PILE FOUNDATIONS : KNOW-HOW

How To Design Pile Groups

By J. Bres Eustis

DESIGNING a pile group is basically a problem of achieving a design to conform to two criteria. The first of these is adequate group bearing capacity, and the second is keeping ultimate settlement of the group within the tolerable limit for the structure. Determination of the required number of piles in the group and the design bearing capacity for the group is based on a knowledge of the load to be supported and the estimated allowable single pile load capacity, together with a judgment of the manner in which the pile group will transfer the structural load to the supporting soils. A realistic estimate of the ultimate settlement which the pile group will undergo also requires of proper judgment of the method of load transfer, as well as soil data concerning the consolidation characteristics of the underlying soils.

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With more than 33 years experience in the soil mechanics and foundation engineering field, 20 years of which have been in private practice, J. Bres Eustis has been a consulting soil and foundation engineer to architects and engineers, government agencies, industrial firms and the construction industry for a wide range of types and size structures. He personally supervises various technical and administrative phases of the company's soil and foundation investigation projects.

After receiving his B.E. in Chemical Engineering from Tulane University in 1934, Mr. Eustis spent three years progressively as soil laboratory technician and assistant to the Chief Technician — Soil Mechanics Laboratory, U.S. Waterways Experiment Station, Vicksburg, Miss. He was able to spend an interim period from 1934 to 1947 as a Junior Engineer on projects involving research and foundation studies and at M.I.T. studying Soil Mechanics. He received his M.S. in Civil Engineering (Soil Mechanics) from Harvard in 1941.

He is a fellow in the ASCE, a past member of the Board of Directors, Consulting Engineers Council of Louisana, and a member of the International Society of Soil Mechanics and Foundation Engineering. He is a registered Civil Engineer in Louisiana, Mississippi, Alabama, Texas, and Arkansas and a member of the Louisiana Engineering Society.

Capacity Of Pile Group

The methods for determining estimated allowable single pile load capacities for various pile embedments have been ably presented by writers of other articles in this series. These methods usually result in reasonably accurate predictions of pile load capacities which are rather easily verified by the driving of test piles and the performance of pile load tests on single piles. However, the determination of the load carrying capacity of any pile in a group and the total load carrying capacity of the group cannot be as accurately predicted, and is also more difficult to determine through full scale tests because of physical and economical limitations.

The bearing capacity of a group of piles which derive their load carrying capacity through bearing on rock or other hard strata can generally be taken as equal to the single pile load bearing capacity times the number of piles in the group. This is also true of those piles which may be driven through softer layers to a significant penetration into an underlying hard layer, and also for those piles that may be considered friction piles in cohesionless soils. All of these piles develop a substantial part of their total load carrying capacity on the lower portion of the pile and on the tip area itself. Any rational analysis of pile groups consisting of these types of piles will indicate the total load carrying capacity to be at least as great as the single pile load capacity times the number of piles in the group.

Thus, we have disposed of three of the four main categories of pile foundations by indicating that the group bearing capacity may be determined by a simple process of multiplication. This indicates the only real problem of determining pile group bearing capacity is concerned with piles driven into cohesive subsoils or into subsoils consisting of a heterogeneous system of silt, sand, and clay layers.

It is generally recognized that, for these piles, the load carrying capacity of a group is less than that of a single pile multiplied by the number of piles in the group. Therefore, these groups are considered less than 100 percent efficient; efficiency being defined as the total capacity of the group expressed as a percentage of the total of the isolated single pile capacity multiplied by the number of piles in the group. It should be noted that efficiency of the pile group is primarily a function of the soil characteristics rather than of the materials of which the piles are composed.

Formulae and Methods

Several formulae and methods for determining group efficiency are in use today. Those formulae usually found in building codes generally consider only the geometry of the pile group, with no consideration given to the physical properties of the supporting soils or to the length of pile embedment. Because of their inclusion in building codes, these formulae are often applied to conditions for which they are not applicable because the theories on which they are based do not apply to the particular design conditions.

