BEHAVIOUR OF BORED PILES IN EXPANSIVE CLAY

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SUMMARY

Four reinforced concrete piles, of the same length and shaft diameter, but having different shaft-soil interface conditions, were constructed in late 1993. The test piles were located at an extremely reactive clay site where the potential surface movement exceeds 120 millimetres. Observations were made, with time and changing moisture regimes, of the soil movements, soil properties, pile movements and pile strains.

Computer modelling has commenced in order to simulate and better understand the pile behaviour. The types of numerical analysis being investigated include an elastic thermomechanical model (that is, no pile-soil slip) and a simplified boundary element model which simulates the soil as an elasto-plastic continuum (that is, pile-soil slip is possible). The paper summarises the experimental observations to date and provides preliminary results of the theoretical modelling.

INTRODUCTION

Reactive clay soils shrink upon drying and swell upon wetting. Trees, landscaping, and plumbing leaks, in addition to seasonal changes, all affect the moisture condition of the soil. The volumetric changes of the soil can lead to differential foundation movements of lightly loaded structures. Unsightly cracking, jamming doors and windows and structural damage can occur in buildings that have been constructed upon the moving surface. Expansive clays are found in many parts of the world, particularly in semi-arid environments, including Australia, Egypt, Israel, South Africa and the United States. Damage to housing due to differential soil movements is a costly concern in these parts of the world. Foundation engineers are faced with the challenge of designing economical and effective footing systems that can withstand the extreme soil conditions.

Piled foundations are common in areas where the characteristic site soil movement is calculated to be in excess of 100 millimetres. The floor slab is suspended above the soil by piles which are founded at a stable depth as recommended by AS2870 [4]. Due to the nature of expansive clays, bored reinforced concrete piles, sometimes with underreaming, are generally the most economical type of piling. However, the piles must still resist the actions of the surrounding soil and are susceptible to uplift and tensile forces due to the swelling clays.

One type of piled footing being developed is the Tri-Ped footing. The design is based on a reinforced concrete floor slab suspended above the moving soils by three piles. The Tri-Ped has been developed over the past four years by the University of South Australia in collaboration with the South Australian Housing Trust. The SA Housing Trust has used the footing for houses in its normal building program at highly reactive sites. A prototype footing was successfully constructed and test loaded in November 1992 [6]. The prototype house construction was completed in February 1993 after which it was tenanted. Since then twenty eight houses have been built using the Tri-Ped concept. Other piled foundations that have been used include pile and beam with timber floors, and pile-and-beam with concrete floors using lost cardboard formwork or steel tray decking.

As part of the research and development program undertaken at the University of South Australia, methods of isolating the piles from the soil have been trialed. The investigation into the effectiveness of different pile-soil interfaces began in late 1993 with the construction of four piles at a highly reactive clay site.

THE EXPERIMENT

Piles

A known reactive site in the north-eastern suburbs of Adelaide, South Australia was chosen to conduct an investigation into pile behaviour in heaving clays with varying soil-pile interface conditions. The site has a characteristic site soil movement of 120mm, and therefore, is classified as an extremely reactive site [4]. Four piles were constructed during November 1993. All of the piles were bored, cast in-situ reinforced concrete, five metres long with a 0.5 metre diameter shaft. Each of the piles consists of different combinations of soil-pile conditions and anchorage (that is, underream) conditions as shown in Figure 1.

