The Design of Deep Foundations in Expansive Soils in Semi-Arid Areas

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Summary.—Evidence is briefly reviewed to indicate that expansive movements occur to depths beyond the normally adopted founding depths for movement controlling foundations in semi-arid areas, contrary to the common assumption of a shallower limit to such movements. An analysis is presented of the performance of deep foundations under these conditions and it is concluded that pile conformation is critical to controlling differential movement. Guidelines and design curves are presented for the selection of optimum pile conformations, with examples of their application.

LIST OF SYMBOLS

c_a	ultimate shaft adhesion
c_u	undrained cohesion of soil
d	pile shaft diameter
D	pile base diameter
E_s	Young's Modulus of soil
L	length of pile shaft
m_v	coefficient of volume compressibility
p	degree of potential expansive movement at the neutral level
p_{crit}	degree of expansive movement at the neutral level above which ultimate adhesion stress conditions provide a better estimate of uplift or drag-down loads, than $P_{a_{elast}}$
P	total load in pile shaft
$P_{a,b}$	uplift or drag-down loads on pile shaft
$P_{a_{elast}}$	uplift or drag-down load estimated for a rigid foundation in an elastic soil with no slip between the foundation and the surrounding soil
$P_{a_{ult}}$	uplift or drag-down load at ultimate adhesion stress conditions
Q	load applied to head of the pile
Q_u	ultimate load capacity of foundation
R	soil resistance at the base of the pile
R'	uplift resistance of soil on base enlargement
z	depth to the neutral level
ρ	soil movement
ρ_0	soil movement at ground surface
ν	Poisson's ratio for soil

1.—INTRODUCTION

A deep foundation system is commonly employed as a means of controlling foundation movement in expansive soils, those soils subject to movement with changes in moisture conditions.

The principle on which this type of foundation is designed is that of providing "anchorage" in the sub-soil at a depth at which differential soil movements are acceptable. This anchorage is assumed to derive from adhesion stresses between the foundation unit and the soil, plus the pull-out resistance of an enlarged base if employed, counteracting disturbing forces produced by adhesion stresses above the anchorage level (Refs. 24 and 27).

Selection of a suitable "anchorage" depth has been based on the assumption of a depth limit to expansive movement which would allow anchorage in deeper soil, free of such movement. This assumption has been based on experiences of the depth of moisture changes produced by seasonal effects, many of which have been for climatic conditions in other countries, and such depths have been suggested for the various climatic and soil conditions, in the range 1.2 m to 3 m (Refs. 5, 8, 11, 24 and 26).

However, there is evidence that the depth of expansive movements can be considerably greater for semi-arid climatic conditions, such as can be experienced over much of Australia. This evidence is briefly reviewed, and the significance of such deeper movement with respect to the design of movement controlling foundations is analysed in this paper.

2.—DEPTH OF EXPANSIVE MOVEMENT

2.1 Expansive Movement Beneath Exposed Areas

There are a considerable number of reports of the movement of shallow foundations in expansive soils (Refs. 8, 11, 24 and 25) with differential components of up to 75 mm or more, and the possibility of such movement is generally recognised. There are also reported cases of the movement of deep foundations in expansive soils, indicating the existence of expansive movements to considerable depths, and the development of high uplift loads due to shaft adhesion (Ref. 9).

Expansive soil movements result from changes in effective stress produced by changes in pore pressure. Pore pressures are generally negative with respect to atmospheric pressure where expansive movements are brought about by climatic conditions or plant growth, and lenticular water in the voids of fine grained soils such as silts and clays can sustain very high negative pressures. Measurements of the total suctions of soils are reported by Aitchison (Ref. 1) for thirty sites of clay and sand-clay subsoil profiles (classification CH to SC) throughout south western New South Wales, Victoria and South Australia, made over a drought period which occurred during the summer of 1967-68. Suction values generally in excess of 1 MPa were measured to the full depth of exploration of at least 8 m, and the highest values measured ranged from 8 MPa near ground surface to 2.5 MPa at depths of 9 m. Although matrix suctions or negative pore pressures were not measured independently of solute suctions, it was concluded that changes in soil volume could be expected throughout the soils investigated.

It is noted that the climatic conditions reported for this drought period corresponded to normal conditions for more arid areas, such as central and western Queensland, and for western New South Wales.

The author has had occasion, during the performance of routine site investigations in semi-arid areas, known by experience to be highly expansive, to note evidence of movements at depths of up to 6 m or more, in the form of fissures or shrinkage cracks, containing recent surface plant debris. Measurement of soil suction did not form part of these routine type investigations, but determinations of degree of saturation were made to obtain indications of the soil moisture conditions. Relatively low degrees of saturation have frequently been encountered to depths of 6 m or more, indicating the presence of high negative pore pressures over all this range.

