FINITE ELEMENT ANALYSIS OF PILE FOUNDATIONS SUBJECT TO PULL-OUT

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ABSTRACT:

The behaviour of piles in sand under tensile loading was investigated using finite element analysis and centrifuge modelling, and the results were compared. The implications of using simplified constitutive models are discussed, along with the modelling of interfaces between sand and structure when high degrees of relative movement are seen.

The variation of pull-out resistance with soil density, pile size and speed of pull was investigated in an attempt to predict pile behaviour.

Keywords: piles & piling; mathematical modelling; centrifuge modelling.

1. INTRODUCTION

Many structures are constructed using deep piled foundations in order to transfer structural dead load through unstable ground to a solid stratum. The behaviour of these piles acting in compression has been widely studied using numerical modelling, centrifuge modelling and full-scale testing. Upwards tensile loads may however be applied to these piles owing to the action of horizontal wind or wave forces on the structure and the behaviour of the piles under these loads is much less well documented. In this paper, work undertaken at CUED into the behaviour of these tension piles in sand using the techniques of finite element analysis and centrifuge modelling will be described, and their results compared to give an insight into the force-displacement behaviour of these piles.

The resistance of these piles to pull-out comes from two major sources, skin friction between pile and soil and suctions generated at the base of the pile as movement occurs. Both of these effects are greatly affected by the generation of excess or suction pore pressures in the soil due to movement of the pile. Suctions are generated at the base of the pile in all soils owing to the opening up of a void as the pile moves. At the sides of the pile, undrained shearing of the soil when the pile is pulled quickly will result in excess pore pressure generation in loose soils and suctions being generated in dense soils. These pore pressures will alter the effective stress state of the soil and will hence have a great impact on the force-displacement behaviour of the pile.

2. NUMERICAL ANALYSIS

Numerical analysis was carried out using the ABAQUS finite element analysis program, this was used to give load-displacement curves for the pile pull-out along with plots of pore pressure and shear stress distribution for various combinations of pile size, soil density and saturation state.

2.1 MESH DESIGN

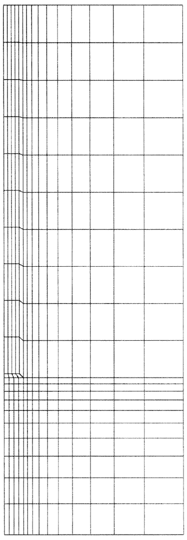


Fig. 1. Finite Element Mesh

The finite element mesh used in the analysis is shown in Figure 1. The elements used are 8-noded axisymmetric elements, with porous properties for those elements modelling the soil. The elements are biased towards the pile in order to give most data in the region of greatest interest, i.e. close to the pile. The limits of the mesh were at a radius of 10m and 10m below the base of the pile, as this was the prototype size of the containers used for the centrifuge modelling, but the low shear stresses generated on the boundaries using this mesh size suggests that enlarging the boundaries would have little effect.

The biggest problems encountered with the mesh design involved the modelling of the interfaces between pile and soil. In order to allow true behaviour to be modelled, slip must occur on the sides of the pile, and a void must open up at the base. The difference between the actual and modelled behaviour is shown in Figure 2.

