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MODELING THE BEHAVIOUR OF PILES SUBJECTED TO SURCHARGE LOADING

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ABSTRACT: Construction of embankments behind piled bridge abutments on soft clay deposits leads to lateral deformation of the soil, inducing pile bending moments and displacements additional to those arising from imposed structural loads. A series of centrifuge model tests was undertaken to investigate the effect of such surcharge loading on a single row of free–headed piles and on a pile group. A parametric study was conducted with different pile spacings for the free–headed piles, two types of head fixity and three foundation layouts.

1 INTRODUCTION

The construction of bridge approach embankments on compressible subsoil can induce lateral loading on the piled foundations, causing bending and shear in the piles together with rotation and translation of the abutment. In a typical scheme, the piles pass through a soft layer and are founded within a stiffer substratum. At present, the design guidelines for piled abutments under these conditions are based on mainly empirical assumptions (De Beer & Wallays, 1972; Frank, 1981). Fundamental understanding of the soil–pile interaction may be greatly assisted by small scale simulation in a centrifuge (Springman & Bolton 1990).

A series of centrifuge model tests were carried out by the Engineering Department of Cambridge University as part of a research programme on this topic for the U.K. Transport and Road Research Laboratory. The configurations were designed to model the performance of a piled full–height bridge abutment, and were necessarily simplified in plane idealisations to enable realistic modelling and more direct comparison and analysis.

2 STRUCTURAL IDEALISATION

Initially, the performance of a single row of vertical piles, driven through a soft layer of soil and embedded in a stiffer substratum, was investigated so that the essentials of soil–pile interaction could be clearly appreciated (Bolton, Springman & Sun, 1990). Thereafter, the prototype case (Figure 1) was more closely replicated by two rows of vertical piles, fully fixed into a rigid pile cap which was, for experimental expediency, positioned just clear of the ground surface (Springman, 1989).

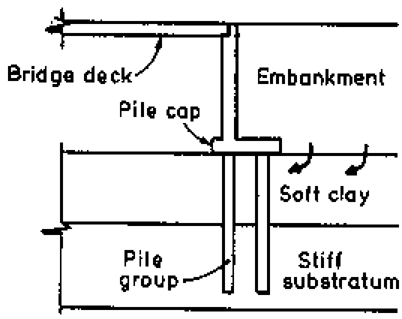


Figure 1 A typical full–height bridge abutment

Previous attempts to pour a sand embankment in–flight had been hampered by lack of control over the sand placement in the critical area around the piles (Springman, 1984). Instead, the embankment was modelled by an equivalent normal surcharge load. This simplification avoids the influence of the magnitude and direction of shear stresses at the embankment–soil foundation interface, which would affect not only the lateral soil deformation profile but also the ultimate bearing capacity of the clay foundation (Jewell, 1987).

It also neglects the thrust of the backfill on the abutment wall, which provides the primary source of lateral loading on the pile cap. This is usually resisted by raking piles. Since the objective was to use model data to generate a new design calculation for the pile–soft clay interaction, vertical piles were used and the behaviour of the embankment material and the precise geometry of the reinforced concrete substructure were eliminated from an already complex soil–structure system. Vertical loading on the pile cap is also omitted on the conventional assumption that lateral and axial loading behaviour of vertical piles can be uncoupled.

Having established hypothetical design calculations on the basis of simplified sub–systems, it always remains open to the design body to commission validation trials on complete models subjected to critical, but realistic, load combinations. It is likely, however, that authoritative validations would have to be performed at prototype scale in the field.

Except for problems in which the soil behaviour will depend on the peculiar geological history of a particular deposit, for example, pile response in calcareous soils (Nunez et al, 1988), it is common to use a restricted set of soils in the laboratory, whose properties are well known and carefully documented. However, the soil stress history, and hence density and strength, expected in the prototype should be reproduced as closely as possible in the model.

3 TEST PROGRAMME

The soil–pile interaction was examined in a series of 1/100 scale model replicas of idealised prototypes of piled full–height bridge abutments, at a nominal acceleration and radius of 100 g and 3.975 m respectively on the Cambridge Geotechnical Centrifuge (Springman, 1989). The principles, geometry and working practices pertaining to this centrifuge are described in detail by Schofield (1980), and the experimental errors associated with these techniques are discussed in Springman (1989).

At prototype scale, a 20 m width of abutment was modelled such that plane strain conditions were obtained overall, with either one row of free–headed piles (1.27 m diameter, spacing 4 m or 6.67 m, Figure 2a) or a pile group (2 rows of 1.27 m diameter piles, 5 m apart at 6.67 m spacing, Figure 2b) driven through 6–8 m soft clay into a 8–10 m layer of either sand or stiffer, heavily overconsolidated clay.

