CENTRIFUGE MODELLING OF PILE GROUPS ADJACENT TO SURCHARGE LOADS

M. F. BRANSBYⁱ⁾ and S. M. SPRINGMANⁱⁱ⁾

ABSTRACT

The behaviour of pile groups when adjacent surcharge loads are applied to a clay layer was studied using centrifuge model tests. Lateral soil movements due to the surcharge loading cause passive lateral pile pressures and traction under the pile cap. These loadings induce pile group displacement and rotation away from the surcharge load. This 3-d soil-structure interaction is complex and requires data upon which design rules can be based.

Short-term loading was found to be the most critical case for design. Results are presented together with investigation of the long-term behaviour and the new 'buttonhole' foundation technique designed to reduce passive lateral pile pressures. Soil contact beneath the pile cap increased pile group loadings and enlarged the region of soil deformation compared to previously investigated geometries in which the pile cap was above the top surface of the deforming soil.

Key words: centrifuge modelling, passive lateral pile group loading (IGC: C9/E12)

INTRODUCTION

When load is applied to a free soil surface, lateral soil displacements are generated. If there is a pile group in the region of soil displacement, the lateral movement of soil past the piles will result in, so called, passive lateral pile pressures as the soil is forced to shear between the piles (Fig. 1). These pile loadings may cause one of two problems: ultimate bending moments are developed in the pile which may lead to pile failure; lateral pile displacements are caused by this pile pressure which may exceed serviceability limits, e.g. 38 mm, as recommended by the US Department of Transportation for bridge piers (Moulton et al. 1985). The latter case is more prevalent and so it is necessary to predict the magnitude of these additional lateral pile group displacements. This is problematic since the pile group behaviour is affected by complicated 3-d soil-structure interaction, which is difficult to predict accurately.

Design methods require data for validation. Obtaining field data is costly and time consuming. Usually instrumentation is relatively sparse, it is not possible to control ground conditions and difficult to characterise the past stress history and current engineering properties. Centrifuge model tests are designed to behave as prototype foundations due to the identical vertical effective stress condition existing in the model and prototype. It is possible to instrument the model with a degree of redundancy and to choose an ideal pile group and foundation

geometry. Additionally, the soil model is prepared in a known sequence, using a laboratory soil of known properties which can be replicated accurately over a number of tests. Springman (1989) and Stewart (1992) both carried out series of centrifuge model tests to investigate the behaviour of piled foundations adjacent to surcharge loads and produced design methods based on these data for pile groups with pile caps above the surface of the soil.

Predicting ultimate passive lateral pile pressures under conservative German design rules where $p_u = 10c_u$, lead to invention of a new design method in Germany to limit these large values. The pile was surrounded by an annulus of very low shear strength (e.g. $c_u \approx 500$ Pa) bentonite in the layer of deforming soil (Fig. 2). As soil moves past the pile, only the low shear strength bentonite deforms,

Fig. 1. Pile group loading due to adjacent surcharge loading

ⁱ⁾ Research Associate, Geomechanics Group, Department of Civil Engineering, University of Western Australia, Nedlands WA 6907, Australia. Formerly Cambridge University.

ⁱⁱ⁾ Professor, Institut für Geotechnik, ETH-Zürich, CH8093 Honggerberg, Switzerland. Formerly Cambridge University. Manuscript was received for review on September 11, 1995.

Written discussions on this paper should be submitted before January 1, 1998 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.

Fig. 2. Geometry of the buttonholed foundation

Fig. 3. Prototype foundation

and negligible lateral pile pressures result (e.g. $p_u \approx 10 \times 0.5 \text{ kPa} = 5 \text{ kPa}$). This technique was named the 'buttonhole' foundation technique because the pile was said to behave like a button in a buttonhole.

This paper reports a pair of centrifuge model tests investigating the behaviour of a two row pile group when an adjacent vertical surcharge load is applied. Unlike in previous model foundations, the pile cap is in contact with the surface of the deforming soil and interaction is observed between the pile cap and the soil in addition to between the deforming soil and the piles (Fig. 1). The first test is of a conventional foundation, whereas the second test is of a buttonholed foundation. This allows direct comparison between results from conventional and buttonholed foundations and current design methods. The experimental geometry and test procedure are described prior to presenting results for three conditions: undrained loading, long-term loading and the undrained buttonhole foundation loading.