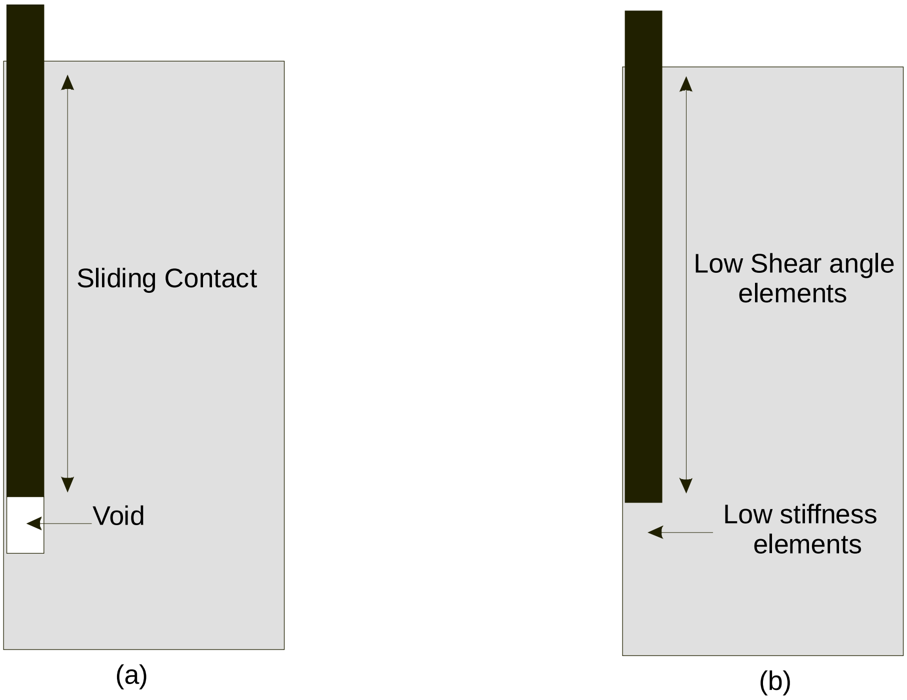


Fig. 3. a) True boundary conditions. b) Modelled boundary conditions.

In order to allow the base void to open up, initially duplicate nodes were employed on the pile and soil, giving no attachment between the two materials. This however gives numerical problems in achieving initial equilibrium, as the soil at the base of the pile yields under the high horizontal effective stress, there being no vertical stress possible across the interface. In saturated tests, the problem also occurs that pore pressures cannot act across the interface, so base suctions do not show an effect on pull-out capacity.

The problem was hence analysed using a layer of boundary elements at the base of the pile with a greatly reduced axial stiffness. This allows forces and pore pressures to act across the boundary, whilst giving very limited resistance to movement from tensile soil stiffness. This works well until failure occurs but adequate post-failure behaviour modelling was never achieved. These elements are modelled in some finite element programs by so-called “Gap” elements which have an infinite compressive axial stiffness but zero tensile axial stiffness. These allow a negligibly thin layer of elements along the interface to model void opening.

At the pile side interface, a layer of elements was incorporated with a reduced friction angle. This was an attempt to model the fact that the friction angle between sand and aluminium is much lower than the internal friction angle ϕ of the sand. This models one aspect of the boundary condition, but ignores another; as the pile moves, the contact surface area between pile and soil is obviously diminished. As the nodes on pile and soil are connected, the contact area modelled remains constant and large shear strains are imposed on the boundary layer elements. Thus once any significant movement has occurred, the modelling breaks down, as the situation being modelled is incorrect.

2.2 CONSTITUTIVE MODEL

The analysis used a Drucker-Prager (1952) elasto-plastic constitutive model for the sand. This model gives a good approximation to Mohr-Coulomb soil behaviour, although the Drucker-Prager model defines the yield surface as being a circle in the deviatoric π plane, rather than a hexagon, assuming failure to be independent of the intermediate principle stress. An associated flow condition was used with a friction angle β of 30^{o} for the soil outside the boundary layer, and a friction angle of 20^{o} for the boundary layer. A value of K, defining the ratio between tensile and compressive triaxial strengths, of 1 was used.

The Yield function is defined by:

$$F = t - d - p \tan \beta \tag{1}$$

where:

$$t = \frac{1}{2} q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^{3} \right] \tag{2}$$

The main problem with the use of the Drucker-Prager model is its inability to deal with the excess or suction pore pressures generated during shearing of loose or dense soils to critical state. The generation of these pore pressures has a great effect on the skin friction experienced by the pile, as they change the effective stress state of the soil around the pile.

This excess pore pressure generation could be modelled using a more complex constitutive model for the soil, such as the Pastor-Zienkiewicz (1985) Mark III model, which models the effect of shearing on pore pressures.

2.3 BOUNDARY CONDITIONS AND LOADING

The nodes at the base of the soil, 10m below the base of the pile, were fixed in the radial direction and the nodes at the side, 10m from the axis of the pile, were fixed in the vertical direction. The pore pressure was fixed at zero at the surface of the sand on the outside of the mesh.

A ramped vertical displacement was applied to the node at the centre of the pile top to simulate pull-out. For fast tests, this was a displacement of 10cm in 20 seconds, whereas for the slow tests it was a displacement of 10cm in 10 minutes.

3. CENTRIFUGE MODELLING

Centrifuge modelling is based on the principle of exactly modelling prototype stresses and strains in scale models. As soil is a very non-linear material, only if this condition is satisfied will correct prototype behaviour be observed.