To achieve the designated strengths, densities and equivalent stress history, the soil strata were prepared in a liner, 200 x 675 mm in plan, and subjected to a known overconsolidation cycle in a consolidometer. The liner was transferred to a strongbox with a thick, stiff Perspex front window and lubricated internal faces, for testing at 100 g.

3.1 Soil model preparation and stress history

A medium and uniformly sized, sub–rounded silica sand (Leighton Buzzard 30/52) was poured into the liner to form a medium–loose deposit of either 80 or 100 mm depth, before slow saturation by upward flow of de–ionised water. Black marker pellets

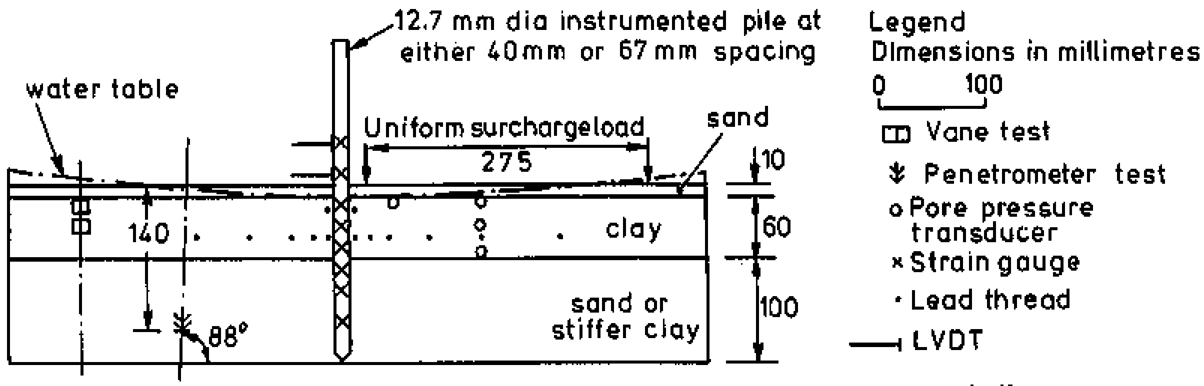


Figure 2a General arrangement of a centrifuge model: single row of free–headed piles

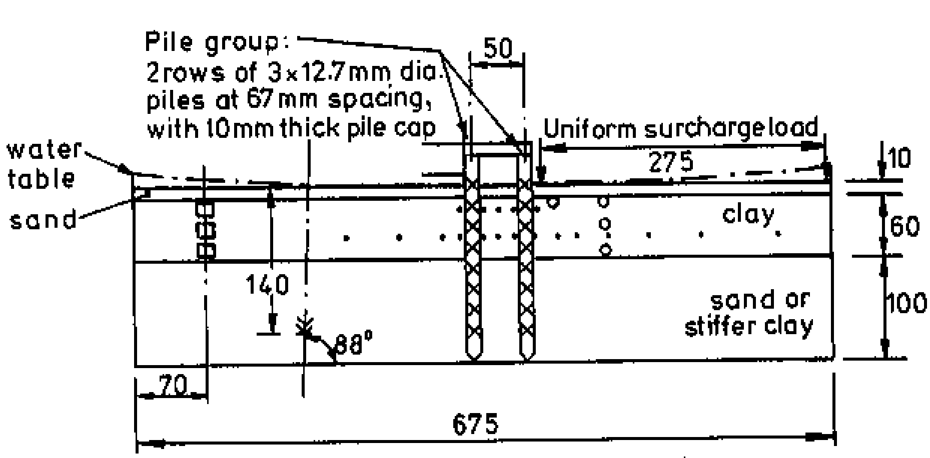


Figure 2b General arrangement of a centrifuge model: pile group (Legend shown above is relevant to both Figures 2a & 2b)

were placed at certain depths to facilitate observations of deformation. Dry sand weight and volume filled were noted, and an average dry unit weight of 15.5 kN/m^3 and a relative density of 50 % were calculated. The liner was transferred to the consolidometer.

Kaolin slurry was mixed mechanically under vacuum with de–ionised water to 120 % moisture content before placing on top of the stiffer substratum to approximately three times the pre–flight required depth. The sample was then consolidated in stages over a period of three weeks to a maximum consolidation pressure of 86 kPa, before trimming to the required depth. At this time, the moisture content averaged 58% and the saturated unit weight was approximately 16.4 kN/m^3. After reconsolidation at 100 g in the centrifuge, the overconsolidation ratios prior to loading were calculated to be 10.5 close to the top of the layer, 1.9 and 1.4 at model depths of 60 mm and 80 mm.