EXPERIMENTAL METHOD

The Geometry of the Prototype Studied

The soil consisted of 6 m of overconsolidated kaolin overlying a 14 m layer of medium dense sand (Fig. 3). The pile group contained two rows of 19 m long piles of external diameter 1.27 m with pile spacing 6.67 m centre to centre along the pile row and with 5 m between pile rows. The pile cap was 1 m thick and could be considered rigid in bending compared to the piles. A vertical surcharge load was applied 1 m from the edge of the pile cap.

A plane strain configuration was used so that the geometry modelled was that of an infinitely long surcharge load and pile group.

The Geotechnical Centrifuge

The model tests were carried out on the Cambridge 10 m Beam Geotechnical Centrifuge. This has a platform radius of 4.125 m and was fully described by Schofield (1980). A plane strain strongbox of dimensions 675 mm $\times$ 200 mm in plan view and 560 mm in height contained a liner which held the foundation soil and could be removed for model preparation.

The principles of centrifuge modelling and the scaling relationships were explained by Schofield (1980) and will not be given here. The model tests were carried out at a nominal radial acceleration of 100 g, so that the model would have the same vertical effective stress profile in flight as a prototype 100 times larger.

The Model Pile Group

Each of the six piles in the pile group was made from 12.7 mm (1.27 m prototype scale) external diameter, aluminium tubing of wall thickness 1.219 mm, and measured 193 mm (19.3 m) in length with an additional 7 mm (0.7 m prototype scale) brass end cone at the tip (Fig. 4). The bending rigidity $E=70 \text{ GPa}$, $I=0.0733 \text{ m}^4$, $EI=5131 \text{ MNm}^2$ was designed to be comparable with a solid reinforced concrete pile of the same diameter at prototype scale.

Four of the piles in the group were strain gauged to allow calculation of pile bending moments so that there was redundancy in the event of instrumentation failure. These were the two central piles (front central (FC), back central (BC)), and the pile pair on the right hand side of the group looking away from the surcharge load (front right (FR), back right (BR)) where the front row of the piles are defined as those closest to the surcharge load (Fig. 4).

To reduce vulnerability to mechanical damage, the need for individual waterproofing and to maintain a smooth external surface of the piles, most of the transducers (BMTs) were located internally. The strain gauges were 2 mm long, with resistance of 350 Ω . They were placed at the minimum practical spacing of 20 mm apart (Fig. 4). This was done by mounting the wired strain gauges up-side-down on a balloon at the appropriate spacing, applying glue to the strain gauges, inserting the

The pile cap was 10 mm thick and 90 mm wide. It was constructed of aluminium and was designed to be rigid compared to the piles.

The loading system was designed to simulate the vertical component of surcharge load applied by an embankment, with no shear stress applied directly to the surface (unlike the loading applied at the edge of an embankment). This followed Springman's (1989) method, using a latex bag filled with air (Bransby, 1993). The air pressure was controlled manually in flight via a hydraulic line, allowing load to be applied on demand, and this was monitored by a pre-calibrated Druck PDCR 510 pressure

The bag was greased externally and fitted inside a greased restraining box which was designed to give lateral and vertical (top face only) support to the bag, but to leave the bottom in contact with the soil surface when pressurised. The grease was applied to limit the shear stress arising if the soil moved laterally past the bag. The self weight of the bag loaded the soil initially with an equivalent uniform surcharge of 13.7 kPa at 100 g. For operational reasons, it was necessary to inflate the air bag in increments to a total pressure of 21 kPa as the centrifuge was reaching full speed. Possible soil disturbance was minimised while ensuring the air bag remained fully inflated. Therefore, an initial total vertical surcharge of 34.7 kPa was applied to the soil throughout the test at 100 g, producing excess pore pressures beneath the air bag which would have dissipated by the start of the loading sequence.

Five linear variable differential transformers (LVDTs) were positioned to measure pile group deflections. Four of the LVDTs (notation: LVDT 1, 2, 3, 4) measured horizontal displacements of the pile cap so that there was both redundancy of data and the option to calculate pile

cap rigid body rotation about 2 axes (Fig. 5). A fifth LVDT was set with a vertical axis and a footing attached to the end of the spindle to measure vertical pile group displacement (LVDT5).