This is accomplished by testing 1:N scale models at Ng in a geotechnical centrifuge. A summary of the scaling laws required to ensure dimensional consistency is shown in Table 1, these can be derived by dimensional analysis, as was done by Langhaar (1951), or by considering governing equations, Schofield (1980).

Parameter	**model / prototype**	**Dimensions**
Length	1/N	L
Acceleration	N	LT^{-2}
Velocity	1	LT^{-1}
Time (dynamic)	1/N	T
Stress	1	$ML^{-1}T^{-2}$
Force	$1/N^2$	MLT^{-2}
Mass	$1/N^3$	M
Seepage velocity	N	LT^{-1}
Time (seepage)	$1/N^2$	T
Force	$1/N^2$	MLT^{-2}

Table 1: Centrifuge scaling laws. Schofield (1980)

A series of twelve models of piles in sand were tested using the CUED 10m beam centrifuge running at a nominal acceleration of 100g, the design and operation of which are described by Schofield (1980).

3.1 TEST CONFIGURATION

Each test package consisted of four models of a pile in sand contained within an 850mm tub. The package is shown in Figure 3. Each model comprises of a plastic tub full of sand, with an aluminium pile with an embedded length of 23cm on the axis of the tub. The tub extends 10cm below the base of the pile, and has an internal diameter of 20cm, thus modelling a cylinder of soil 20m in diameter and 33m deep at prototype scale. The top of the pile is connected to an electrical actuator via a load cell and two pairs of links, which allow movement in two degrees of freedom to the top of the pile to compensate for any non-verticality of the pile.

The models were saturated under vacuum with either water or silicone oil, or were dry depending on the situation being modelled. Silicone oil is used as pore fluid in order to correct the anomaly between scaling laws for dynamic and seepage velocities seen in Table 1.

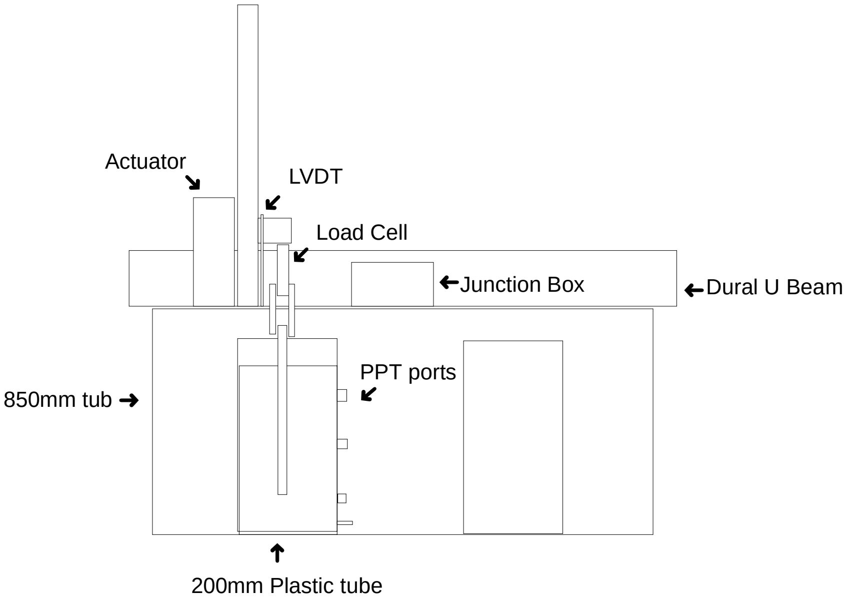


Fig 3. Centrifuge Package

Three Druck PPTs, (Pore pressure transducers), fitted with porous bronze stones, allow the pore pressures at the base and at two positions up the side of the pile, and an LVDT on the actuator allows the position of the pile to be monitored throughout the test.

3.2 TEST PROCEEDURE

The model package was loaded onto the centrifuge and accelerated to a speed of 150 rpm, giving a nominal g-level at the model of 100g. Pore pressures were monitored during swing-up to ensure that the instrumentation was working.

Once a steady speed had been achieved the actuator was started, pulling the pile from the sand at a pre-determined speed and the pore pressures, force and displacement were logged using a computer data acquisition package called Global Lab.

After pull-out was completed, the centrifuge was stopped and the actuator switched to one of the other models in the package. The test procedure was then repeated.