2.2 Expansive Movements Beneath Covered Areas

Covering or sealing an area will cause the pore pressure in the soil above the water table to approach an equilibrium value which is never of such a high negative magnitude as can exist beneath an exposed area in the same region (Refs. 2 and 21). Thus, if higher negative pore pressures exist in an expansive soil beneath an exposed area, these will be greatly reduced by the action of covering it, and swelling movements will result. (These movements could be aggravated by water artificially introduced into the soil, such as from gardens or faulty drainage.) In the event of an area being covered when the negative pore pressures were of a lower magnitude than the covered equilibrium value, (e.g. after prolonged flooding) settlement could be expected as the covered equilibrium value was approached. In addition, high changes in pore pressure and consequent changes in soil movement can still subsequently occur close to the edges of a covered area (Ref. 2).

Although observations of soil movement subsequent to covering an area have not frequently been taken to great depths, the occurrence of deep movements has been confirmed where this has been done on sites with a deep water table. Ground movements have been observed beneath houses in the Orange Free State and Transvaal, South Africa, over a number of years subsequent to their construction, and surface movements of up to 200 mm have been recorded, with movements occurring over depths of up to 12 m (Refs. 9, 13 and 14).

Thus, unless it can be shown that the pore pressure in the soil at the time of construction is close to the covered equilibrium value, and unless future pore pressure variations in the vicinity of perimeter foundations can be suppressed, the soil could be expected to undergo expansive movement subsequent to construction. Where the water table is deep this movement could extend beyond the normally adopted founding depth of movement controlling foundations of up to 6 m.

2.3 Prediction of Expansive Movement

Identification of soils susceptible to expansive movement is a prerequisite to the design of any movement controlling foundation.

There are a number of empirical methods of doing this (Refs. 15 and 26) based on one or more classification tests, and a number of approaches have been made to prediction of expansive movement in terms of soil compressibility and stress and pore pressure changes

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(Refs. 4, 6 and 16). The former type of method must be doubtful when applied under climatic conditions other than those for which they were empirically established and the latter are limited by lack of exact knowledge of the effect of different stress paths, and of pore pressures and effective stresses produced by given climatic conditions.

Methods are available, at a research level, for determining movement directly from soil suction (Ref. 3) but at present, all available methods can provide predictions of order of magnitude only of the range of movement which would occur under extreme climatic conditions. However, these methods and observations which have been made of expansive movement (Refs. 10, 13 and 14), indicate either a constant or a decreasing range of potential expansion with depth, and thus a linear or concave potential movement — depth profile such as shown in Fig. 1.

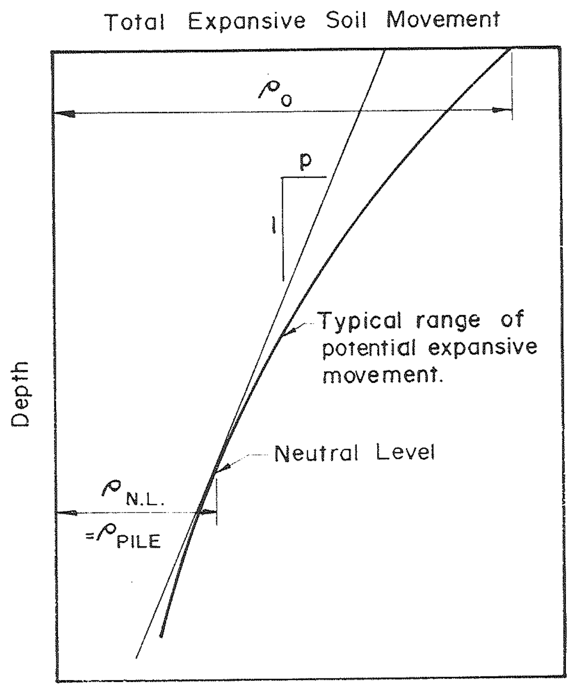

Fig. 1.—Typical profile of range of potential expansive movement—depth.

3.—FOUNDATION PERFORMANCE

3.1 Soil Stresses on Foundation Shafts

Adhesion stresses would be produced on a deep foundation shaft in an expansive soil by the tendency to relative movement between the shaft and the soil, and the general stress pattern developed would be that shown in Fig. 2. The level at which the adhesion stresses reverse on the shaft, which will be defined at the "neutral level", would be the level at which there would be zero relative movement between the shaft and the surrounding soil, and therefore the level at which effective anchorage in the soil would be achieved (Ref. 7). (It is worthy of note that a foundation could not develop both drag-down stresses on the lower portion of its shaft, and end bearing stresses, because of the incompatibility of strains associated with the two conditions.)