Those formulae most commonly included in building codes are generally some variations of the Converse-Labarre Formula, which has given results that are usually considered reasonably adequate for evaluating the efficiency of a single pile incorporated in a group. A typical form of this formula is as follows:

$$G.E. = 1 - 57.3 \times \frac{d}{s} \times \frac{(n-1)\,m + (m-1)\,n}{90\,mn}$$

Where: G.E. = Group efficiency
d = Average diameter of pile
s = Spacing center to center of piles
n = Number of rows in group
m = Number of piles in row

This formula appears to be particularly applicable to friction piles driven into cohesive soil or into heterogeneous systems of silt, sand, and clay layers in which there is no significantly uniform trend toward increasing shear strength with depth. It is understood the conclusions reached through use of this formula have been verified in the past by at least one set of pile load tests. It may be interesting to cite another case with which the writer has personal experience.

Shown on Figure 1 are the logs of borings and the results of soil mechanics laboratory tests obtained during a subsoil investigation for a large industrial development in the New Orleans area. The heterogeneous conditions indicated by these logs and test results are typical of many parts of the New Orleans area. Borings were spaced at intervals of approximately 120 feet, and, although no real uniformity of subsoil conditions is indicated, the subsoil conditions may be described as consisting of five principal strata. The first has a variable thickness of 3.5 to 6.5 feet and consists of various layers of shell fill, medium stiff tan and gray clay, and loose to medium dense sandy silt, silty sand, and sand. Underlying this, the second principal stratum has a thickness ranging from 7.5 to 15 feet and consists of soft to medium stiff dark brown organic clay and gray clay containing humus, wood, and roots. Below this is a loose gray clayey silt, sandy silt, silty sand, and sand which extends to elevations of approximately -29 to -40, and has a thickness varying from about 17 to 25 feet.

Beginning at elevations ranging from -29 to -40 and extending to -51 and -63 is the fourth principal stratum of soft to medium stiff clay and sandy clay or loose to medium dense sand and clayey sand containing shell fragments. Underlying this to the final boring depths are medium stiff to stiff gray or tan and gray clay, silty clay, and sandy clay with some relatively thin layers of loose to medium dense clayey sand, silty sand, sand and clayey silt.

