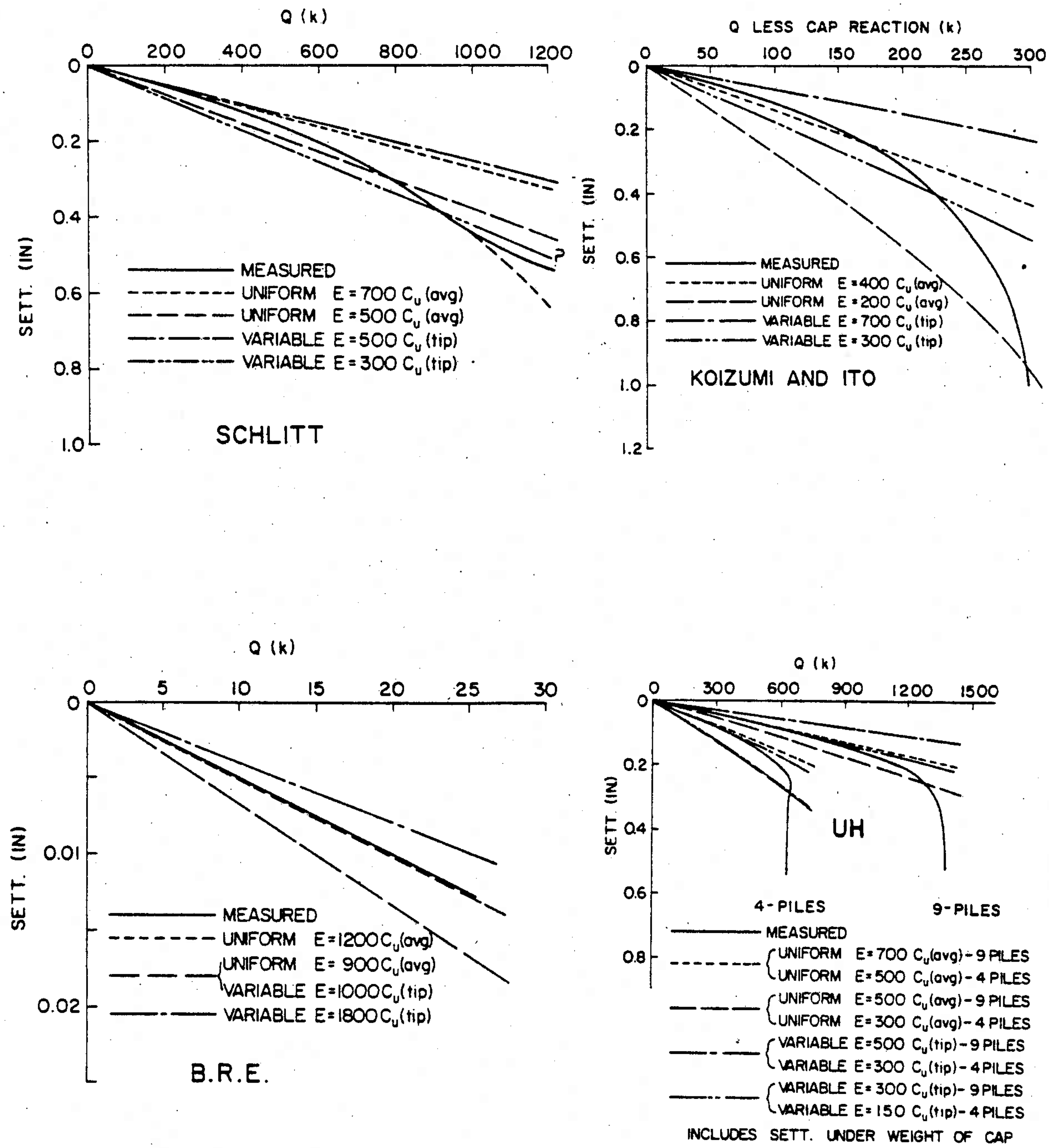


Fig. 4. Load-Settlement Curves; DEFPIG

(1 in. = 25.4 mm; 1 k = 4.45 kN)

