Skin Friction on Underslurry Piles

Frottement Latéral de Pieux Forés à la Boue

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SYNOPSIS In recent years research has been carried out at the University of Natal, Durban (South Africa) into the use of underslurry piles. This research has included a study of the properties of pure bentonite, the thickness, composition, shear strength and consolidation characteristics of the filtercake and the movement of ground around the pile hole.

In this paper the factors affecting the transfer of load between the pile and the surrounding soil are discussed. Research has shown that the shear strength of the filtercake is frictional in nature and therefore dependent on the normal stress acting on it. The factors affecting the magnitude of the normal stress on the filtercake are examined in the light of the Biot Consolidation Theory and Elastic Theory. The change in thickness of the filtercake and the time taken for the construction of the pile are taken into account.

From the research it may be concluded that significant skin friction can be developed between the pile and the soil. This is borne out by full scale pile test results. Limited data from the test loading of a pile to 11 500 kN is presented in the paper.

INTRODUCTION

Underslurry piling has become a commonly accepted founding method especially in areas where a high water table is encountered in unconsolidated sediments. The technique has, for example, been used extensively in the coastal regions around Durban where the soil profile consists of deep sand and clay deposits with a water table a few metres below the surface.

The reasons for using underslurry piles and the techniques employed in their construction have been dealt with extensively in previous publications (Everett et al 1975, Wates 1974, Braadvedt 1976) and will not be covered here.

In recent years it has become increasingly apparent that underslurry piles are capable of carrying far higher loads in skin friction than previously thought possible (Wates 1974, Farmer et al 1970, Corbett et al 1974). One of the main reasons for initially underestimating the contribution of skin friction to the load carrying capacity of the pile was the suspected low strength of the remaining bentonite filtercake between the pile concrete and the surrounding soil. The presence of a layer of bentonite which is not removed during pile concreting has been established by Scott (1978). However, research conducted at the University of Natal has shown that the shearing resistance of the filtercake is considerable. The shearing resistance is dependent both on the properties of the filtercake and on the normal stress acting on the circumference of the pile. (Scott 1978, Day 1988).

PROPERTIES OF THE FILTERCAKE

Despite the scouring action of the concrete during casting, a portion of the filtercake deposited on the wall of the hole remains trapped between the pile and the soil (Scott 1978). The thickness of this layer depends on the soil type and the time for which the hole remains filled with betonite. (Wates 1974, Scott 1978, Schreiner 1978).

While the angle of shearing resistance of pure bentonite is low, typically 8°, Wates (1974) has shown that the cohesive strength of pure bentonite is substantially increased in the immediate proximity of hydrating cement.

In subsequent work, Scott (1978) collected numerous samples of bentonite filtercake in the field and conducted shear strength tests on the material. These tests revealed that the shear strength of the filtercake is largely frictional with angles of shearing resistance between 21° and 38° being recorded. This increase in shear strength is due in part to the contamination of the bentonite with in-situ material and fine aggregate from the concrete (Scott 1978).

In order to evaluate the consolidation characteristics and compressibility of the filtercake, Schreiner (1978) conducted a series of model tests. In these tests, the soil was modelled as a porous but fixed boundary i.e. the effects of stress relief and consolidation of the soil were not taken into account. After casting concrete against the filtercake, the pressure exerted by the filtercake against the "soil" was monitored for a period up to three days. While the concrete was still fluid, the pressure exerted on the soil was equal to the pressure exerted by the concrete on the filtercake. If, by the time the concrete had set, the filtercake had undergone full consolidation, this pressure remained constant.

However, if the filtercake was not fully consolidated when the concrete had set, the pressure exerted on the soil decreased as the remaining excess pore water pressure in the filtercake dissipated.

After full consolidation of the filtercake, the residual pressure against the soil (p*) was measured. The filtercake was then loaded incrementally and the change in thickness measured. In this way, the load deflection characteristics of the drained filtercake were determined. In all tests the relationship between load and deflection was found to be approximately linear.

From the observations, the following relationship between normal (or radial) stress across the filtercake (P_r) and inward deflection of the soil or change in thickness of the filtercake (d) subsequent to setting of the concrete may be deduced:

$$P_r = p^* + kd \qquad (1)$$

where p^* = residual pressure on the filtercake in the absence of soil movement.

and k = the observed slope of the load deflection curve for the drained filtercake.