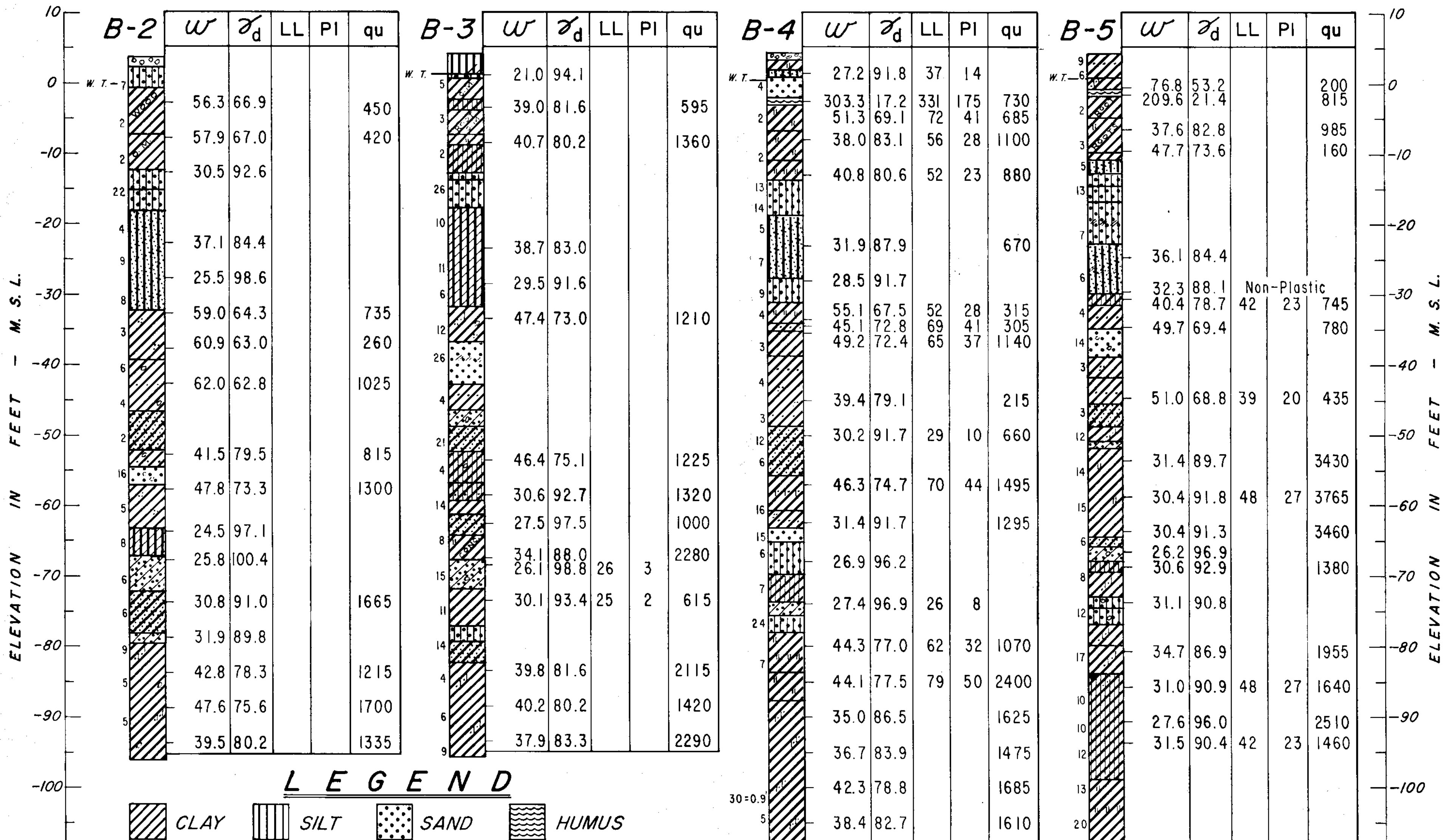
Observations indicated ground water level to be at a depth of 3 to 4 feet below existing ground surface.

The variable subsoil conditions encountered in both the vertical and horizontal dimensions make it difficult to accurately predict the load carrying capacity of even a single pile, and almost preclude the possibility of an accurate prediction of the load carrying capacity of a pile group. In this particular case, estimates of single pile load capacity were made based on several types of test data. The results of computations using the results of unconfined and confined compression shear tests performed in the laboratory on undisturbed samples obtained from the borings indicated that a pile driven to a tip embedment of 70 feet below existing ground surface would have an ultimate load carrying capacity of 50 to 60 tons. Computations based on the results of field miniature vane shear tests (not shown on Figure 1) indicated an ultimate capacity of 98 tons for this embedment. It has been our experience that the use of the field miniature vane tests usually results in unrealistically high predictions of single pile load capacity. Based on these computations, it was tentatively estimated that a 70-ft. pile would sustain an ultimate load on the order of 60 to 70 tons, and it was recommended that test piles be driven and a load test performed to verify this estimate. The use of the group efficiency formula contained in the New Orleans Building Code, similar to the one previously given, was recommended with the belief that it would give conservative results.

5-Pile Group — 290 tons

Because this particular project was a rather extensive development in which both the cost and the action of the pile foundations were major considerations, our client decided not only to verify the single pile load capacity by means of a pile load test, but also to evaluate the effect of group action by performing a full scale load test on a 5-pile group. The vicinity of Boring 2 was selected as the area for testing on the basis that the weaker subsoils were encountered in this area. The driving record for a typical test pile is shown on Figure 2. All test piles were allowed to set for approximately two weeks prior to test loading.