4. FINITE ELEMENT ANALYSIS RESULTS

The ABAQUS program was used to produce results in the form of force-displacement plots for the pile, and contour plots of the distribution of pore pressures and shear stresses at failure. All the tests produced force-displacement plots with the general features of Figure 4, showing an initial linear section to failure and linearly increasing post-failure load with displacement.

Fig. 4: Predicted force-displacement behaviour of a 19mm pile in dense dry sand.

Plots of pore pressure at failure have the features of Figure 5, showing suctions generated at the base of the pile owing to the void opening up and suctions generated up the side of the pile owing to relaxation of the horizontal stress in the soil as the pile moves.

Fig.5 Pore pressure distribution at failure in loose sands.

4.1 COMPARISON OF RESULTS FOR SATURATED AND DRY SANDS

Dry tests show significantly higher resistance to pull-out than do saturated tests. This is due to an increase by a factor of approximately two in the effective earth pressures at any point in the model in dry tests. This increases by a factor of two the skin friction experienced by the pile.

In the saturated tests, a further contribution to pull-out resistance is provided by suctions generated at the base of the pile as it moves. The absolute pore pressures at the base were shown to have fallen to approximately zero at failure, corresponding to a suction of 230 kPa in the tests carried out. This suction contributes to an increase of around 15% in pull-out resistance.

4.2 COMPARISON OF RESULTS FOR DENSE AND LOOSE SANDS

Dense models show an increase in pull-out resistance of approximately 20% over that shown by loose models. The change in buoyant unit weight of the soil between these two cases is approximately 17.5%, giving a corresponding rise in skin friction experienced. It can hence be seen that in this case the difference between forces predicted is almost entirely due to the increase in effective pressure in the soil due to increased soil density.

4.3 COMPARISON OF RESULTS FOR DRAINED AND UNDRAINED TESTS

Drained tests showed a lower pull-out resistance than did undrained tests. The suctions seen at the base were reduced to 50 kPa for the slow tests, and this 180 kPa drop in suction corresponds almost exactly to the fall in pull-out resistance.