By performing a series of tests, Schreiner determined the effects of the degree of contamination of the filtercake with sand, the head of slurry used (h_f) and the filtration time or time for which the hole was filled with bentonite (t_f). Using the values of p* and k obtained by Schreiner (1978) the load deflection curves for various filtercakes have been plotted as broken lines in Figure 1.

CONSOLIDATION OF THE SURROUNDING SOIL

In the case of saturated soil, particularly a clayey or slow draining soil, the movement of the ground is expected to be time dependent and governed by the theory of consolidation. However, Day (1980) has pointed out that elastic theory and the Biot Theory of Consolidation (Biot 1941) predict that the movement of the ground around a circular hole will be instantaneous for the following reasons:

Firstly, in Biot Theory, the initial change in pore water pressure is generated by a change in the average total stress applied to the soil i.e. the average of the three principal total stresses. In the case of a circular hole in an elastic solid, the change in radial stress is equal in magnitude but of opposite sign to the change in tangential stress and the axial stress remains constant (Muir Wood 1975). The average principal stress is therefore unaltered by a change in pressure on the face of a circular hole and, according to Biot Theory, no excess pore water pressure is generated. The same is substantially true for eliptical or oblong holes with l/b less than 2,5 (Day 1980).

Secondly, elastic theory predicts that the deformation of soil around a circular hole in response to a pressure applied to the face of the hole will occur at constant volume. If there is no volume change, no excess pore water pressure can be generated.

Finally, the consolidating soil around the hole may be regarded as an equivalent elastic solid and the problem analysed in terms of total stress following a suggestion by Mandel (1953). This is done by ascribing elastic properties to the soil which vary according to the degree of consolidation at that point in the soil mass.

The relationship between the elastic parameters (elastic modulus and Poisson's ratio) is such as to keep the shear modulus (G) constant (Gibson et al 1963). This is a prerequisite of the equivalence of the total and effective stress analyses. Elastic Theory however shows the deflection of the ground around a circular hole to be governed by the shear modulus and therefore independent of the distribution of pore water pressure.

The deflection of the side wall (d) of a circular hole in elastic ground in response to an applied radial pressure (P_r) is given by

$$d = \frac{P_r a}{2G} \qquad (2)$$

where a = radius of hole

A series of lines have been plotted on Figure 1 representing the inward deflection of the wall of the pile hole in response to the reduction in the applied stress which occurs during consolidation of the filtercake. The lines have been drawn to intersect the pressure axis at a pressure of 100 kPa representing the pressure exerted by the wet concrete at a depth of about 4 m.

APPLICATION TO A PILE HOLE

The observations of the previous two sections have been combined in Figure 1. In order to satisfy the requirements of equilibrium and compatibility:

(i) The radial stress in the soil adjacent to the pile and the normal stress on the filtercake must be equal, and

(ii) the inward displacement of the soil and the change in thickness of the filtercake subsequent to the setting of the concrete must be equal.

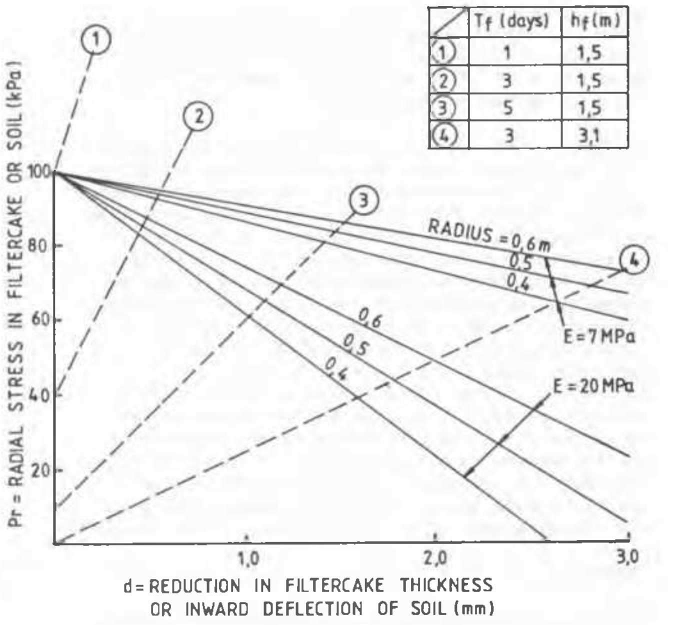

FIGURE 1: LOAD DEFLECTION CURVES FOR FILTERCAKE AND SOIL.