The results of the pile load tests performed by Shilstone Testing Laboratory of New Orleans, Louisiana, are shown on Figure 3. The test plot for the single pile indicates the yield point to be 78 tons, somewhat higher than the estimated capacity of 60 to 70 tons. Of particular interest are the results of the load test performed on the 5-pile group. This test plot indicates the yield point to be 290 tons, or a load of 58 tons on each pile in the group. Considering the single pile load



B-2

w	γ_d	LL	PI	qu
56.3	66.9			450
57.9	67.0			420
30.5	92.6			
37.1	84.4			
25.5	98.6			
59.0	64.3			735
60.9	63.0			260
62.0	62.8			1025
41.5	79.5			815
47.8	73.3			1300
24.5	97.1			
25.8	100.4			
30.8	91.0			1665
31.9	89.8			
42.8	78.3			1215
47.6	75.6			1700
39.5	80.2			1335

B-3

w	γ_d	LL	PI	qu
21.0	94.1			
39.0	81.6			595
40.7	80.2			1360
38.7	83.0			
29.5	91.6			
47.4	73.0			1210
46.4	75.1			1225
30.6	92.7			1320
27.5	97.5			1000
34.1	88.0			2280
26.1	98.8	26	3	
30.1	93.4	25	2	615
39.8	81.6			2115
40.2	80.2			1420
37.9	83.3			2290

B-4

w	γ_d	LL	PI	qu
27.2	91.8	37	14	
303.3	17.2	331	175	730
51.3	69.1	72	41	685
38.0	83.1	56	28	1100
40.8	80.6	52	23	880
31.9	87.9			670
28.5	91.7			
55.1	67.5	52	28	315
45.1	72.8	69	41	305
49.2	72.4	65	37	1140
39.4	79.1			215
30.2	91.7	29	10	660
46.3	74.7	70	44	1495
31.4	91.7			1295
26.9	96.2			
27.4	96.9	26	8	
44.3	77.0	62	32	1070
44.1	77.5	79	50	2400
35.0	86.5			1625
36.7	83.9			1475
42.3	78.8			1685
38.4	82.7			1610
43.1	77.5	66	39	1815
50.7	71.3	80	49	2710

B-5

w	γ_d	LL	PI	qu
76.8	53.2			200
209.6	21.4			815
37.6	82.8			985
47.7	73.6			160
36.1	84.4			
32.3	88.1	Non-Plastic		
40.4	78.7	42	23	745
49.7	69.4			780
51.0	68.8	39	20	435
31.4	89.7			3430
30.4	91.8	48	27	3765
30.4	91.3			3460
26.2	96.9			
30.6	92.9			1380
31.1	90.8			
34.7	86.9			1955
31.0	90.9	48	27	1640
27.6	96.0			2510
31.5	90.4	42	23	1460
35.8	86.3	64	45	1505

Predominant type shown heavy.

Modifying type shown light.

Figures on left side of borings indicate number of blows of 140-lb. hammer dropped 30-inches required to drive a 2-inch dia. split spoon sampler 1-foot after first being seated 6-inches (Standard Penetration Test).

W. T. – *Ground Water Table.*

LL – *Liquid Limit.*

PI – *Plasticity Index.*

w – *Water content in percent dry weight.*

γ_d – *Dry unit weight in pounds per cubic foot.*

qu – *Unconfined compressive strength in pounds per square foot.*

FIG. 1 - LOGS OF BORINGS AND LABORATORY TEST DATA

capacity of 78 tons, the efficiency of the group is approximately 75 percent. It is intersting to note that this is a very close check on the group efficiency that would be computed by means of the formula previously cited.

Although the Converse-Labarre Formula apparently gives reasonably adequate results when considering pile groups in cohesive soils or heterogeneous soil systems in which there is no significant trend toward increasing shear strength with depth, it is believed that a more realistic estimate of group capacity may be made for pile groups embedded in soils that do exhibit an increase in strength with depth. If adequate subsoil data is available for a proper analysis, it may often be found that such pile groups actually have an efficiency close to 100 percent. Thus, analysis by this method might permit the use of substantially smaller pile groups than those required by the usual building code efficiency formulae. For this type of subsoil condition, and if adequate subsoil data is available, an analysis may be made using the following formula adopted from a formula shown in "Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck.

$$Q_{ult} = (P \times L \times C) + 2.85\, q_u \left(1 + 0.3 \frac{w}{b}\right) \times A$$

Q_{ult} = Ultimate bearing capacity of pile group (lbs.)
P = Perimeter distance of pile group in feet
L = Length of pile in feet