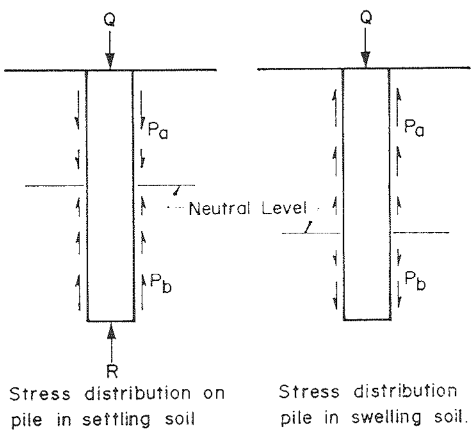

Fig. 2.—Stress systems on deep foundations in an expansive soil.

The equilibrium equations for the general stress patterns shown in Fig. 2 would be, for a swelling soil,

$$Q + P_b = P_a$$

and for a settling soil

$$Q + P_a = P_b + R \tag{1}$$

When expansive movement begins in the soil surrounding a foundation shaft there will be no slip between the soil and the shaft. However, as movement progresses, slip will develop from the ends of the shaft, as ultimate adhesion values are reached, as shown in Fig. 3. For soil movements above a certain magnitude the area within a stress diagram which has no ultimate shaft adhesion limitation would be greater than the area within the stress diagram representing ultimate adhesion conditions over the full length of the foundation shaft (Fig. 3(d)). Thus, this latter condition would give a better estimation of shaft load and foundation performance at this stage of soil movement.

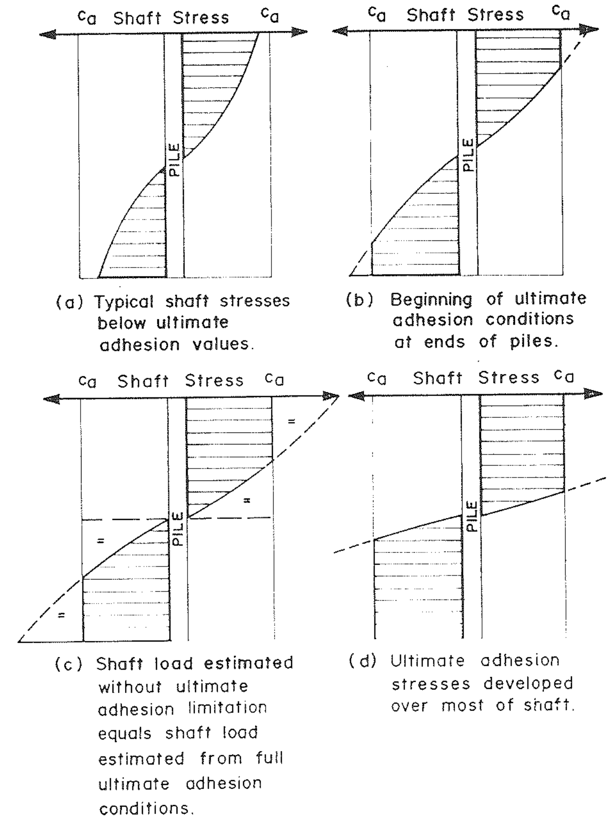

Fig. 3.—The development of ultimate stress conditions on a pile shaft in an expansive soil.

Poulos and Davis (Ref. 22) have analysed the performance of piles in expansive soils as rigid shafts in an elastic medium, with no slip between shaft and soil and have indicated a convex stress diagram such as shown in Fig. 3(a). For these conditions, the assumption of ultimate adhesion stress over the full length of a shaft in the appropriate range of soil movement gives a maximum over-estimate of less than 25% on drag-down load for a stress limited elastic analysis, as indicated in Fig. 3(c).

The range of soil movement for which such ultimate adhesion analysis would be more appropriate has been estimated from the work by Poulos and Davis (Ref. 22). For example, Poulos and Davis indicate that, for all piles of normal dimensions on which no slip occurs between pile and soil

$$\frac{P_{a\,elast}}{E_s\ \rho_0\ d} \geq 1$$

If this expression is re-written as

$$\frac{P_{a\,elast}}{P_{a\,ult}} \cdot \frac{\pi\ z\ c_a}{E_s\ \rho_0} \geq 1$$

then

$$\frac{P_{a\,elast}}{P_{a\,ult}} \geq 1 \text{ if } \frac{\pi\ z\ c_a}{E_s\ \rho_0} \leq 1 \tag{2}$$