To model a deep clay deposit, the sand foundation was replaced by a stiff clay layer. This substratum was prepared in a similar fashion to the soft, upper clay layer; the sample was consolidated under 254 kPa and trimmed back to 100 mm depth before the soft clay was placed as before. The overconsolidation ratio at 100 g was approximately 5.3 at the top of the stiff clay, and 2.3 at the base.

A 10 mm deep sand layer was loosely poured on top of the model prior to centrifuge model testing. This prevented unlimited swelling or desiccation of the clay surface, and enabled the water table to be established at the height of the top of the clay surface at the location of the piles.

3.2 Surcharge load

Surcharge loading was applied by means of an inflatable reinforced latex rubber bag, which replicated a prototype load area of 20 m x 27.6 m. Allowance was made for the vertical settlement of the bag by incorporating an extension flap. The bag was constructed using liquid latex, reinforced by open weave first aid dressing, initially retained in a plywood mould and heated at 60°C until set. In position on the model, the bag was restrained from horizontal deformation on all sides by an internally greased steel box, which permitted base settlement only. The bag was subject to measured air pressures up to 250 kPa.

3.3 Pile design and installation

The piles were made from 12.7 mm diameter, 1.219 mm thick aluminium alloy with a 10 mm long driving cone at the tip and a bending rigidity scaled to a reinforced concrete pile of the same diameter. Tinsley (120 Ω – 3/120/PC23) strain gauges were glued externally to the piles and sealed with an acrylic moisture barrier and two layers of plastic shrinkfit tubing.

For the free–headed pile tests, half bridge circuits were installed on the pile, with the dummy resistors located in the junction box on the top of the package, to give bending moment data (following calibration) at 8 levels on two of the piles. The operating power supply was 3 V and the signals were amplified 100 times before passing through the sliprings. The piles were 300 mm long and embedded to 170 mm depth.

The piles for the group were shorter, 200mm, embedded as before to 170 mm depth. A solid cylinder of aluminium alloy was glued inside the top 30 mm of each pile to increase the bending rigidity above the soil surface and to allow the piles to be fixed to the stiff, 10 mm thick, aluminium alloy pile cap.

Pairs of strain gauges were glued alongside each other on the front and rear faces of four of the six piles and were connected in a full bridge circuit to measure bending moments at 8 depths. The drift of offset at zero load, of the calibration constant, and other errors due to temperature changes were reduced by this measure. Signals were multiplexed and amplified by 100 in the package junction box.

A template rig was used to position the piles correctly, at both tip level and 100 mm higher, to minimise misalignment and rotation from the vertical, prior to jacking in at 1 g. The single piles were installed separately; the entire group was jacked in together. Whilst this did not replicate the 'field' stress–strain conditions, Barton (1982) had achieved good agreement using a similar technique during modelling of models for lateral load tests on piles in sand. The region of high strain closest to the ground surface, where there is less difference between 1 g installation stresses and insitu stresses at 100 g at shallow depths, governs pile behaviour during lateral loading. This modelling variation is less important than for axially loaded piles.

3.4 Additional instrumentation

Sangamo linear variable differential transformers (LVDTs) were used as displacement transducers, and were fixed to the piles or pile cap above ground level. The locations are shown on the general arrangements (Figures 2a and 2b).

Druck pore pressure transducers (PDCR81–350 kPa) were installed in the clay to monitor pore pressures during the final phase of consolidation at 1 g and in–flight at 100 g.

Lead threads were inserted laterally into the clay parallel to the rows of piles during the final pre–flight model preparation, for exposure to X–rays after test completion. These radiographic techniques were developed by James (1973) and are used to investigate the internal soil movement close to the piles.

In–flight observation of the deformation of the clay layer was aided by a grid of black marker pellets and painted reference lines, which were also prepared following 1 g consolidation.