A final LVDT (LVDT6) was mounted to measure the vertical clay surface movement in a region away from the surcharge. A footing was glued to the spindle and placed on top of the clay surface, to ensure that there was neither bearing capacity failure nor significant vertical deformation of the LVDT spindle caused by self weight loading under enhanced gravity.

All LVDTs were mounted on a rigid gantry which was bolted to the strongbox to create identical positioning in both tests.

Black marker pellets were placed on the front face of the clay using an orthogonal 15 mm template grid. Determination of soil displacements was achieved by differential measurement from photographs taken at various instants during the tests.

Preparation of the Model

The model preparation for the tests was designed to be as repeatable as possible. The only difference in preparation between the pair of tests was construction of the bentonite filled annuli in the second test (shown in Fig. 6).

All tests had the same foundation soil arrangement: 60 mm of medium strength kaolin ($c_u = 35\text{--}50$ kPa) overlaid a 140 mm layer of dense sand. A 10 mm thick medium, dense sand layer was placed on the clay layer to ensure drainage occurred beneath the air bag and that the effective stress at the top surface of the clay layer was not zero (Fig. 5).

Sand Substratum

Leighton Buzzard sand with 100/170 grading was used. The maximum and minimum voids ratio were found to be $e_{\max} = 0.99$ and $e_{\min} = 0.61$. The specific gravity of the particles $G_s = 2.65$.

The sand was poured slowly into the liner using a hopper to form a dense, uniform dry layer 140 mm deep. The sand was saturated from the base upwards with de-

ionised water to try to achieve full saturation. The internal faces of the consolidometer were then greased and the liner was transferred to it in preparation for adding the clay. The values quoted above are for the sample just after sand placement, but it is likely that the density would have increased slightly during the model transfer to the consolidometer for the addition of the clay layer and then onto the centrifuge arm.

Table 1. Results of the sand layer preparation

Test	Depth of sand, mm	Dry unit weight, kN/m ³	γ_{sat} , kN/m ³	Voids ratio	Relative density, %
1	140	15.7	19.8	0.68	80.5
2	140	15.7	19.7	0.69	78.1

Clay Layer

Speswhite kaolin clay powder was mixed mechanically with de-ionised water under vacuum to form a slurry at about 120% moisture content. This was then placed carefully on top of the sand layer, avoiding inclusion of air bubbles, to a depth of approximately 200 mm.

The sample was pre-consolidated in stages to a vertical effective stress, $\sigma'_v = 450$ kPa in a 1-d consolidometer loaded by an hydraulic jack. Pore pressure transducers were installed following temporary unloading from a vertical effective stress of 256 kPa before the maximum stress was applied. After equilibration at the full load, the vertical stress was reduced to 60 kPa, by controlled swelling to a similar clay specific volume to that occurring in the centrifuge test at 100 g.

Table 2. Initial clay layer properties

Test	Moisture content w , %	Permeability k_v , m/s	Unit weight, kN/m ³
1	44.5	5.3×10^{-10}	17.72
2	45.2	5.7×10^{-10}	17.66

Model Making

The vertical stress was removed and the clay was trimmed to a thickness of 60 mm, leaving a total soil depth of 200 mm. Samples were taken from the trimmings for moisture content determination (Table 2). Permeability was estimated from records of settlement obtained during consolidation.

A custom-made apparatus was designed to auger a borehole for each buttonhole and to control the subsequent pile group insertion. The pile and buttonhole were to be concentric, so the lateral movement of the pile group had to be minimised during placement.

Lubricated 20 mm outer diameter tubes were pushed in stages through the guide plate into the clay to a depth of 55 mm at the position of each pile to begin creating the buttonhole. A clay auger was screwed downwards inside the tube so that the clay within each tube was removed. Preliminary tests showed that suction in the clay would re-

Fig. 6. Detailed pile cap-buttonhole arrangement

move water from the bentonite suspension into the clay layer, drying out the bentonite suspension, which would crack. Clingfilm was placed on the wall of the boreholes to ensure the bentonite remained at the original moisture content ($\approx 600\%$) in the hole. The clingfilm modelled the filter cake which would form in a prototype foundation but which would not scale correctly in the centrifuge. The shear strength of bentonite increases with time since last disturbance and the suspension was designed to have a shear strength of 500 Pa during the test. Approximately 9 cm³ of newly disturbed bentonite suspension of concentration 170 g of bentonite to 1 litre of water was then syringed into each borehole by a tremie method (Fig. 6).