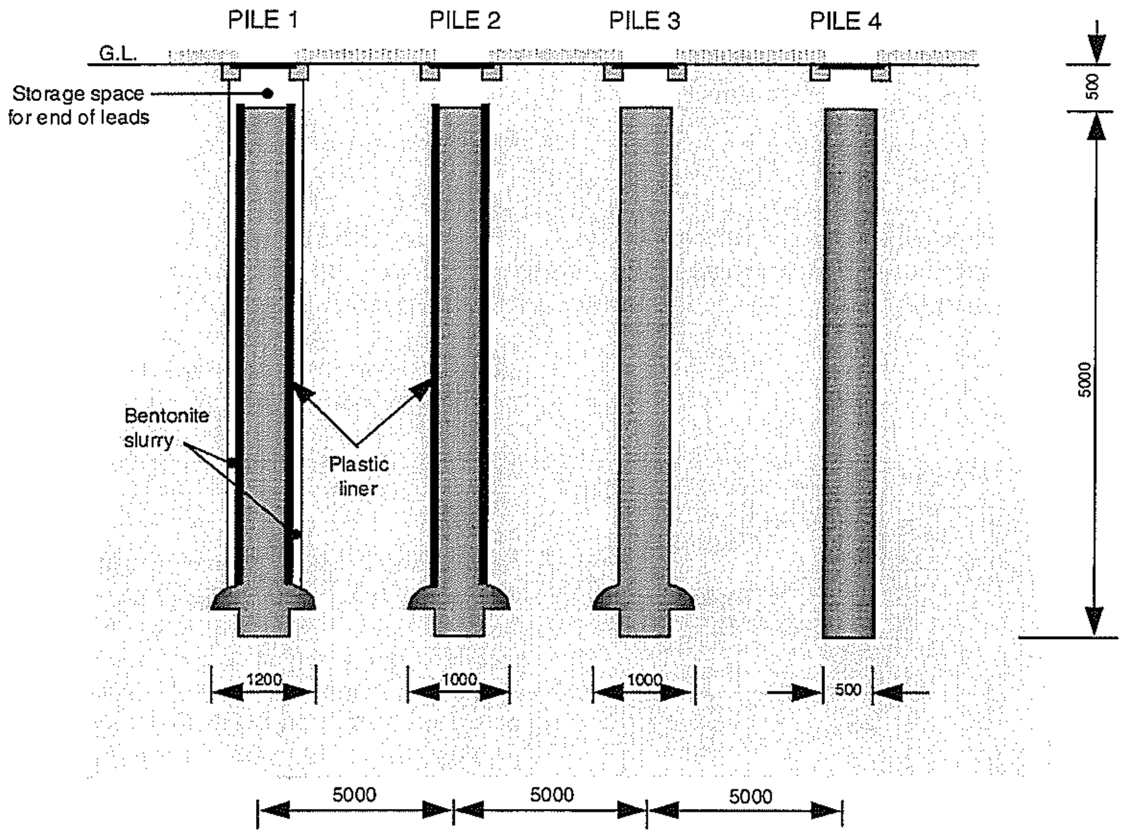

Figure 1 - Elevation of piles

Pile 1 was designed to be independent from the surrounding soil movements. The shaft of the pile was formed inside a 500 mm diameter smooth plastic liner and a 50 mm annulus of bentonite slurry was placed between the liner and the soil to the depth of the underream. The bentonite slurry consisted of pure bentonite powder mixed with water at a ratio of 1:8 by mass. In the hydrated state, bentonite exhibits negligible shear strength, but also prevents water from migrating to the base of the pile. The liner is a spirally wound, ribbed plastic with the ribs on the inside. This configuration is aimed to eliminate all shaft friction. The borehole of 600 mm diameter was underreamed to form a 1200 mm diameter base. This type of pile was used for the prototype Tri-Ped footing.

Pile 2 was constructed with the plastic liner directly between the soil and the pile to the depth of the underream with the aim to reduce shaft friction. The borehole diameter was 500 mm and the outside diameter of the liner was 490 mm which resulted in a neat fit against the side of the bored hole.

Pile 3 was constructed as a standard underreamed pile with no special pile-soil interface. This type of pile was used for all Tri-Ped footings except the prototype.

Pile 4 was constructed as a standard straight pile with no special pile-soil interface and acts as a control pile.

The piles are reinforced with eight Y20 reinforcing bars, W6 ties at 150 mm centres and 60 mm cover. The steel area of 1.3% of concrete area is significantly greater than the minimum steel of 0.5% of gross concrete area recommended by the piling code [5]. Since the pile with the bentonite surround (pile 1) has no lateral support from the soil, the pile is required to be designed as a column and therefore minimum steel required is 1% of gross pile area. The concrete was specified as compressive strength (f'_c) of 20 MPa, 20 mm aggregate and 80 mm slump. The concrete was mechanically vibrated to the full depth of the pile. Laboratory testing showed a compressive strength of 18 MPa at 28 days.