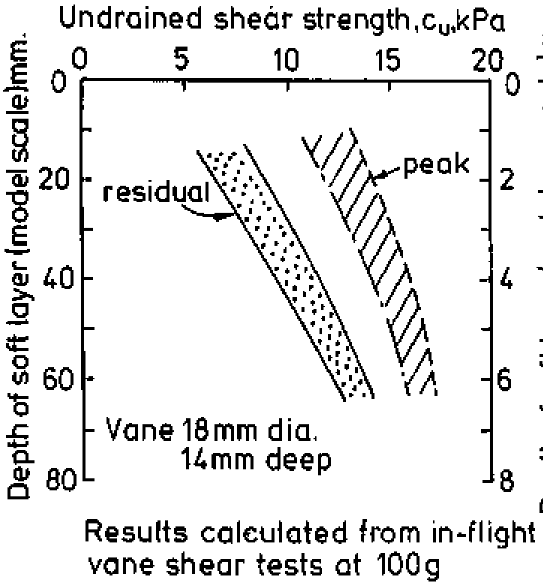


Figure 3 Vane strength profiles

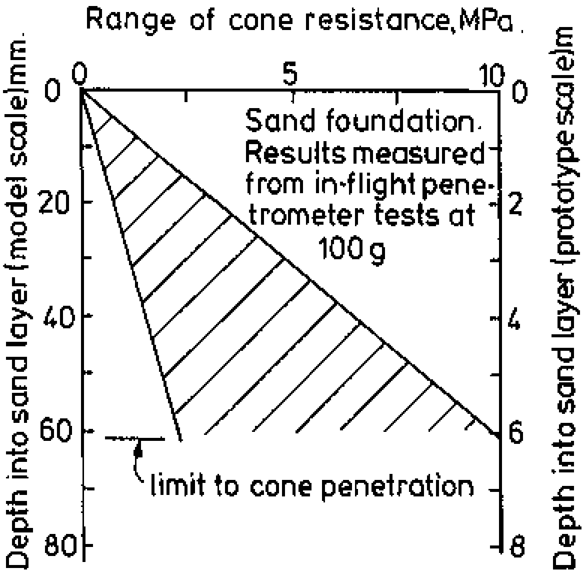


Figure 4 In–flight cone penetrometer tip resistance

3.5 Site investigation

Vane shear tests were conducted in–flight (Davies & Parry, 1982) at various depths in the soft clay layer to determine the peak and residual undrained shear strength profiles (Figure 3). An 18 mm diameter by 14 mm deep vane, nominally rotated at 72 °/minute, was recommended following parametric studies by Almeida and Parry (1983). The strength derived from the measured torque was ascribed to the mid–height of the vane. Time elapsing between completion of penetration and commencement of rotation was always less than 1 minute.

Similitude between field practices and model testing procedures is complicated by the differing representation of time in a centrifuge for strain rate and diffusion effects (Springman, 1989).

In the absence of in–flight pressuremeter data, and hence information on strain history in relation to future strain directions (Bolton & Sun, 1991), it is difficult to assess soil stiffness. However, in the present study, the movement of the soft clay will be inversely proportional to its stiffness, whereas the pressure applied to the pile will be proportional to the product of stiffness and relative displacement (Fleming et al, 1985). Therefore to a first approximation, soil stiffness cancels out of the calculation for lateral pressure due to surcharge. A semi–empirical correlation between soil stiffness and strength was therefore considered sufficient.

A 12.7 mm diameter penetrometer was also used to probe the soil model but the pore pressure measurement in the clay layer was not effective. This is required to correct for pore pressures acting on the annulus behind the cone, which were apparently equivalent to the cone resistance in the clay layer. Therefore cone tip resistance is reported for the permeable sand layer only (Figure 4). Increased values of cone resistance may be expected within 10 probe diameters of the base of the liner (Phillips & Valsangkar, 1987), which may therefore affect the entire sand layer in this suite of tests.

The major benefit of these small scale site investigation devices lies in the ability to measure the repeatability of certain soil properties in terms of consistency of soil model preparation.