The pile group was pushed into the soil using the guide apparatus until the pile cap base was touching the top clay surface. This required a force of 510 N per pile, as measured by the pile axial load transducers.

The annuli were then fully filled with bentonite by using a hypodermic needle to tremie through the top-up holes in the pile cap. This eliminated air gaps in the annulus (Fig. 6). The top-up holes were closed to ensure there was no further water loss from the bentonite due to evaporation before the test.

Final Preparation

The front perspex window was sprayed with lubricating oil and the strongbox was assembled with a manometer and digital clock in view of the monitoring cameras.

A pre-weighed amount of Leighton Buzzard 100/170 sand was then poured above the clay layer to form the 10 mm deep medium dense top sand layer. This was designed to apply a uniform surcharge of 17 kPa to the clay layer.

The water supply and drainage systems were set up so that the water table remained 2 mm (model) above the clay top surface in the centre of the model. The LVDTs were positioned and the air bag and restraining box located adjacent to the pile group. The instrumentation was plugged in and final checks were made before the package was weighed and bolted firstly onto the swinging platform and then onto the centrifuge arm.

Pore pressure readings revealed suctions of only about 5 kPa in the clay prior to centrifuge flight. The clay swelled significantly during model preparation.

Centrifuge Test Procedure

Data acquisition

71 channels of data (including 19 multiplexed channels) were recorded in 9 bit ASCII format throughout the tests at a variable logging rate ranging from 2 Hz to 0.01 Hz. The data acquisition system was also able to plot graphs of calibrated data against time in-flight so visual checks could be made during the test.

Consolidation

The centrifuge was accelerated to a constant radial acceleration field of 100 g, and the soil was subject to excess pore pressures in the clay corresponding to the increase

in total stress due to the increased self weight of the soil. About 2 hours (2.28 years prototype) was required to allow equilibration of pore pressures in the clay layer before the surcharge loading phase could be initiated.

Undrained surcharge loading

Prior to the surcharge loading stage of the test, reference photographs were taken of the front face of the package. The air bag pressure was increased in a continuous loading increment over 2 minutes (13.9 days) duration from 20 kPa to a maximum bag pressure of about 230 kPa (corresponding to a maximum net surcharge load, $q \approx 210$ kPa). After each 15 seconds during the loading phase, a photograph was taken of the package cross section, and all channels were logged continuously at a rate of 1 Hz.

Drained/long term surcharge loading

The maximum air bag loading was then maintained for up to 2 hours (2.28 years prototype), while the logging rate was reduced. Manual readings were taken at 15 minute intervals and pore pressure dissipation was observed until equilibration was achieved under the surcharge loading. Photographs were taken of the cross-section to determine the long-term deformation behaviour of the clay.

Post-test site investigation and radiography

In the buttonholed test, the clay was carefully removed near the buttonholes so that the final annuli positions could be inspected and measured relative to the pile group.

Stress History of the Clay

The stress history of the kaolin is complicated. Initial 1-d consolidation up to $\sigma'_v = 450$ kPa, $\sigma'_h = 357$ kPa ($p' = 388$ kPa, $q = 93$ kPa, point A on Fig. 7), where $K_{onc} = 0.69$ (Al-Tabbaa, 1988), is followed by swelling to $\sigma'_v = 60$ kPa ($p' = 90.3$ kPa, $q = 45.5$ kPa, point B on Fig. 7). On dismantling the consolidometer and removal of the liner, all total stress is removed and so only suction remains, hence $p' = -u = 90.3$ kPa, $q = 0$, (point C on Fig. 7) where u is the pore pressure. However, the pore pressure transducer readings after model preparation, but prior to centrifuge swing-up, revealed suctions of only about 5 kPa. This implies that the soil had swollen to a very low effective stress, with some lateral total stress due to the rigidity of the strongbox walls (point D on Fig. 7). These small effective stresses prior to testing are confirmed with the measurement of clay settlements during self-weight consolidation at 100 g of 2.3 mm (model scale) for the 60 mm (model) clay layer. From D to E the incremental loading path will be initiated by increasing vertical stress, described here as the active mode ($K < 1$) and so the path DE on Fig. 7 is likely to be followed with $K_o \rightarrow K_{onc}$. Considering the small effective stresses prior to swing-up, $K_o \approx K_{onc}$, so despite the high overconsolidation ratio of the clay, the clay is in active mode and the most recent load path will also have been active.