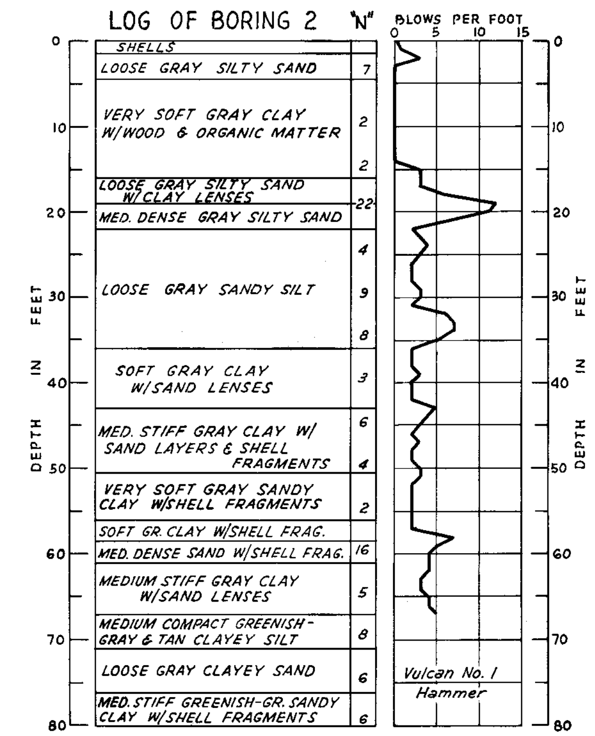


FIG. 2 - LOG OF BORING AND TEST PILE DRIVING RECORD

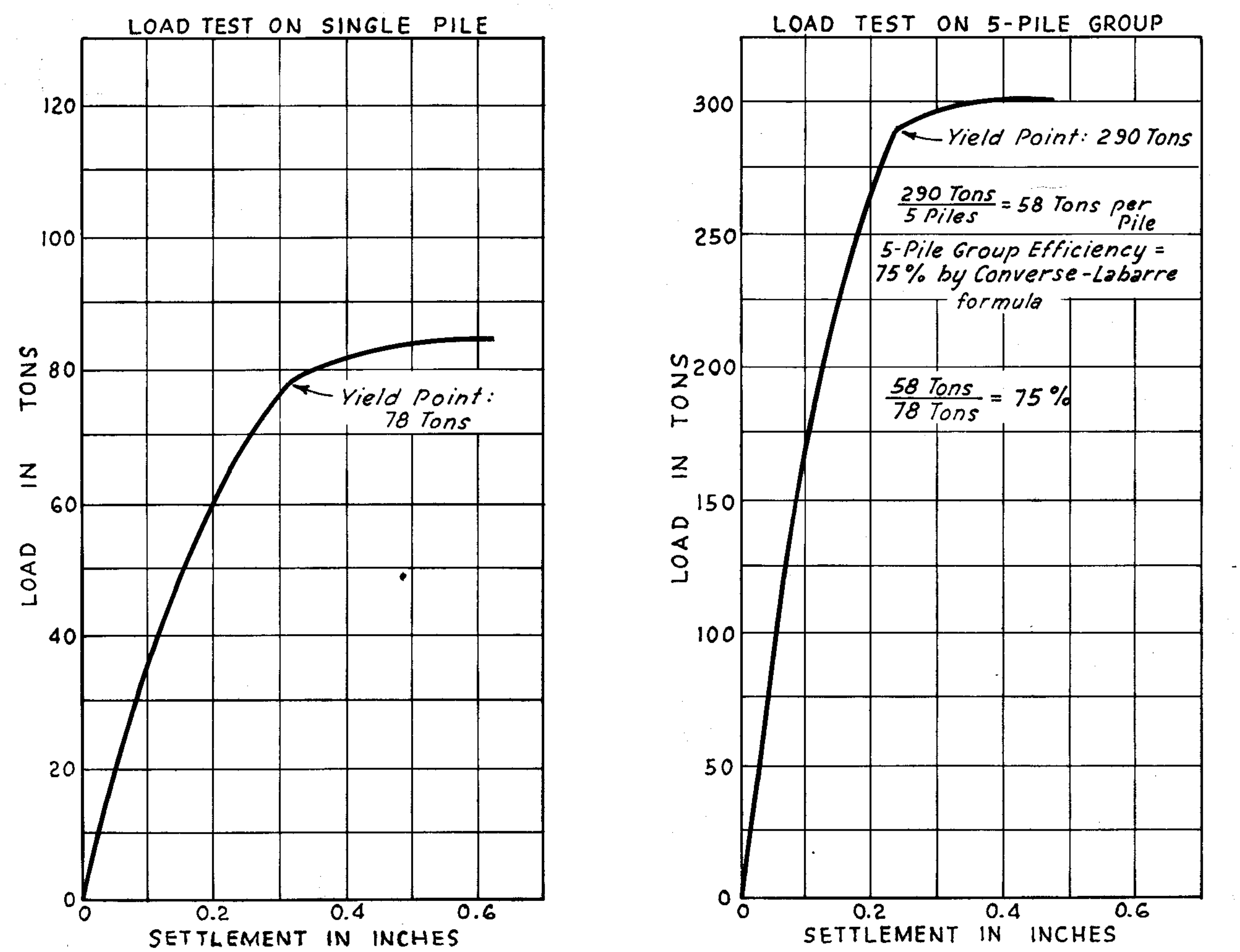


FIG. 3 - COMPARISON OF LOAD TEST RESULTS ON SINGLE PILE AND 5-PILE GROUP

C = Average (weighted) cohesion or shear strength of material between surface and depth of pile tips (lbs. per sq. ft.)
q_u = Unconfined compressive strength of material below pile tips (lbs. per sq. ft.)
w = Width of base of pile group in feet
b = Length of base of pile group in feet
A = Base area of pile group (sq. ft.)

Though this formula permits a more reasonable analysis based on known soil conditions, it is not generally permitted or included by most building codes. However, it should be noted that at least one code, that of the Parish of Jefferson, Louisiana, now permits this type of analysis. This building code includes the usual efficiency formula, and then states, "However, when subsoil shear strength data is available from soil borings and laboratory tests, and which indicates that piles will be embedded in increasingly firmer soils, the safety of a group of piles may be investigated on the basis of 'perimeter shear' or a possible group failure into the underlying soils. When it is shown by the Soils Engineer Report, that based on design pile spacing and loads, a factor of safety of 3.0 exists against a group failure by this method, no reduction for efficiency is necessary."

Settlement Of Pile Groups

The settlement of a pile group has little or no relation to the settlement of a single pile. Actually, the conventional efficiency formulae are also an attempt to recognize that the settlement of a pile group is greater than that of a single pile. Inasmuch as the magnitude of the settlement of a