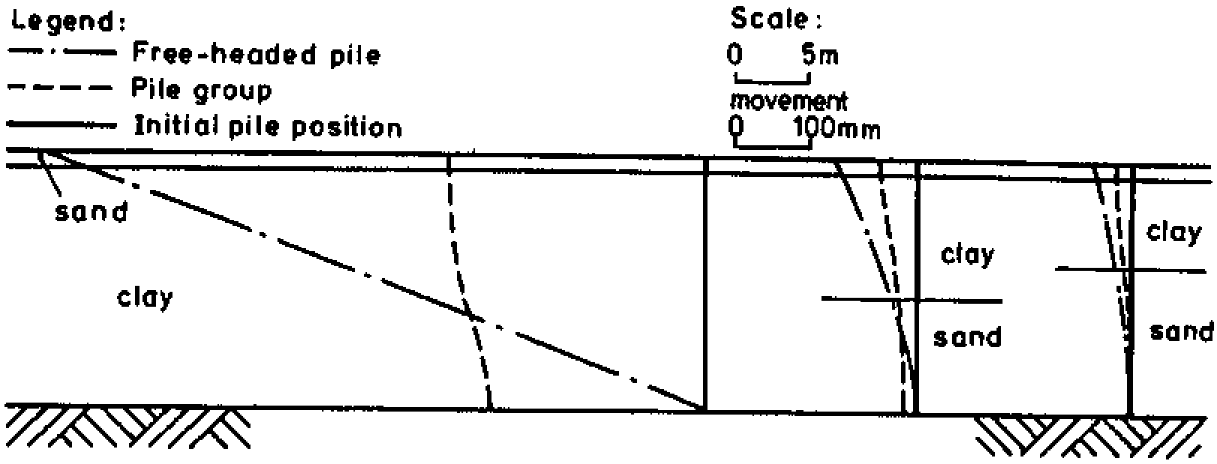
3.6 Testing procedures

Following dissipation of pore pressures induced by the stress field imposed by a centrifugal acceleration of 100 g, surcharges were applied in approximately 20–25 kPa increments, roughly every 14 days at prototype scale, with unload–reload loops at (nominally) 100, 150, 200 and 250 kPa, with consolidation pauses of around 6 months at 100 and (nominally) 200–250 kPa.

The vane and penetrometer tests were conducted following completion of the embankment construction.

4 PILE RESPONSE

When a compressible foundation is surcharged by an embankment, the soft soil deforms further than the piles, inducing passive lateral thrust on them, which is resisted by the lower section of pile embedded in the stiffer substratum. The magnitude of this thrust is largely dependent upon the differential soil–pile displacements and the stiffness of the soft soil. Knowledge of this lateral pressure distribution allows evaluation of the pile bending moment and deflection profiles, leading, ultimately, to predictions of abutment performance.



Pile fixity	Prototype pile bending moment range (MNm) (nominal surcharge (kPa) in brackets)		
Free–headed:	+2.8 (152)	+3.6 (132)	+1.8 (153)
Pile group:	−1.3/+0.8 (150)	−1.8/+1.3 (146)	0/−3.6 (151)
	a) Short pile ($\ell << \ell_c$)	b) Medium pile ($\ell < \simeq \ell_c$)	c) Long pile ($\ell > \ell_c$)

Figures 5a, 5b and 5c Pile displacement mechanisms for a surcharge ≃ 150 kPa

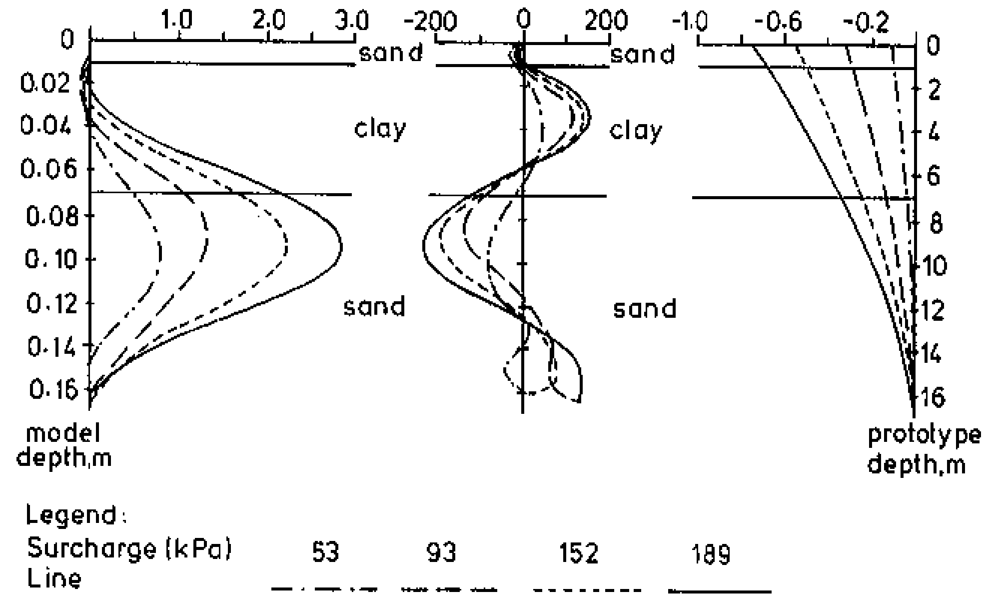


Figure 6 Selected data from a free–headed pile (at prototype scales)

4.1 Pile displacement mechanism

The displacement mechanism of the piles depended on the head fixity and the relative soil–pile stiffness. In Randolph's notation (1981), a pile is flexible if for radius, r, its length in the supporting stiffer substratum exceeds that required to resist lateral loading, $\ell_c = 2r\,(E_p/G_c)^{2/7}$, where the equivalent modulus of a solid pile in terms of the actual pile bending rigidity EI, is $E_p = EI/(\pi r^4/4)$. G_c is a characteristic shear modulus for the stiffer layer at a penetration, $\ell_c/2$. Most piles fall into this category due to the length required for axial capacity.