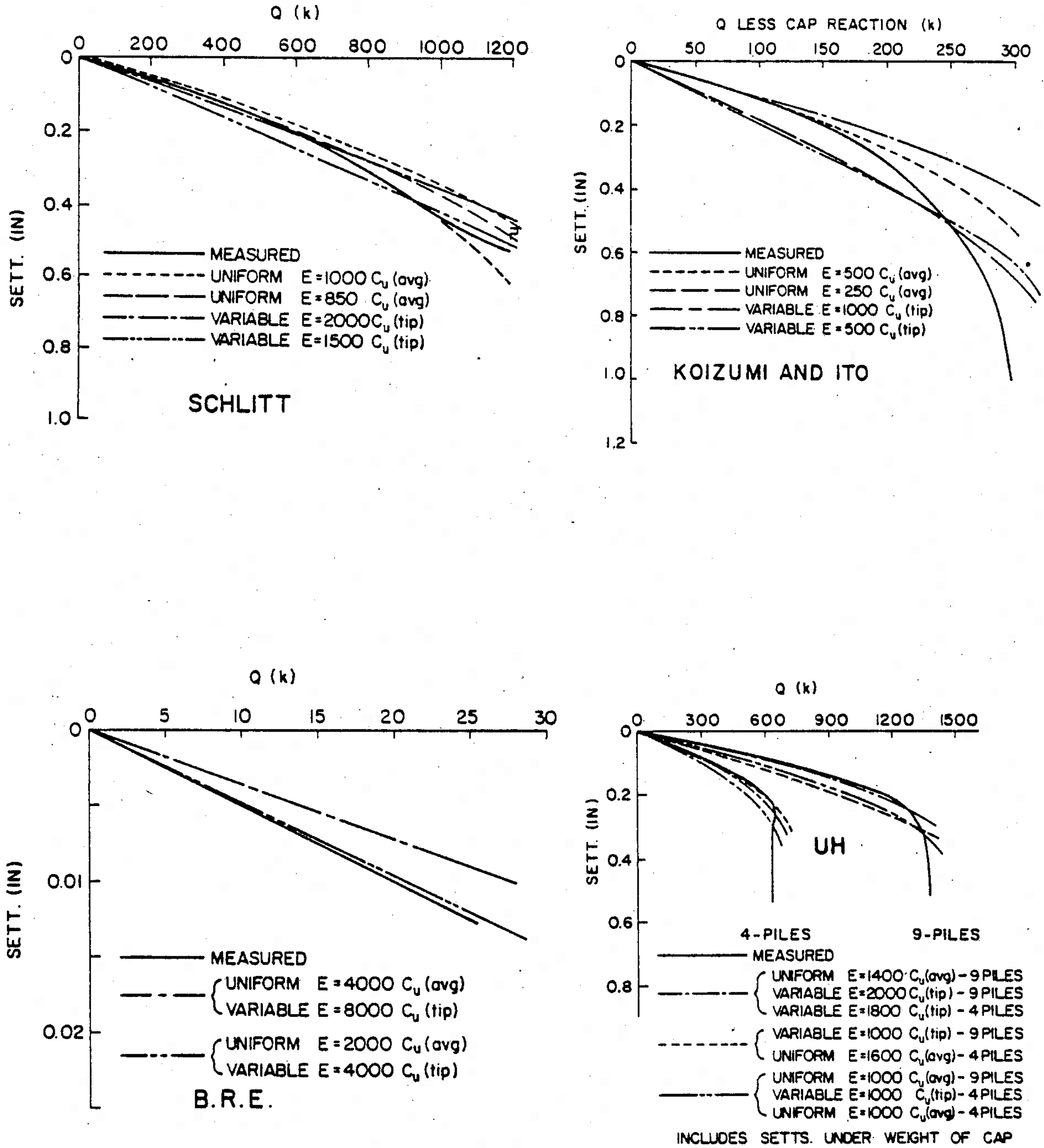


Fig. 5. Load-Settlement Curves; PILGP1
(1 in. = 25.4 mm; 1 k = 4.45 kN)

TABLE 1. TYPICAL RESULTS AT WORKING LOAD (1 k = 4.45 kN)