Fig. 7. Stress history of the kaolin in the centrifuge model test

RESULTS

Three types of system behaviour are investigated in the tests: the almost undrained response to the surcharge loading, long-term drained behaviour and the performance of the buttonhole foundation. The undrained loading will be studied as the primary loading case as this was found to be the critical case for design.

All units are given at prototype scale unless otherwise stated.

Pile Group Behaviour

Sign convention

A positive pile displacement will describe pile movement away from the surcharge load. A front row pile is one which is nearest the surcharge load. Bending moments which cause tension in the front side of the pile will be deemed positive. Positive shear forces and pressures are those which act in the direction away from the surcharge load i.e. causing positive pile group displacements (Fig. 8).

Analysis of individual pile behaviour

Signals from the bending moment transducers along each pile were taken as the primary source of data. Each pile data set was examined separately, representing the

Fig. 8. Pile group sign convention

pile loading at one surcharge load or time. A program was written to fit two polynomials to the bending moment data in a least squares sense, describing the continuous bending moment distribution profile with length along the pile (Fig. 9). One polynomial described the moment distribution in the clay layer (the top 60 mm (model scale) below the pile cap), the second fitted the moments in the sand. The polynomials were required to have an equal value and slope at the clay-sand interface and to give zero moment and shear force at the pile tip. The order of polynomials used to fit the data depended on the number of bending moment transducers functioning on each pile in each layer. Typically a 5th order polynomial was used in the sand layer and a 3rd order was applied in the clay.

From the continuous moment distribution curve, the single and double differentials of the moment, M , with respect to length along the pile, z , were found. From linear elastic beam bending theory, the first derivative is the shear force distribution, $F(z)$ and the second derivative divided by the pile diameter, d , is the lateral pressure distribution, $p(z)$. i.e.

$$p(z) = \frac{1}{d} \frac{d^2 M}{dz^2}.$$

Differentiation of these 3rd and 5th order polynomials gave a linear distribution of lateral pile pressure in the clay layer, and a cubic distribution of pressure in the sand (Fig. 9). Although the bending moment transducers appeared to give both consistent and repeatable results, with little drift or noise, the process of curve fitting and taking the second derivative mean that the deduced pressure distribution will be an indication rather than an accurate value.

Double integration of the moment distribution gave the lateral pile displacements due to bending of the pile alone. The net lateral pile displacement distribution, $\delta_p(z)$ of a pile of length l would also depend on the tip rotation, θ_t and tip displacement, δ_{pt} :

$$\delta_p(z) = \delta_{pt} + \theta_t(l-z) + \int_l^{(l-z)} \frac{M}{EI} \cdot dz \cdot dz.$$

The pile group movements predicted from the LVDT

Fig. 9. Test 1 front row pile behaviour during loading

data showed that there was significant pile rigid body rotation and translation as well as bending. Direct determination of the rotation and translation of the piles from curve fitting the LVDT data was attempted, but was only partially successful because of LVDT signal noise. At low displacements when the LVDT signal errors were more significant, results seem unrealistic but appear more accurate at higher loads when rigid body rotation is indicated about a point 20–30 mm above the pile tip. The pressure distributions along the piles (Fig. 9) show zero pressure at approximately the same depth. It was assumed that the point of pile rotation, where there was no pile translation, occurred at this null pressure depth. The final pile displacement profile was produced by adding a rigid body rotation about the null point to the pile bending to fit the average LVDT displacement.

Short term pile group loading

Profiles of pile bending moment and deduced shear force, pile pressure and displacement for each pile row are shown in Figs. 9 and 10 for increasing surcharge loads during the loading stage.