If the depth of expansive movement is greater than the founding depth of the pile, and the shape of the soil movement profile is similar to that shown in Fig. 1 (since Fig. 1 shows total expansive soil movement then p, the degree of potential expansive movement at the neutral level, will be represented by the gradient of the total movement curve at this level) then Poulos and Davis indicate that, for a uniform diameter pile and elastic conditions

$$\frac{z}{\rho_o} \leqslant \frac{0.6}{p} \tag{3}$$

Also, for $\nu = 0.3$,

$$\frac{c_a}{E_s} \leqslant \frac{c_u}{E_s} = \frac{(1-\nu)}{1-\nu-2\nu^2} c_u\, m_v = 1.34\, c_u\, m_v \tag{4}$$

(Values of ν have generally been found to be of the order of 0.3 in over-consolidated soils.)

Values of the product $c_u\, m_v$ have been obtained by the author, for known expansive soils from various sites throughout Queensland, and are shown in Fig. 4.

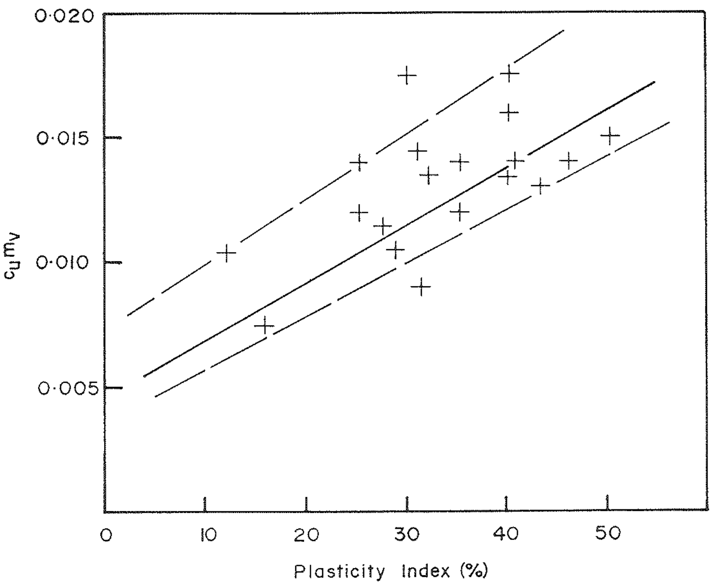

Fig. 4.—Observations $c_u\, m_v$ against plasticity index for known expansive soils.

Values of c_u were obtained from undrained triaxial compression tests. Values of m_v were obtained from oedometer consolidation tests on conventional soaked specimens or specimens tested at their field moisture content, and for effective stress increments on the estimated *in-situ* vertical effective stress approximating to the respective c_u values of the specimens. In the first case the effective stress condition was estimated from the degree of swelling on soaking and in the second the slope of the test curve was assumed to approximate the slope of the true effective stress curve (Ref. 4).

The majority of these soils have Plasticity Indicies less than 40%, and thus an average value of $c_u\, m_v$ of less than 0.0135. Thus, from (2), (3) and (4) it can be shown that

$$\frac{P_{a\,elast}}{P_{a\,ult}} \geqslant 1 \text{ if } p \geqslant 0.03$$

The results of a more detailed analysis of the critical values of p are given in Fig. 5. It is of interest to note that the use of an enlarged base causes the development of ultimate adhesion conditions at lower soil movements.

Both the methods of expansive movement prediction and the observations of such movement, which have been discussed, indicate the occurrence of degrees of expansion in semi-arid areas greater than 2% to 3%, to depths beyond the normal founding depth for deep movement controlling foundations. Thus ultimate adhesion conditions could be developed over the full length of foundation shafts, and prudent design would have to be based on this assumption, unless it can be determined for any particular case, that such extremes of movement would not occur. (Values of ultimate shaft adhesion with respect to shear strength properties of a soil are discussed in the Appendix.)

3.2 Design to Control Differential Movement

Deep foundations have been employed, founding within the depth of potential expansive movement in semi-arid areas, and where these foundations have been of sufficient depth in any area, they have satisfactorily controlled differential movement. Therefore the neutral levels of these foundations must be placed in a zone of relatively uniform expansive movement. Prediction of the depth of this zone is beyond the present limits of soil mechanics theory, and we must rely on experience of satisfactory founding depths for various areas as a basis for its estimation. A general review of these founding levels, based on the following analysis has indicated a range of increasing depths with increasing aridity, for which typical examples are given below

Area	Estimated Min. N.L. Depth
Brisbane and S.E. Queensland near coastal areas	2.0 m to 2.5 m
Darling Downs	3 m
Central and western Queensland	3.5 m to 4 m

However, as these values are based on observations of foundation performance without observations of sub-soil moisture changes, it is considered that it would be prudent to adopt slightly greater depths if construction is to follow a prolonged dry period.