TEST/ MODEL	LOAD (k)	BEST - FIT MODULUS (X c_u)	SETTLEMENT RATIO		AVG. PILE-HEAD LOADS (k)					
					CENTER		EDGE		CORNER	
			M^1	C^2	M	C	M	C	M	C
SCHLITT	545									
DEFPIG		U; 600*	2.3	2.3	74	29	65	53	64	76
		V; 400	2.3	2.3	74	37	65	55	64	71
PILGP1		U; 850	2.3	2.3	74	57	65	59	64	63
		V; 2000	2.3	2.4	74	57	65	60	64	62
KOIZUMI-ITO	123									
DEFPIG		U; 400	3.3	3.8	7	4	14	11	19	18
		V; 500	3.3	3.3	7	8	14	12	19	17
PILGP1		U; 500	3.3	3.2	7	12	14	13	19	14
		V; 900	3.3	3.3	7	12	14	13	19	14
BRE	26									
DEFPIG		U; 1200	1.1	1.1	8	7	-	-	9	9
		V; 1500	1.1	1.1	8	8	-	-	9	9
PILGP1		U; 2000	1.1	1.1	8	8	-	-	9	9
		V; 4000	1.1	1.1	8	8	-	-	9	9
UH 9-PILE	581									
DEFPIG		U; 700	1.7	1.7	60	35	64	58	66	78
		V; 350	1.7	1.8	60	44	64	60	66	74
PILGP1		U; 1400	1.7	1.7	60	62	64	64	66	66
		V; 2000	1.7	1.7	60	62	64	64	66	66
UH 4-PILE	288									
DEFPIG		U; 700	1.2	1.2	-	-	-	-	72	72
		V; 300	1.2	1.2	-	-	-	-	72	72
PILGP1		U; 1600	1.2	1.2	-	-	-	-	72	72
		V; 1800	1.2	1.2	-	-	-	-	72	72

1 MEASURED 2 COMPUTED *U: uniform; V: variable; numerical value = appropriate E/c_u (avg: U; pile tips: V)

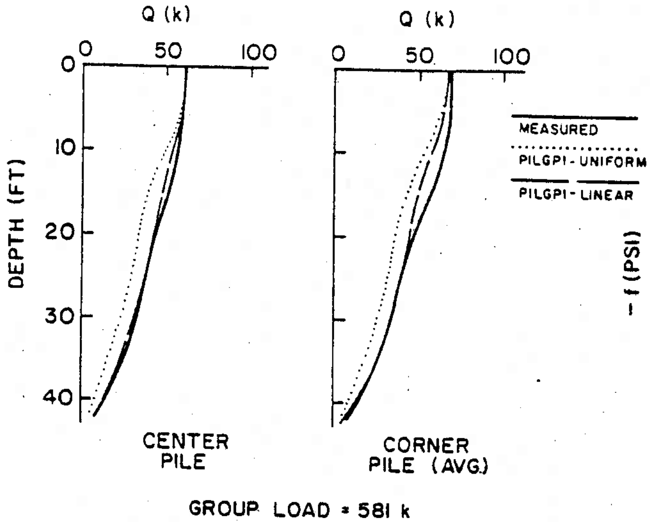


Fig. 6. Predicted Load Transfer; UH 9-Pile Test (1 ft. = 0.305 m; 1 k = 4.45 kN)

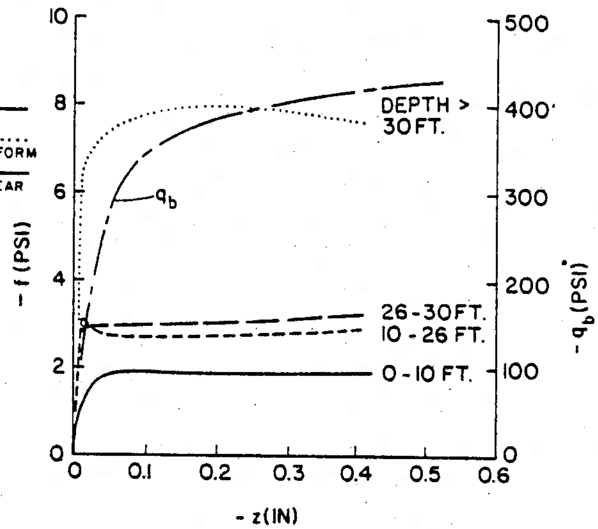


Fig. 7. Unit Load Transfer Curves in Uplift; UH Test (1 in. = 25.4 mm; 1 ft. = 0.305 m; 1 psi = 6.89 kPa)