The average lateral pile pressure in the clay layer for each row is plotted against surcharge load in Fig. 11. Front row pile pressure increases linearly with respect to surcharge load, whereas the back row pressure increase is non-linear. The pressures are almost equal at a surcharge load of 120 kPa.

The shear force applied to the pile cap and the shear

Fig. 10. Test 1 back row pile behaviour during loading

Fig. 11. Test 1 pile pressures during loading

Fig. 12(a). Test 1 pile shear forces at pile cap during loading

Fig. 12(b). Test 1 pile axial forces during loading

force applied to each pile row at pile cap level (from which the total shear force was calculated) is shown in Fig. 12(a). The difference in loading between the front and back pile row is due to the lateral pressure regimes for the front and back row piles and pile-soil-pile interaction in the sand layer. The pile interaction causes the lateral soil response for the back row pile to be stiffer than the front row so that larger shear loading of the back row piles is required for equal displacement with the front row. This is confirmed by comparing Figs. 9 and 10.

In the absence of vertical structural loading on the pile cap, uplift loading of the pile group can be deduced from the pile axial load cell data. There is negligible axial force applied to the back row but a maximum force of 5000 kN applied to the front row piles (Fig. 12(b)). This implies an average uplift stress on the bottom surface of the pile cap of 78 kPa for a surcharge load of 210 kPa.

Displacement Behaviour

Pile group displacements are plotted against surcharge load increase in Fig. 13. There is significant non-linearity in both the displacement and rotation behaviour with respect to surcharge load. Little pile group movement occurs until a surcharge load of about 70 kPa, but the recommended lateral displacement of 38 mm (Moulton et al. 1985) is exceeded at a surcharge load of 125 kPa.

The lateral response of the pile group in the sand layer (Figs. 9 and 10) was significantly softer than would be expected using conventional active loading design methods (Randolph, 1981). From these, it is predicted that the

Fig. 13. Test 1 pile group displacements during loading

piles would behave as if they were long, i.e. there would be a depth beneath which negligible lateral pile displacement or rotation would have occurred. However, in the centrifuge tests, rigid body pile rotation was deduced. In 3-d finite element analysis (Bransby, 1995) the piles rotated but the sand layer also deformed laterally with application of surcharge load (Fig. 14) resulting in additional lateral pile displacements. This suggests that there is combination of pile rotation about the null pressure point and translation due to lateral deformation of the sand layer caused by surcharge loading (Fig. 14).

Figure 15(a) shows vectors of clay displacement due to surcharge load application, measured from markers on

Fig. 14. Pile and soil displacement profiles

Fig. 16. Test 1 clay inclinometer plots at $q=200$ kPa

the front face of the clay. Figure 16 shows profiles of lateral soil displacement at the position of each pile row, whereby lateral soil displacement past the front pile row is larger than past the back row due to vertical soil movement beneath the pile cap. This movement has two causes. As the sand is loaded by the surcharge, there is some settlement at the top of the sand layer. Since the clay deforms in an undrained constant volume state, less lateral movement will occur along the centreline of the rear row of piles. It is also likely that there is a small gap beneath the pile cap which would have formed during consolidation of the clay at 100 g. When surcharge load is applied there is upwards deformation of the clay to close the gap. This mechanism is prevalent at small surcharge loads, accompanied by more lateral soil displacement at the position of the front row piles than the back row (Fig. 17(a)). However, at larger surcharge loads, when the gap is closed, the incremental soil displacement past each pile row will be almost equal (Fig. 17(b)).

Despite the difference in lateral soil displacements at the position of each pile row, back row pile pressures are comparable to front row pressures at surcharge loads larger than 130 kPa (Fig. 11). This implies that the pile pressure-relative soil displacement behaviour is stiffer for the back row pile than for the front row. The reason for this behaviour has been investigated by Bransby (1995) us-

Fig. 15. Test 1 clay displacement vectors

Fig. 17. Deformation mechanism change due to gapping

ing 3-d finite element analysis.

Time Dependent Behaviour

The pile group remained almost stationary after the initial application of full surcharge load (Fig. 18). Despite this, profiles of bending moment, shear force and lateral pile pressure change significantly with time (Figs. 19 and 20) and lateral soil movements occur (Fig. 16).