This situation occurs at the intersection point of the load deflection curves for the soil and the filtercake given in Figure 1.

The following conclusions may thus be drawn from Figure 1:

(i) High filtration times (t_f) produce thicker and more compressible filtercakes yielding lower final stresses. If the concrete is cast within 24 hours of forming the hole, the final stress on the filtercake will be equal to that exerted by the wet concrete as the filtercake is fully consolidated prior to setting of the concrete.

(ii) The final stresses on the filtercake are lower in stiff ground.

(iii) High filtration pressures (h_f) yield lower final stresses due to a greater thickness of the filtercake.

(iv) Final stresses are higher for larger diameter holes.

(v) In the absence of creep, the residual stress is dependent on the pressure exerted by the wet concrete and not on the initial horizontal earth pressure.

(vi) Creep stains producing inward movement of the soil will increase the stress on the filtercake. (Peck 1969, Day 1980).

Although the numeric values given in this figure are only indicative of expected values, the figure does serve to illustrate the factors influencing the final pressure that can be expected on the filter-cake in the absence of creep.

To estimate the anticipated skin friction, the normal pressure on the filtercake should be multiplied by the tangent of the angle of shearing resistance for the filtercake. For example, consider a 0,8 m diameter (radius = 0,4 m) circular pile, cast 5 days after forming the hole (t_f = 5 days). Assume the drained elastic modulus of the soil is 20 MPa and that the bentonite level in the hole was kept 1,5 m above the water table (h_f=1,5 m). At a depth of 4 m, where the pressure exerted by the wet concrete is approximately 100 kPa, Figure 1 predicts that the final stress scross the filtercake will be approximately 60 kPa. If the angle of shearing resistance of the filtercake is approximately 30° the maximum skin friction that can be developed is 60. tan 30° = 35 kPa.

FULL SCALE LOAD TEST

In order to substantiate the assertion that the skin friction on an underslurry pile can be significantly higher than intially expected, the following case history is presented.

In 1975 Frankipile S.A. (Pty) Limited installed a number of 1,2 m diameter circular piles by augering under a bentonite slurry for the construction of a freeway bridge over the Umlaas Canal, Durban. The piles were socketed into the Dwyka Tillite bedrock at a depth of approximately 28 m using reverse circulation drilling.

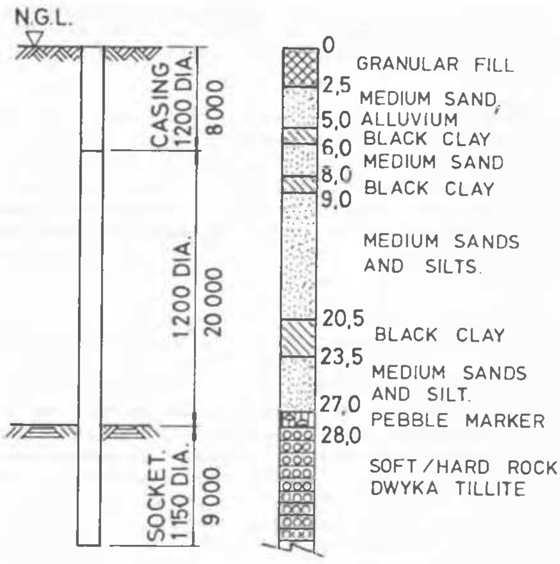

FIGURE 2: PILE DATA AND STRATIGRAPHIC COLUMN

The test pile was selected after installation and was therefore not specially instrumented. Due to the occurance of a flash set during concreting the tremmie tube was withdrawn and a steel casing forced into the concrete to form a seal. After pumping out of the slurry and chipping the exposed concrete, the remaining 9 m of the pile shaft was cast inside the casing using 45 MPa concrete. Figure 2 gives a schematic view of the pile.