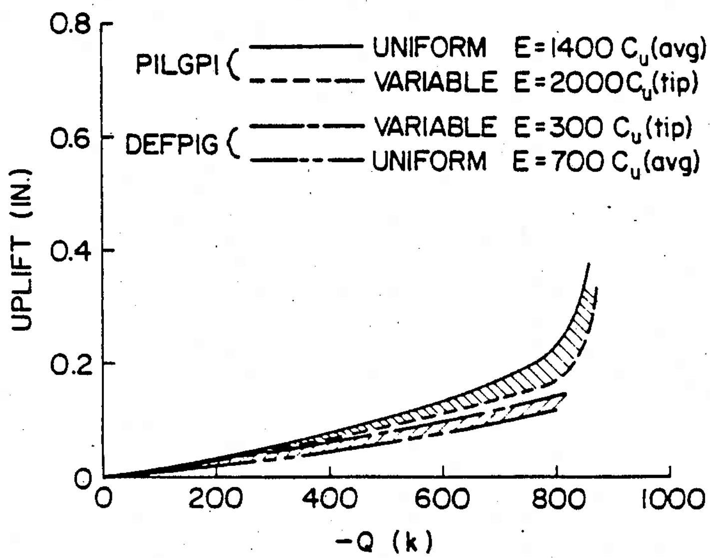


Fig. 8. Comparison of Predicted Load-Uplift Curves; UH 9-Pile Group (1 in. = 25.4 mm; 1 k = 4.45 kN)

DEFPIG gave superior results at working load. The response of the piles at the BRE and UH sites was very stiff, so that relatively minor flexibility in the caps could have produced more uniform pile-head loads than would have occurred in softer soils, resulting in an apparent practical (but not theoretical) advantage for PILGP1. The Schlitt test cap was semi-flexible, although the mode of loading (use of ballast beams) should have increased its rigidity. The pile response was also relatively stiff in the Schitt test, which was a test conducted after an initial cycle of load was applied.

The soil moduli from Table 1 can be expressed as follows for steel piles, as functions of measured undrained cohesion:

DEFPIG:

$$E/c_u\ (avg) = 3600\ (A_p/A_g)^{0.4}(c_u(tip)/c_u(avg))^{0.25} \quad \ldots\ldots \quad (9)$$

$$E/c_u\ (tip) = 2300\ \{\cos\ (1000\ A_p/A_g)\}^{-3}(L/d)^{-0.25} \quad \ldots\ldots \quad (10)$$

PILGP1:

$$E/c_u\ (avg) = 8100\ (A_p/A_s)^{0.5}(c_u(tip)/c_u(avg))^{0.5} \quad \ldots\ldots \quad (11)$$

$$E/c_u\ (tip) = 4800\ (A_p/A_s)^{0.4}(L/d)^{0.2} \quad \ldots\ldots \quad (12)$$

where A_p = cross-sectional area of pile material in the group (average for tapered piles); A_s = area of soil enclosed within perimeter of group; and A_g = total area (including piles) enclosed within perimeter of group. The argument of the cosine term is in degrees. The average and maximum (in parentheses) errors in the above equations in % are 10 (19), 18 (24), 6 (11), and 15 (25), respectively.

When applying these equations to future analyses, it should be realized that they are highly dependent on the measured undrained cohesion values and the manner in which those values are obtained. In each of the tests modeled the practical test method judged most likely to approximate in situ shear strength was adopted: UU triaxial tests on thin-walled tube samples in all cases except Koizumi and Ito and UH. Quasi-static cone values were used for Koizumi and Ito's test in very sensitive clay and normalized parameters (12) were used for the UH tests. In an offshore environment sampling disturbance may lead to lower indicated strengths, and the ratios in Eqs. 9-12 may need to be increased.