Although ultimate stress conditions may exist over virtually the full length of a pile, bearing capacity failure would not be imminent unless the neutral level was close to the toe of the pile. Rather, changes in load and ultimate shaft adhesion would result in re-adjustment of the position of the neutral level. Similarly, differences in values of these parameters applying concurrently to similar piles would result in differences in the neutral level depths on those piles.

A difference in neutral level depths (Δz) between two piles, would result in a differential movement between them ($\Delta\rho$) if the soil undergoes a deep expansive movement subsequent to their installation, which may be expressed as

$$\Delta\rho = p\,\Delta z \tag{5}$$

Dimensionless expressions can be obtained from the equilibrium (1) for the location of the neutral level on cylindrical shafts with allowance for enlarged bases if employed. For a swelling soil,

$$\frac{z}{d} = \frac{L}{2d} + \frac{7}{8}\frac{c_u}{c_a}\left(\frac{D^2}{d^2} - 1\right) + \frac{1}{2\pi}\frac{Q}{d^2\, c_a}$$

and for a setting soil,

$$\frac{z}{d} = \frac{L}{2d} + \frac{9}{8}\frac{c_u\, D^2}{c_a\, d^2} - \frac{1}{2\pi}\frac{Q}{d^2\, c_a} \tag{6}$$

Unless the neutral level was located near the base of the pile a large relative movement would occur between the base and the surrounding soil for the range of expansive movement under consideration, indicating an ultimate bearing capacity condition. This condition has been assumed in (6) with an ultimate nett bearing pressure of 9 c_u in the downward direction and 7 c_u in the upward direction, in which the sloping shoulders of the base would be in bearing.

By combining (5) and (6) the inter-relation between the factors effecting differential movement due to differences in neutral level depths may be determined and is for a swelling soil,

$$\frac{\Delta\rho}{pd} = \frac{7}{8}\left(\frac{D^2}{d^2} - 1\right)\Delta\left(\frac{c_u}{c_a}\right) + \frac{1}{2\pi}\Delta\left(\frac{Q}{d^2\, c_a}\right)$$

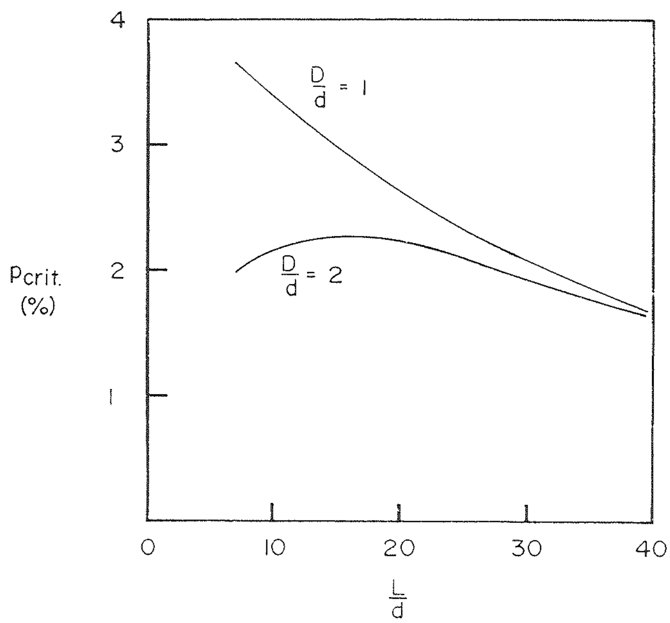

Fig. 5.—Estimated degree of average expansive movement at which ultimate soil stress conditions become more appropriate than elastic soil stress conditions in design.

and for a settling soil,

$$\frac{\Delta\rho}{pd} = \frac{9}{8}\frac{D^2}{d^2}\,\Delta\left(\frac{c_u}{c_a}\right) - \frac{1}{2\pi}\,\Delta\left(\frac{Q}{d^2\,c_a}\right) \qquad (7)$$

These relationships have been plotted in Fig. 6, for selected conditions.

Although the value of ρ will generally not be known exactly in practical cases, due to lack of knowledge at this stage, Fig. 6 will still provide a basis for examination of comparative performance of various pile conformations.