The piles were instrumented with embedded electrical strain gauges in order to monitor the stresses developed in the piles due to the soil movements. Each of the 120mm gauges was precast into a 'dog bone' shaped concrete block to ensure a strong clean bond with the gauge. The blocks were then fixed at the appropriate locations to the prefabricated steel reinforcement cages ready for construction of the piles. Each pile has 12 gauges, that is, two at each of six depths along the length. One of the piles was also instrumented with temperature gauges in order to correct the strain gauge readings for errors due to temperature changes. A dummy gauge next to each of the piles, wrapped in compressible rubber foam, was also buried at the depth of the shallowest gauge in the pile for control purposes.

Soil

Soil sampling was taken at 0.5 metre intervals to a depth of 5.0 metres, using hydraulically pushed, thin-walled sample tubes. Laboratory tests were undertaken to determine suction, moisture content, soil modulus and undrained shear strength profiles. Atterberg limits and loaded shrinkage indices were also determined.

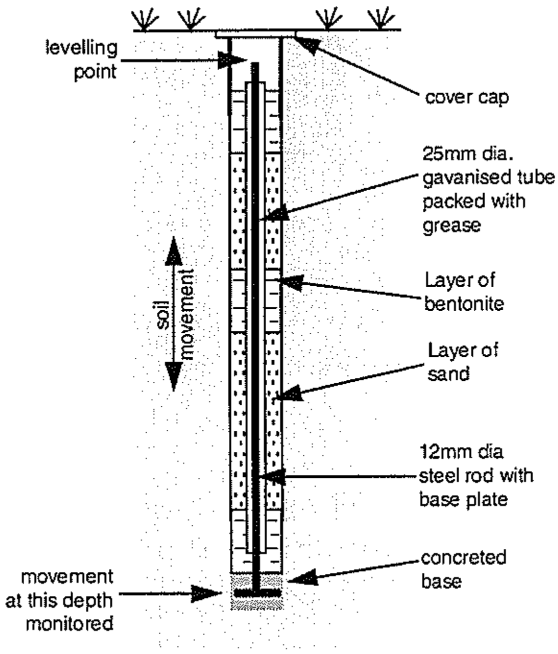

Figure 2 - Soil probe details

Three nuclear meter probe holes were also constructed with the aim of giving instantaneous values of moisture conditions without the need of sampling. The results are not reported within this paper.

Movements have been monitored at the surface and at depths of 0.5, 1.0, 1.5, 2.0, 3.0 and 4.0 metres via level surveys. The movements at depth were measured by two sets of deep probes across the site. The probes consist of a 12 mm diameter steel rod placed inside a 20 mm galvanised pipe filled with packing grease. The base of the rod is welded to a 75 mm base plate which was concreted into the soil. The 100 mm diameter hole was backfilled with layers of sand and bentonite powder which acts as a water sealant and reduces skin friction adjacent to the pipe surrounding the rod as shown in Figure 2.

A benchmark was constructed using the same methods and materials. It is founded at a depth of six metres, and therefore is founded in the Tertiary sands and is anchored below the expansive soil movements.

RESULTS

Soil

The soil profile at the site is typical of the north-eastern suburbs of Adelaide where reactive soils have damaged a significant proportion of housing. Table 1 lists the general descriptions and properties of each of the three soil horizons overlaying the Tertiary sands. The clays and silts were deposited by wind and water erosion of the nearby Adelaide Hills during the early to middle Quaternary period.

The layer of fill was placed in 1993 as part of the development of a new subdivision and consists of the shallow soil cut to form wet lands approximately 500 metres from the pile site. The land was previously used for agricultural research.