pile group is dependent on the size of the group, the length of the piles and the type of soil in the foundation, estimates of settlement should be determined by conventional soil mechanics engineering procedures utilizing the results of laboratory tests on undisturbed samples of the foundation soils. However, some general statement regarding the type of pile support and relative magnitude of settlement may permit an approximate evaluation of the settlement of specific types of pile groups. If the piles are primarily end bearing in a stratum of sand or gravel with no underlying clay, the effect of the size of the pile group would be relatively insignificant and settlements usually very small. However, where such a stratum is underlain by clay and the load imposed on the clay is greater than its preconsolidation pressure, the magnitude of the settlement may be large and will increase with increasing size of the group for a given spacing and load per pile.

In a deep clay foundation, where friction piles are used, increasing the length of the piles will reduce the settlement for a given foundation area and load per pile. For a given pile length and load per pile, the settlement will increase with the size of the loaded area. Where piles are to be driven into cohesive soils, or will be underlain by cohesive soils, analyses should be made to determine the type and length of piles and size and shape of the foundation which will result in a minimum of settlement. For wide foundations, long heavily loaded piles widely spaced, will result in less settlement than short lightly loaded piles, closely spaced.

I would like to credit Curtis L. Lundstrom of our engineering staff for assistance in the preparation of this article. ■