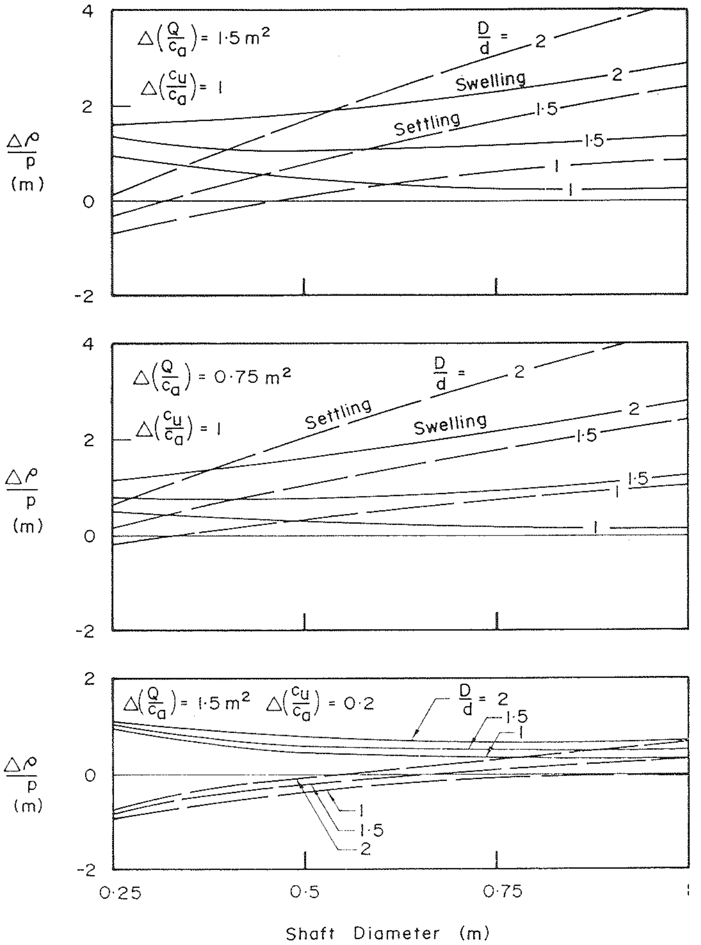

Fig. 6.—Potential differential movement of piles in expansive soils.

Once the shaft and base dimensions are fixed, the founding depth for a pile can be determined directly from (6) to give the neutral level at the required depth. However, it is necessary to ensure that the worst combination of design conditions is considered, and to allow this combination to be visually selected, (6) are presented graphically in Fig. 7.

The maximum load in the pile shaft can be evaluated from the following equations.

For a swelling soil,

$$P = Q - \pi dz c_u$$

and for a settling soil,

$$P = Q + \pi dz c_u \qquad (8)$$

Although values of z will vary with various combinations of Q and c_u, this difference will be slight if the pile has been designed to limit movements to normally acceptable limits for building structures. Therefore, and in view of the inexactitudes present in the selection of maximum shaft adhesion values, it would generally be satisfactory to use the limiting values of z in (8).

4.—CONCLUSIONS

From an examination of (7) and Fig. 6, a number of general conclusions can be drawn about the performance of piles in expansive soils.

(i) In a swelling soil potential differential movement of piles will be relatively insensitive to changes in shaft diameter for any given ratio of D/d, but will be significantly increased by increase in D/d at any given shaft diameter.

(ii) In a settling soil potential differential movement of piles will be sensitive to changes in shaft diameter for any given ratio of D/d, and also to changes in D/d at any given shaft diameter.

(iii) In a settling soil potential differential movements could be made very small by selection of an appropriate shaft diameter. (This characteristic of pile performance may explain, at least in part, why "floating" piles in a consolidating soil stratum have been effective in controlling differential settlements under load.)

(iv) In a soil subject to either swelling or settling there will be a shaft diameter for which potential differential movements would be the same for both modes of soil movement. This would be the optimum diameter for pile movement performance.

(v) Potential differential movement is sensitive to variations in shaft adhesion.

Most pile design for expansive soil will be for swelling only or for swelling or settling conditions, and from the preceding points the following conclusions can be drawn.

Uniform diameter piles are to be preferred from the point of view of controlling differential movement.

If design is for swelling conditions only, then shaft diameter will not be critical to movement performance.

If design is for both swelling and settling conditions the optimum diameter would be preferable, but there would be little degradation in