Pressure on the front row piles increases with respect to time whereas back row pile pressures reduce (Fig. 21). Pile-soil-pile interaction leads to an increase of average total stress around the front row piles and a decrease of total stress around the back row piles (Bransby, 1995). If drainage occurs due to the spatially varying excess pore pressures generated near to the piles, soil will consolidate and stiffen around the front row piles but soften slightly around the back row. Thereafter, the pressure-relative displacement response of the front row will be stiffer than that of the back row. As the soil consolidates under the surcharge load, there are further lateral soil movements past the piles. Lateral soil displacements at the position of the back row piles are smaller than at the position of the front row piles because soil consolidates between the pile rows. This effect, together with the disparity in the stiffness of the pressure-relative displacement behaviour of the pile rows, ensures that pressures on the front row piles increase with time, but those on the back row reduce.

Total pile cap shear force decreased with time to almost zero (Figs. 19 and 20). Back row pile shear remained almost constant but front row shear decreased to a negative value. At the same time, net uplift force on the pile cap reduced to almost zero as the soil consolidated beneath the pile cap. Contact may have been lost between the clay and the pile cap as the soil consolidated under the surcharge load.

The overall group behaviour is the result of a balance between the decreasing shear force applied to the pile

Fig. 18. Test 1 pile group displacements with time

Fig. 19. Test 1 front row pile behaviour with time

Fig. 20. Test 1 back row pile behaviour with time

Fig. 21. Test 1 pile pressures with time

cap, the increasing passive pressure on the front pile row and the decreasing passive pressure on the back row piles. For the geometry studied, the net result is that the bending moments on each pile row reduce and there is negligible further pile group movement, probably dictated by the lateral sand group movement and irreversible rigid body rotation of piles. For other geometries, for example the deep soft clay layers studied by Stewart (1992), the balance between the various loading mechanisms will be different and so pile group displacements and bending moments may increase with time.

The Behaviour of the Buttonholed Foundation

Profiles of bending moment, shear force, pressure and pile displacement during loading are shown for the buttonholed foundation in Figs. 22 and 23 for front and back row piles. Lateral pile pressures are lower with respect to those from the conventional foundation (Fig. 24) from an average value of 125 kPa to 40 kPa at a surcharge load of 200 kPa. The buttonholes were functioning correctly, shielding the piles from a majority of the passive lateral pressure with an accompanying drop in the pile bending moments. However, the overall net shear force applied to the pile group was larger than for the conventional foundation. The reduced pile pressures led to smaller lateral pile group displacements and in-

Fig. 24. Centrifuge test pile pressures during loading

Fig. 25. Centrifuge test pile group displacements during surcharge loading

Fig. 22. Test 2 front row pile behaviour during loading

Fig. 23. Test 2 back row pile behaviour during loading

creased lateral soil displacement (because of less 'reinforcement' by the piles) which produced larger relative soil movement past the pile cap.

Using the new technique, Fig. 25 shows the significant reduction in the lateral pile group displacement from 0.077 m to 0.043 m at a surcharge load of 200 kPa (Fig. 25). The threshold surcharge load for 38 mm displacement is 188 kPa for the buttonhole foundation compared to 125 kPa for the conventional foundation.

CONCLUSIONS

Highly instrumented and carefully controlled centrifuge model tests were carried out to investigate the behaviour of a pile group when an adjacent surcharge load was applied. Passive lateral pile pressures, shear transfer to the pile cap and uplift loads combined to induce bending moments and lateral pile displacements. The long and short-term behaviour of a conventional foundation and the new 'buttonhole' foundation designed to limit pressures and displacements were studied.

For both foundations, the maximum pile bending moments and pile group movements occurred immediately on application of the surcharge load. The latter were larger than recommended serviceability limits at $q=200$ kPa. There was an increase in front row lateral pile pressure with time.

The pile cap was designed to be in contact with the top surface of the deforming soil which increased pile group loading compared to previous studies when the pile cap

was above the surface of the soil (Springman, 1989; Stewart, 1992) by two mechanisms. Firstly, shear force was applied to the pile cap as soil sheared beneath. Secondly, the pile cap constrained the undrained soil displacement field, focussing greater soil deformations and hence pile pressures on the back row of piles compared to previous tests when the pile cap was above the surface of the soil (Springman, 1989; Stewart, 1992).