Table 1 - Typical borehole

MATERIAL	SOIL DESCRIPTION	MATERIAL PROPERTIES
0.3m	Fill - mixture of Black Clay and Clay	-
1.0m	Black Clay: dark grey to black, hard structure (dry) sticky, very firm, highly plastic (wet)	I_{ls} = 6.0% LL = 95%, PL = 35%
4.5m	Clay: light grey to olive brown heavy clay, stiff and highly plastic, with yellow to red mottles (Keswick Clay)	I_{ls} = 4.5 - 6.0% LL = 100%, PL = 25% LEGEND I_{ls} - loaded shrinkage index PL - plastic limit LL - liquid limit
6.0m	Clayey Silt/Silty Clay: multi-coloured (highly mottled) yellowish brown, brownish yellow, dark red and grey, very stiff (Hindmarsh Clay)	I_{ls} = 0.5 - 2.0% LL = 65%, PL = 20%
EOH	Silty Clayey Sands: quartz sands, weakly cemented, some gravel (Tertiary Sands)	I_{ls} = 0.0%

Movements

Initial soil moisture conditions at the site showed relatively high (dry) suction values near the surface, grading down to medium suction values at depths greater than 2.5 metres. For the first year, the winter (wet) season was drier than usual and therefore changes in suctions only occurred in the top one metre of soil; very minor soil movements were observed, as is shown in Figure 4.

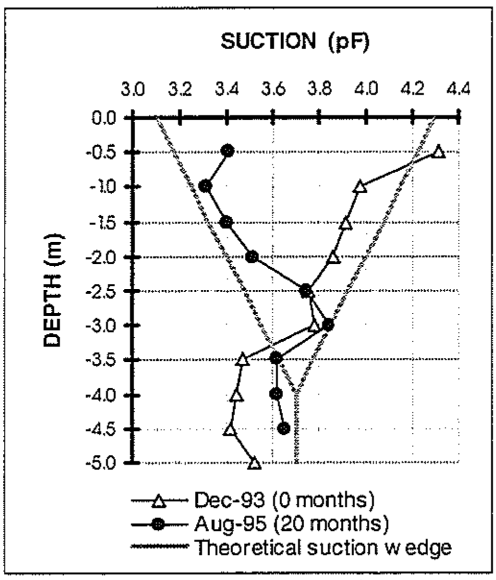

Figure 3 - Suction profiles

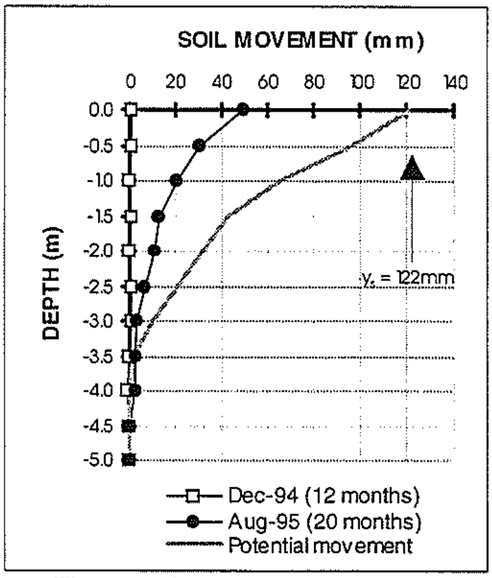

Figure 4 - Soil movement profiles

In order to achieve significant surface movements of the soil over a short period of time, a soakage trench was constructed adjacent to the piles. The trench is located 2 metres from the piles and is 0.3 metres wide and 1.5 metres deep. Below the trench, a series of holes were bored to a depth of 3 metres for deeper water penetration. The trench was first artificially filled with water in January, 1995 and significant soil movements were recorded almost immediately. The soil continued to heave for approximately four months. The top 2.5 to 3.0 metres of soil had considerably decreased in suction over this period and significant soil movements had been recorded to these depths. Figures 3 and 4 show the suction changes and corresponding soil movements from near the time of construction, December 1993 until August 1995.