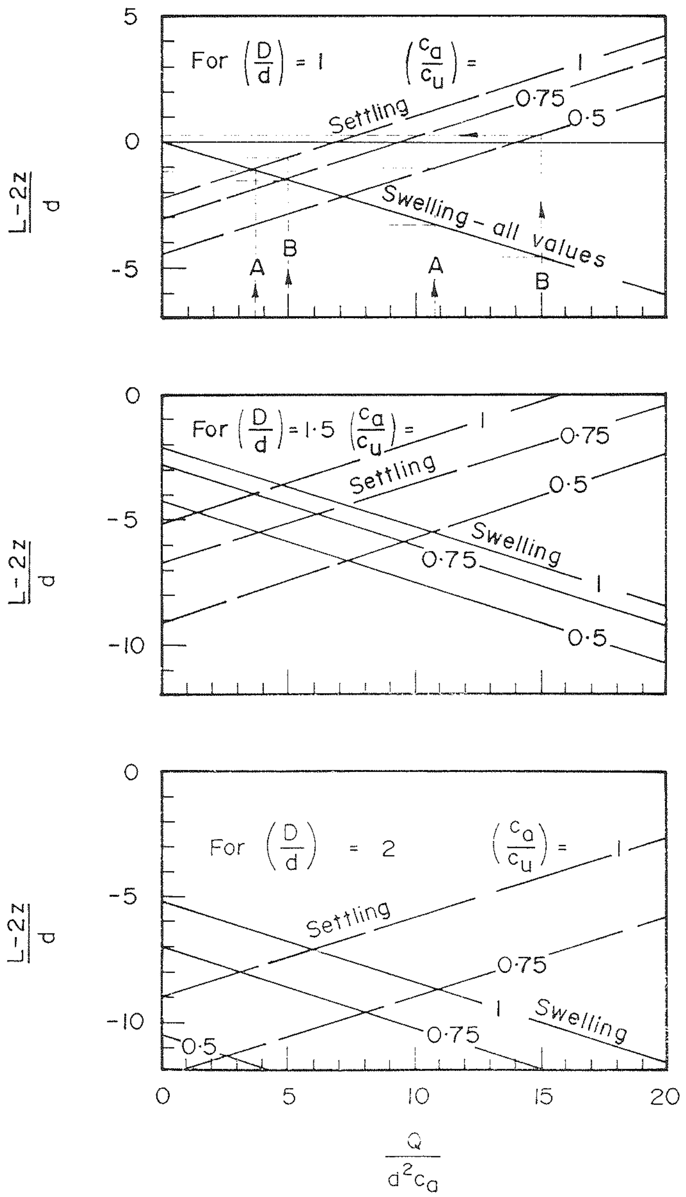

Fig. 7.—Design curves for selection of pile length.

Note: Low values of $\frac{Q}{d^2c_u}$ *will be critical in swelling soil.*

High values of $\frac{Q}{d^2c_a}$ *will be critical in settling soil.*

movement performance if smaller diameter piles were used for practical or economic reasons. For uniform diameter piles, potential differential movements would be relatively low for all normal conditions for shaft diameters of up to 600 mm at least.

Construction techniques to reduce variations in shaft adhesion would be desirable, especially if an enlarged base is to be used.

Application of these design principles is illustrated by the following example.

5.—EXAMPLE

Select piles to carry loads of 180 kN to 270 kN in an expansive soil which may undergo either swelling or settling movement and in which a minimum neutral level depth of 3 m is required.

(a) if $c_u = 250$ kPa and $c_a = 125$ kPa to 250 kPa

(b) if $c_u = 100$ kPa and $c_a = 50$ kPa to 100 kPa

Allow 600 mm for ground beam or pile cap depth, giving $z = 2.4$ m for design.

Use $D/d = 1$ to minimise differential movements, and select d at or below, say, 600 mm diameter to limit potential differential movements.

Case	A	B
Try design d of	450 mm	600 mm
$\left(\frac{Q}{d^2 c_a}\right)_{\max}$	10.7	15
$\left(\frac{Q}{d^2 c_a}\right)_{\min}$	3.6	5
From Fig. 7 select $\frac{L-2z}{d}$ for critical values of $\left(\frac{Q}{d^2 c_a}\right)$	− 1.2	+ 0.2
$\therefore L$	4.3 m	4.9 m
Check $Q_{u\ \min}$ for pile ($= \pi d L c_{a\,min} + \frac{\pi D^2}{4} . 9\,c_u$)	1130 kN	716 kN
Add depth of pile cap to give founding depth	4.9 m	5.5 m
Calculate shaft loads $Q - \pi d z c_a$	− 670 kN	− 272 kN

In each case there are four possible limiting combinations of Q, and c_a for each mode of soil movement within which the critical combination will lie. The critical combination will be that giving the highest value of $\frac{L-2z}{d}$, and thus the greatest founding depth to ensure that the required minimum depth of z is achieved. Inspection of Fig. 7 indicates that the minimum value of $\frac{Q}{d^2\ c_a}$ will give the critical combination for a swelling soil and the maximum value of $\frac{Q}{d^2\ c_a}$ will at least approximate the required founding depth for the critical combination for a settling soil. Thus, although the critical condition for design cannot always be readily recognised, analysis is reduced to consideration of, at the most, two combinations and the values of $\frac{L-2z}{d}$ for these combinations can be readily read from Fig. 7.