In these centrifuge model tests, the boundary conditions were further altered because a gap formed beneath the pile cap during consolidation prior to surcharge loading. This was closed by soil displacement under small surcharge loads and will have affected both the surcharge load-shear force relationship and the soil displacement field. The boundary conditions were initially those of a pile cap above the surface and finally of a pile cap on the surface. This highlights the importance of the pile cap boundary conditions on the overall foundation behaviour. It is believed that the tests modelled the prototype foundation at $q > 50$ kPa.

Lateral pressure on the back row pile was lower than on the front row for $q < 130$ kPa after which the loading on each pile row was almost equal. As smaller lateral soil displacements were measured at the position of the back row pile, the pressure-relative soil displacement relationship for the back row piles was stiffer than for the front row.

The behaviour of the piles in the sand layer was softer than expected. This was caused by lateral sand movement due to the surcharge loading. This gave large lateral pile displacements in the sand layer, dependent on both the active pile loading and the surcharge load. This has been neglected in the past when it was assumed that the stiffer sand layer was unaffected by the surcharge load.

The new buttonhole foundation technique reduced passive lateral pile pressures as intended, but increased pile cap shear force. Nevertheless, pile group displacement dropped from 0.077 m for the conventional foundation to 0.043 m for the buttonhole foundation at $q = 200$ kPa. This suggests buttonhole foundations will reduce lateral pile group displacement due to passive lateral pile loading.

ACKNOWLEDGEMENTS

The centrifuge tests reported were carried out in Cambridge as part of an E.C Science twinning contract with Bergische Universität, Wuppertal and the authors would like to thank Arndt Kremer and Kay Hock-Berghaus for

help during the preliminary centrifuge tests. The first author was studying with financial support from the Engineering and Physical Sciences Research Council.

NOTATION

c_u	undrained shear strength
d	external pile diameter
D	diameter of buttonhole
e	voids ratio
$e_{\max}$	maximum voids ratio
$e_{\min}$	minimum voids ratio
E	Young's modulus
F	shear force
g	acceleration due to gravity
I	second moment of area (of pile)
K	coefficient of lateral earth pressure
K_0	coefficient of lateral earth pressure at rest
K_{onc}	coefficient of lateral earth pressure at rest for 1-d compression
k_v	vertical soil permeability
M	moment
l	embedded pile length
p	passive lateral pile pressure
p'	mean effective stress
q	vertical surcharge
q	deviatoric stress
w	moisture content
z	depth below top surface of soil
δ_p	lateral pile displacement
δ_{pt}	lateral pile tip displacement
γ_{sat}	saturated unit weight of soil
σ'_h	horizontal effective stress
σ'_v	vertical effective stress
θ	rotation of pile
θ_t	rotation of pile tip

REFERENCES

- 1) Al-Tabbaa, A. (1988): "Permeability and stress-strain response of speswhite kaolin," Ph.D thesis, Univ. of Cambridge.
- 2) Bransby, M. F. (1993): "The buttonhole foundation technique I," Cambridge Univ. Centrifuge data report.
- 3) Bransby, M. F. (1995): "Piled foundations adjacent to surcharge loads," Ph.D thesis, Univ. of Cambridge.
- 4) Moulton, L. K., GangaRao, H. V. S. and Halvorsen, G. T. (1985): "Tolerable movement criteria for highway bridges," Final report FHWA/RD-85/107, Federal Highway Administration, USA.
- 5) Randolph, M. F. (1981): "The response of flexible piles to lateral loads," *Géotechnique*, Vol. 31, No. 2, pp. 247-259.
- 6) Schofield, A. N. (1980): "Cambridge University Geotechnical Centrifuge Operations," Rankine Lecture, *Géotechnique*, Vol. 30, No. 3, pp. 227-268.
- 7) Springman, S. M. (1989): "Lateral loading on piles due to simulated embankment construction," Ph.D thesis, Univ. of Cambridge.
- 8) Stewart, D. P. (1992): "Lateral loading of piled bridge abutments due to embankment construction," Ph.D thesis, Univ. of Western Australia.