In the deep clay layer, the piles were relatively stiff, with length less than ℓ_c. The pile group translated through the soil, with soil resistance approaching the ultimate value, leading to maximum pile bending moments for the group as the soil sheared around the pile. The free–headed pile rotated about the tip causing ground level pile

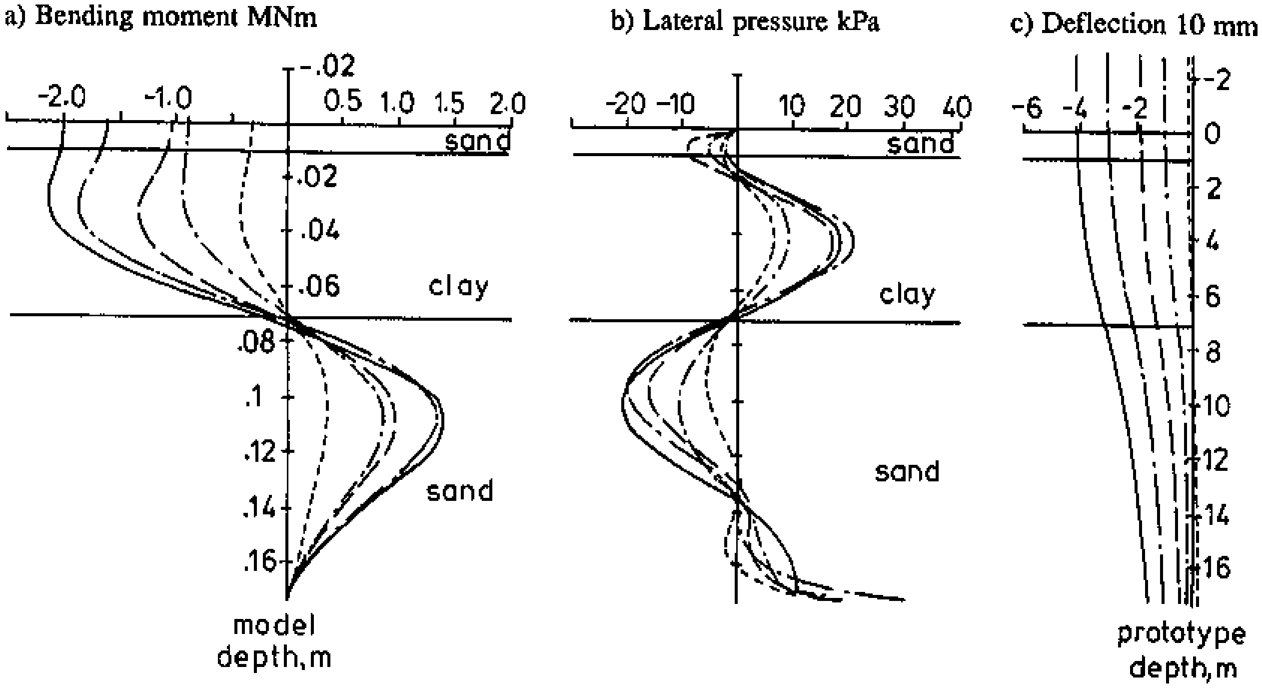


Figure 7 Selected data from a front pile of the group (at prototype scales)