The range of E/c_u values for both algorithms is within that often assumed for undisturbed soils, suggesting that group behavior is controlled largely by behavior of minimally disturbed soil outside (below and laterally beyond) the immediate volume of the piles and enclosed soil, Particular attention needs to be paid, therefore, to the effects of the stiffness of the soil below the pile tips. These effects are handled in a rational way in DEFPIG, but they must be considered by artificially raising the modulus if necessary in the depth range of the piles in PILGP1. Furthermore, the soil modulus in PILGP1 is used only to model group action, not individual pile behavior, as is also necessary in DEFPIG, so that the modulus more strongly reflects the reinforcement (stiffening) of the soil by the piles in PILGP1. These two effects combine to require considerably higher moduli in PILGP1.

In all tests evidence exists that the in situ soil modulus increased with depth, in an approximately linear fashion. Somewhat closer correlations with pile-head load distributions were obtained with DEFPIG when the depthwise variable modulus was used, as might be expected in such soils. No major differences could be observed in distributions of loads to piles in PILGP1 with respect to the depthwise variation of the soil modulus, which reflects the inability of PILGP1 to model directly the stiffness of the soil below the pile tips. As seen in Fig. 6, however, use of the linearly variable modulus did improve prediction of load distribution along the piles.

In all but three cases the magnitude of the best-fit E/c_u at the tip for the variable modulus form was higher than the corresponding magnitude for the uniform modulus form. In three cases for DEFPIG the opposite effect occurred (Schlitt and both UH tests). In each of these cases the pile tips were in soil that was considerably stiffer than the overlying soil, so that stronger control of the settlement came from soil at or beneath the pile tips. Where such a condition occurs, the pile tips carry more load and the settlement ratio tends to be lower that when pile tips carry neglible loads, as when the group floats in a completely uniform soil (7).

<u>Uplift Behavior</u>

Little experimental data are available on pile groups under uplift loading. The topic is of increasing importance, however, because new offshore structure concepts use pile groups in tension. Both algorithms described can treat load-uplift behavior. Experimental unit load transfer curves are available in uplift from the reference piles for the UH tests (Fig. 7) so both models were used to synthesize load-uplift behavior of the UH 9-pile group. No physical uplift test was actually performed on that group.

The unit load transfer curves in Fig. 7 differ from those in Fig. 3 because of residual stress effects. In both figures, the curves are "apparent" relationships, in that a zero stress condition was assumed after installation in order to generate the curves. The base resistance curve in Fig. 7 represents a release of residual load that must be included in PILGP1. The reference piles had been failed in compression before uplift testing, so that the residual stresses may have been higher than had monotonic uplift testing been conducted directly after driving (12). Also, installation group effects exist with respect to residual stresses. In general, residual stresses are reduced in piles placed in groups (12), indicating that use of the post-compression-test unit load transfer curves from single, reference piles in PILGP1 with best-fit soil moduli from compression tests may overpredict pile rebound and thus yield a limiting (high displacement) condition for group load-uplift behavior. On the other hand, application of the best-fit moduli for compression testing directly to uplift loading in DEFPIG yields load-uplift curves that essentially mirror the load-settlement curves and may thus produce an opposite limit. Synthesized group load-uplift curves from both models are shown in Fig. 8. Considering the preceding comments, it appears that use of the variable modulus form with either model may produce the most realistic load-uplift curves for this site.

Conclusions

1. DEFPIG and PILGP1 require different soil modulus inputs to achieve reasonable compatability with measured load-settlement relationships for full-scale compression tests in clay.

2. Appropriate correlations for moduli are given in Eqs. 9-12 for the undrained loading condition.

3. DEFPIG modeled load distribution among piles better in relatively soft, sensitive soil where a massive cap was used. In stiff soils where a small degree of either soil nonlinearity or cap flexibility

existed, PILGP1 yielded better load distribution.

4. In modeling load-uplift behavior, unit load transfer curves from single pile uplift tests combined with soil moduli appropriate for modeling compression behavior may result in an overestimation of uplift magnitude with PILGP1. DEFPIG may underestimate uplift magnitude when moduli from compression calibrations are used.

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