6.—GENERAL DESIGN CONSIDERATIONS

The analysis which has been presented has been for the differential movement of piles simultaneously experiencing one mode of soil movement, either swelling or settling. If different piles on a given site simultaneously experience different modes of soil movement, differential movements between these piles would be considerably greater if the expansive movements are deep with respect to the pile depths. Therefore, avoidance of the simultaneous occurrence of both modes of soil movement would be a design consideration where this type of foundation was employed in semi-arid areas.

In-ground structures, such as ground beams or pile caps, are frequently employed with this type of foundation, and where this is done it would be necessary to avoid uplift or drag-down soil stresses on these members to achieve satisfactory foundation performance.

Shaft loads in this form of foundation are very high. However, as the major proportion of these loads will generally be produced by an ultimate stress condition, and the suggested design procedure slightly over-estimates their magnitude, a relatively low factor of safety can be accepted in structural design. High tensile loads are common, requiring heavy reinforcement, and the problem of providing sufficient bond length for this reinforcement would arise. Where shaft adhesion values are very high, the shaft loads produced may become the critical factor in design.

APPENDIX

Information which has been obtained on ultimate shaft adhesion values in clays has been empirical rather than analytical (Refs. 17, 18, 19, 20, 28, 29, 30 and 31) and for firm to stiff insensitive clays, similar in consistency to the less stiff of the expansive soils commonly encountered values of c_a/c_u have been observed in the range 0.3 to 0.7. However, all these observations have been for saturated clays, and to assess the likely relative shaft adhesion values in unsaturated clays we must rely on consideration of the factors to which influence on shaft adhesion has been attributed.

Remoulding of the soil in the sides of shafts during their excavation has been found to influence ultimate shaft adhesion in some soils. However, expansive soils are generally stiff, over-consolidated and relatively insensitive and under normal construction conditions, the strength loss due to remoulding by excavation operations could be expected to be of the same order of magnitude as that inherent in routine investigation sampling and testing. Under these conditions this factor would have little influence on the apparent ratio of c_a/c_u.

Ultimate shaft adhesion values in saturated soils has been found to be influenced by the formation of a "smear" of cuttings on the sides of shaft excavations. Because of the absence of free groundwater and the presence of negative pore pressures in unsaturated soils, there would be much less tendency for cuttings to smear, and excavations in clays having high negative pore pressures often present a clean appearance, similar to a rock surface.

In a saturated soil the stress reduction produced by shaft excavation has been found to cause migration of pore water to the vicinity of the excavation, and consequent reduction in the shear strength of the soil. The magnitude of negative pore pressures and of effective stresses in unsaturated soils will frequently be much greater than the stress changes produced by excavation and the moisture content and effective stress changes would be appreciably less.

Ultimate shaft adhesion has been influenced in saturated soils by free water absorbed from wet concrete in the pile shafts, and the presence of negative pore pressures in unsaturated soils would accelerate this rate of absorption over that for similar saturated soils. No information is available on the long term effect of this absorbed water, but where the surrounding ground was still subject to natural control of the moisture regime after construction, there would be reason to expect that the moisture content in the softened zone would revert to close to that of the remainder of the site, at some stage in the life of the structure, and that a regain of shear strength would occur.

Effects on ultimate shaft adhesion have been attributed to water added to shaft excavations to assist in digging or surface water entering excavations before concreting but both these are currently considered bad practice and are usually avoided.

Because of the variable and difficult to control construction factors which may influence ultimate shaft adhesion, its prediction would be very difficult and design would therefore have to be based on estimated limiting values.

The preceding considerations indicate that the value of ultimate shaft adhesion could be greater in an unsaturated soil than in a saturated soil of the same shear strength. Where natural control of the moisture content regime was anticipated after construction it is considered that the maximum value of ultimate shaft adhesion could be taken as the shear strength of the soil at its equilibrium moisture content (Ref. 7). Undrained tests on unsaturated soils frequently give values of apparent undrained internal friction which reduce as moisture contents increase, while values of apparent undrained cohesion are less influenced by moisture contents. It is suggested that c_u from undrained tests at field moisture contents could be taken as a reasonable, conservative approximation to the shear strength at equilibrium moisture content when this latter value is not readily available.

Lower values of ultimate shaft adhesion are possible, as a result of construction techniques and also as a result of variations in soil strength over a site from the values determined from investigation. On the basis of the ratio of maximum to minimum ultimate shaft adhesion values which have been recorded for saturated soils, a minimum value of c_a/c_u of 0.5 is suggested for normal construction conditions.

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