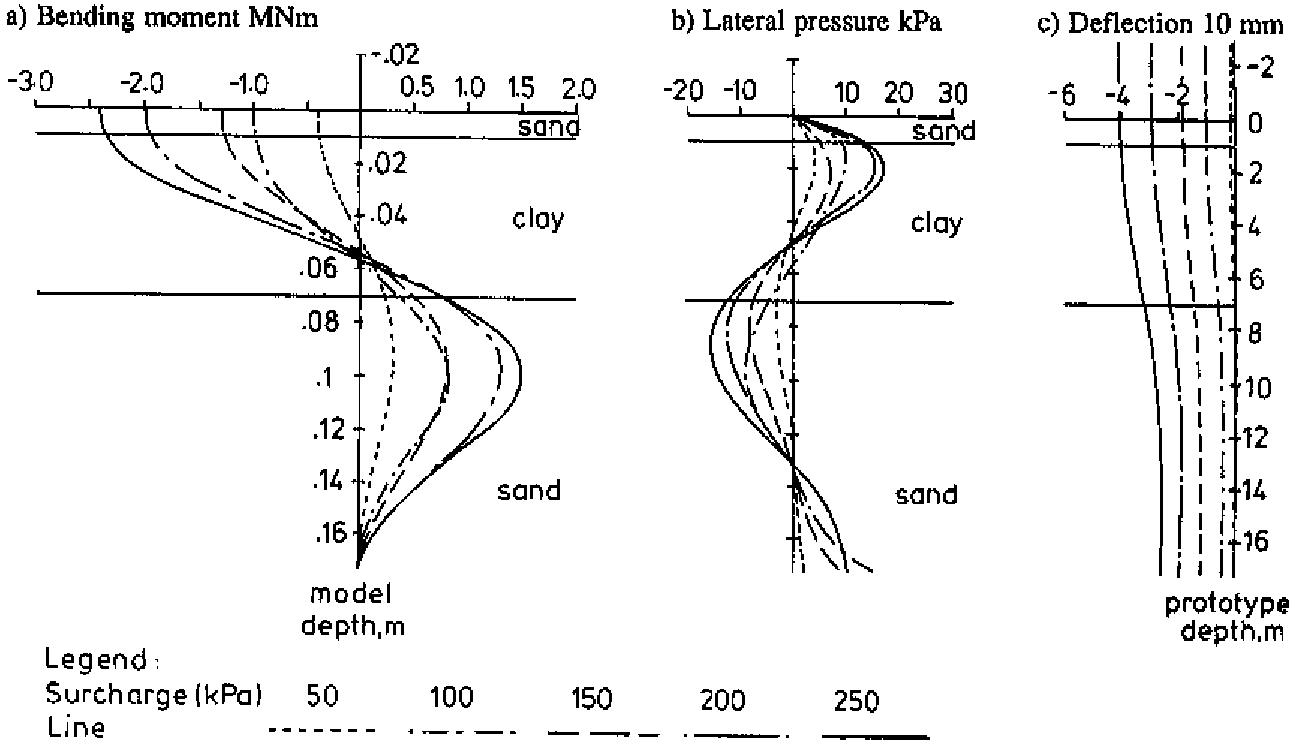


Figure 8 Selected data from a rear pile of the group (at prototype scales)
(Legend shown above is relevant to both Figures 7 & 8)

displacements that were nearly 3 times larger than those of the pile group, for a surcharge load of ≃ 150 kPa (Figure 5a).

If the pile was almost as long as ℓ_c (≃ 80 mm clay on ≃ 80 mm sand), a free–headed pile tended to rotate about its base, showing twice as much displacement at pile cap level as the group, which, due to assumed pile cap boundary conditions, appeared to have translated (Figure 5b).

If the pile was longer than ℓ_c (60 mm clay on 100 mm sand), the pile displacement and rotation was zero at ℓ_c, and the net lateral pile movement at

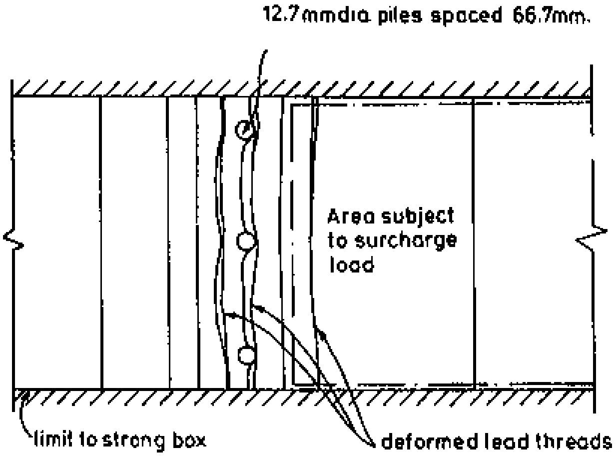


Figure 9 Internal soil–pile interaction for a row of free–headed piles

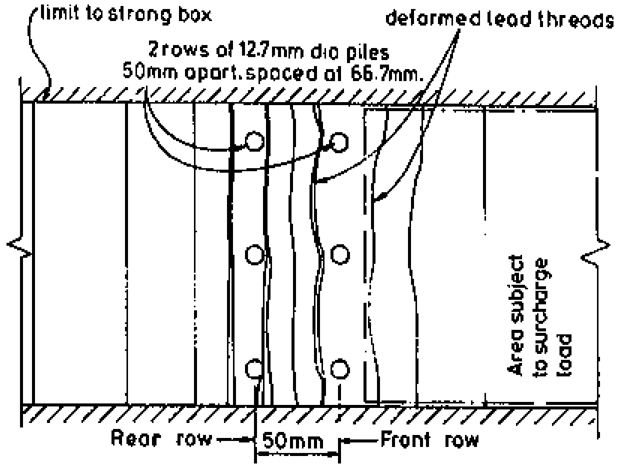


Figure 10 Internal soil–pile interaction for a pile group

ground level was considerably smaller for both pile head conditions (Figure 5c).