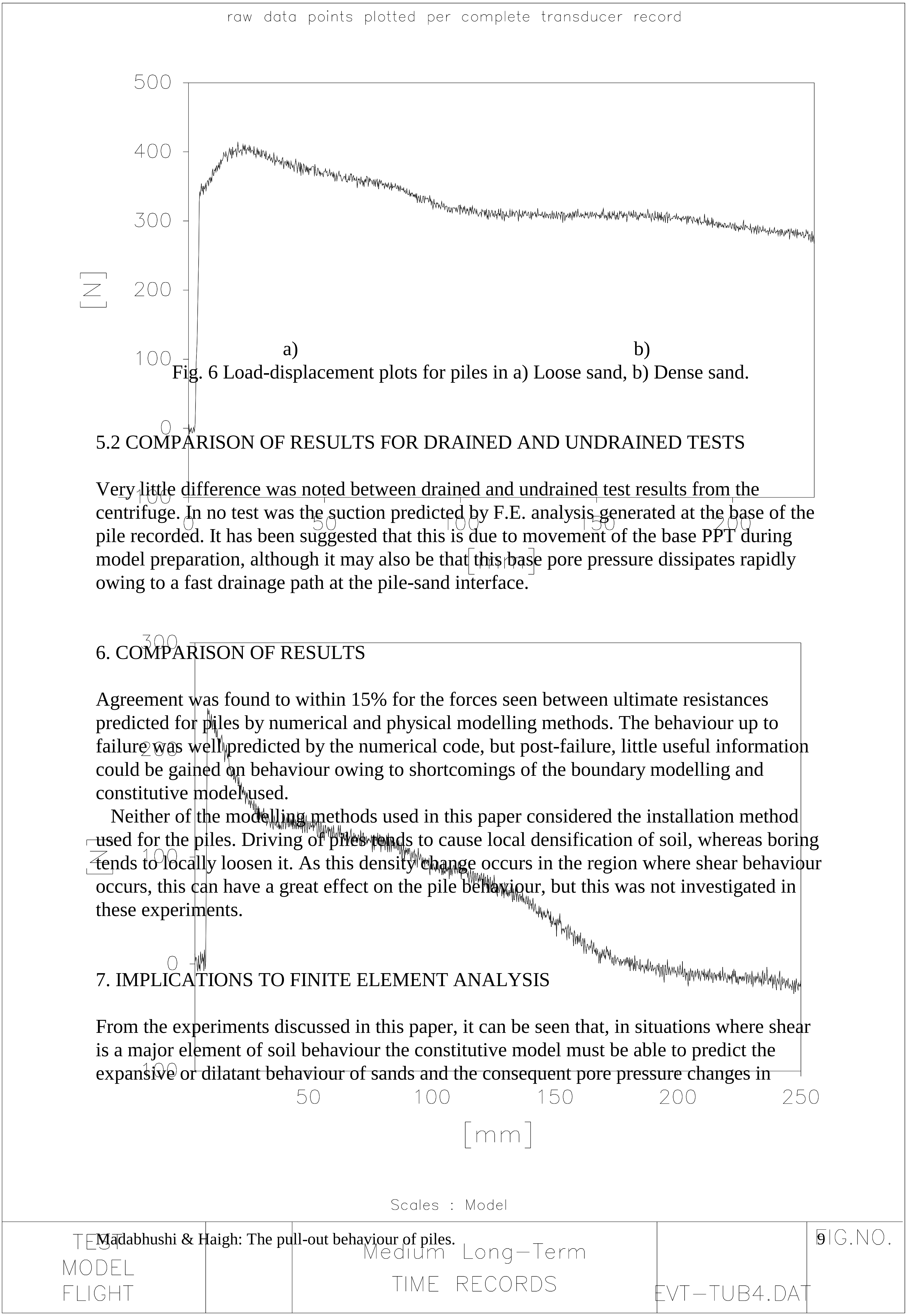
5. RESULTS FROM CENTRIFUGE TESTS

The centrifuge test results gave agreement to within about 10-15% for the forces required to achieve pull-out in each of the cases modelled. The tests however gave a great deal more in formation about post-failure behaviour than could be seen from the finite element results

5.1 COMPARISON OF RESULTS FOR LOOSE AND DENSE TESTS

As loose sands shear around the boundary of the pile, they contract to critical state, causing excess pore pressures to be generated in undrained events, lowering the effective stress and hence the skin friction experienced by the pile. Conversely, with dense sands, suction pore pressures are generated and increased skin friction is seen.

This is illustrated by Figure 6, showing load-displacement curves for loose and dense sands, in which it can be seen that the pile in dense sand shows a much more sustained resistance to pull-out, owing to the presence of suctions around the pile. These pore pressures were observed by the PPTs situated at the side of the pile.



a) b)

Fig. 6 Load-displacement plots for piles in a) Loose sand, b) Dense sand.

5.2 COMPARISON OF RESULTS FOR DRAINED AND UNDRAINED TESTS

Very little difference was noted between drained and undrained test results from the centrifuge. In no test was the suction predicted by F.E. analysis generated at the base of the pile recorded. It has been suggested that this is due to movement of the base PPT during model preparation, although it may also be that this base pore pressure dissipates rapidly owing to a fast drainage path at the pile-sand interface.

6. COMPARISON OF RESULTS

Agreement was found to within 15% for the forces seen between ultimate resistances predicted for piles by numerical and physical modelling methods. The behaviour up to failure was well predicted by the numerical code, but post-failure, little useful information could be gained on behaviour owing to shortcomings of the boundary modelling and constitutive model used.

Neither of the modelling methods used in this paper considered the installation method used for the piles. Driving of piles tends to cause local densification of soil, whereas boring tends to locally loosen it. As this density change occurs in the region where shear behaviour occurs, this can have a great effect on the pile behaviour, but this was not investigated in these experiments.

7. IMPLICATIONS TO FINITE ELEMENT ANALYSIS

From the experiments discussed in this paper, it can be seen that, in situations where shear is a major element of soil behaviour the constitutive model must be able to predict the expansive or dilatant behaviour of sands and the consequent pore pressure changes in undrained tests if adequate model behaviour is to be predicted. This is a limitation of simple soil models but is implemented in more complex constitutive models.

It can also be seen that the handling of soil-structure boundaries plays a significant role in the behaviour of numerical models of soil-structure interaction problems, especially when significant movement occurs between the two substances. Slip contact and the opening of voids are two complications which need to be dealt with if behaviour is to be modelled accurately once significant movement has occurred.

Local variations in soil properties caused by construction techniques may have a major impact on the performance of structures, these should be incorporated into the finite element model for accurate results.

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