The pile was loaded incrementally to the working load of 7600 kN, rebounded and then reloaded incrementally to 1,5 times working load. The load was applied by 7 x 200 ton jacks acting against a reaction beam held down by four rock anchors. Deflections were measured using 4 dial gauges spaced around the perimeter of the pilehead and checked by levelling to a benchmark.

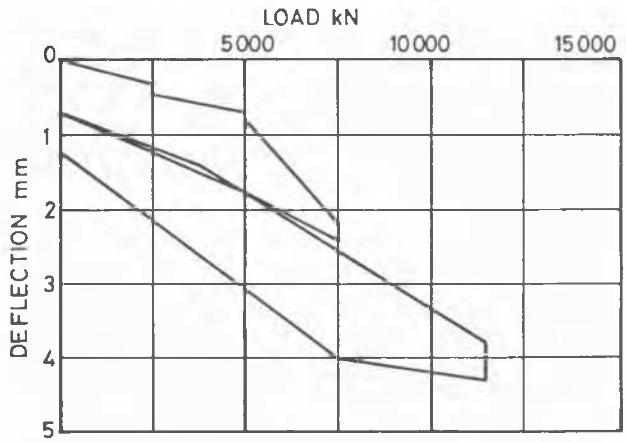

FIGURE 3: LOAD SETTLEMENT CURVE.

Settlement readings were continued until the settlement of the pilehead had effectively stopped. The load settlement curve is given in Figure 3. A maximum settlement of 4,2 mm was recorded. Settlement under working load was 2,5 mm.

Accurate calculation of the mode of load transfer from an uninstrumented pile is not possible as the problem is indeterminate. It is however possible to form reasonable upper and lower bound estimates of the mobilised skin friction in cases where the pile is socketed into rock and where the deflections under load are less than the elastic shortening of the pile shaft in the absence of skin friction.

Assuming composite elastic moduli of 20 GPa (Wates 1980) and 48 GPa for the tremmied concrete and encased concrete respecitvely, a settlement of 11,4 mm is anticipated under full test load in the absence of skin friction. From the load deflection curve it would appear that pile behaved elastically up to approximately 5000 kN. The relatively small deflections indicate that most of the load is being carried in skin friction. Assuming elastic behaviour and uniform load shedding in friction, the skin friction on the pile may be estimated by successive approximations (Tomlinson 1977) since the load on the base is relatively small.

Consider the first load cycle. Assuming a uniform shed of load down the pile shaft, the calculated average skin friction is 135 kPa. Alternatively assuming no friction is shed from the cased section of the pile the average skin friction is 290 kPa. These cases represent the upper and lower bounds respectively.

Since neither of these cases are likely, an average skin friction of 20 kPa was assumed for the cased shaft and an average skin friction of 230 kPa was calculated for the section from 9,5 m to 17, 5 m.

Assuming the pressure exerted by the tremmied concrete prior to setting was equal to the hydrostatic head of the concrete and that the filtercake was fully consolidated before the concrete set, the average stress exerted on the filtercake between the depths of 9,5 m and 17,5 m is 340 kPa. Thus, in order to develop an average skin friction of 230 kPa the angle of shearing resistance of the filtercake must be in the region of 34°. This value is within the range of 21°-38° recorded by Scott (1978).

DISCUSSION

The authors are fully aware that the method used for estimating the normal load on the filtercake is based on the theory of elasticity and fails to take account of the stress path followed by the soil and the filtercake or of possibility of dilatency of the soil on shear. It is however felt that the theory of elasticity serves as a convenient and simple first approximation to the actual situation.

Although, in the case history quoted, the test pile was not fully instrumented and as such yields an approximation to the mobilised skin friction, there can be no doubt that a substantial portion of the load was carried in skin friction. Results from other tests (Wates 1980) some on fully instrumented piles, lead to similar conclusions although perhaps not as dramatically as the pile test quoted.

CONCLUSIONS

The skin friction developed on underslurry piles is substantially higher than initially thought possible. This is due to the relatively high shear strength of the in-situ filtercake and the pressure exerted on the side of the pile by the surrounding soil. Even in the case of a saturated soil, elastic theory predicts that the movement of the soil in response to a pressure change in a circular pile hole will be immediate. The implications of the research are borne out by full scale pile tests in which substantial skin friction was developed on the pile shaft.

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