4.2 Pile bending moment and displacement profiles

The below ground pile displacements were derived from double integration of a polynomial fit to the 8 sets of bending moment data, choosing the order subjectively (generally 5) to give a small root mean square error. Initially it was hoped that the LVDTs would give head displacement and rotation, but the linearity, and hence accuracy, of the LVDTs over almost their full range of movement was not sufficient to ensure high quality rotation data, so the average displacement was calculated for the mid–height between the two transducers and a pinned pile tip was assumed for the free–headed piles. For the pile group cases, zero rotation of the pile cap was assumed, which implied some translation at the pile tip (Figure 5). An alternative assumption might include a small pile cap rotation and zero pile tip displacement.

Similarly, double differentiation of the bending moments allowed an estimation of the lateral pressures applied to the piles. However, this can only be taken as a guide, owing to the accumulation of error inherent in differentiation of the experimental moment profiles.

Figure 6 shows experimental data for a long free–headed pile from which it may be deduced that:

1. The pile was almost long enough to be flexible (some tip rotation was backfigured from this analysis).
2. Bending moments increase to a maximum at about 2 pile diameters into the stiffer stratum.
3. The lateral pressure applied by the clay throughout most of this softer layer, which was resisted by the stiffer stratum, could be approximated by a parabola.

Throughout these tests, it was observed that the effect of the group was to stiffen, markedly, the lateral response, reducing the horizontal displacements by a factor of 2 (Figure 5). The resulting bending moment, lateral pressure and displacement profiles for a pile group comprising a pair of long flexible piles ($\ell >> \ell_c$) are shown in Figures 7 (pile closest to surcharge has been designated front pile) and 8 (rear pile).

It must be noted that the surcharge did not simulate the lateral pressures acting on an abutment wall, which would in turn contribute an additional horizontal load at the pile cap; nor was the pile cap in contact with the ground surface. Consequently, the soil was able to squeeze vertically upwards between the rows of piles and so the lateral pressure on the rear pile was less than for the front, with the resultant lateral thrust at a shallower depth.

The maximum hogging and sagging bending moments were less than for the free–headed pile under similar loads, with slightly greater values for the rear pile. The bending moments increased only marginally for surcharge loads in excess of 200 kPa, when the soft clay had imposed the ultimate lateral pressure on the pile, implying that this condition could be used to describe a design upper bound for pile capacity.

4.3 Pile–soil interaction

Radiographs taken following the model tests on a single row and a group of piles show the pile–soil interaction at ultimate lateral pile capacity. Clear evidence of soil shear close to the single row of piles can be seen (Figure 9), with localised displacement confined to an area between 1–2 pile diameters around each pile. The soil between the piles appeared to be deforming uniformly outside this region.

The soil had clearly sheared past the front row of piles for the group (Figure 10), but considerably less displacement was noticeable in the vicinity of the rear row (due to the unrestrained top surface of the soil which was allowed to deform vertically, causing relief of lateral pressure on the rear row).

Partially for this reason, consolidation under the surcharge and around the piles merely caused a small 10–20 % reduction in bending moments, while the displacements increased minimally, by around 5–10 %.

5 CONCLUSIONS

In this parametric study of the performance of piles adjacent to surcharge loads, original data has been obtained. This has led to examination of the fundamental pile–soil interaction for a row of free–headed piles and of simple pile groups.

It is this ability to examine foundation displacement mechanisms at both working load, for serviceability analyses, and at the ultimate capacity which makes centrifuge model testing such a valuable experimental tool. Simple theories may be developed, and validated by data from both these models and appropriate finite element analyses (Springman, 1989), before being used by the geotechnical engineering profession for design purposes.

Idealisation of the field problem was necessary in order to concentrate on one aspect of the behaviour under investigation. In retrospect, some simplifications gave rise to other unanticipated complications, for example, the elevated pile cap.

But by focussing on the clay–pile interaction, it has been possible to propose a new method of analysis (Bolton, Springman & Sun, 1990) and incorporate it in a new approach to the design of piled full–height bridge abutments (Springman & Bolton, 1990), which is coded into an interactive computer program, SIMPLE, and a spreadsheet alternative, SLAP (Randolph & Springman, 1990).

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