Since the top 2 to 3 metres of soil were now low in suction, a nearly impermeable barrier was formed against the seasonal rains that fell in the winter months of June to August. Very little watering of the trench was undertaken between April and August 1995. Almost no movements were observed during these months even though this is the expected time for the soil to take in water and heave. Due to the lack of surface movements the trench is now being filled with water regularly in order to obtain further (and deeper) soil movements.

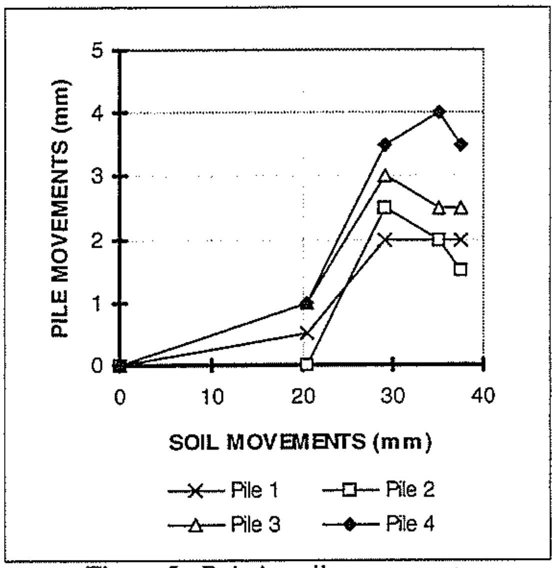

Figure 5 - Relative pile movements

Many more months of natural and artificial wetting will be required to achieve movements of above 100mm at the surface, which is the potential movement in an urban environment. Also, since the tops of the piles are 0.5m below the surface, the soil movements (at the top of the pile) are significantly less, by approximately 25 mm of heave, than at the surface.

The pile movements are taken as an average of two readings either side of the pile and have been taken monthly. Since the soil movements at the top of the pile have only reached 35 mm, theoretical predictions only expect about 4 mm pile movement in the worst case of the straight pile (pile 4). The measurements are taken using a dumpy level and require two change points from the location of the benchmark. Therefore, an accuracy of approximately ±2 mm was achieved. Since the pile movements were all less than 5 mm, a 2 mm error is significant. However, the pile movement results are in agreement with expected values. Figure 5 shows that the straight pile with no shaft lining (pile 4) has moved the greatest and the underreamed piles with the plastic linings (piles 1 and 2) have moved the least.

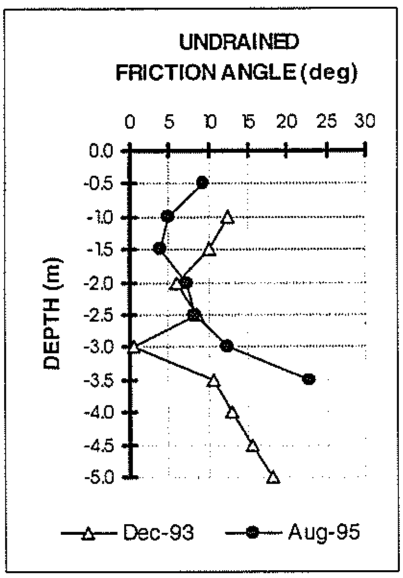

Figure 6 - Friction angle profiles

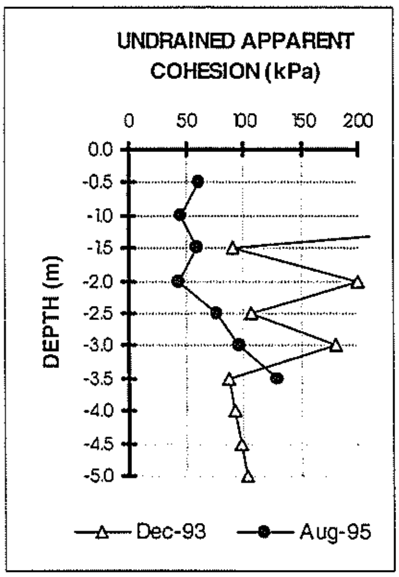

Figure 7 - Cohesion profiles

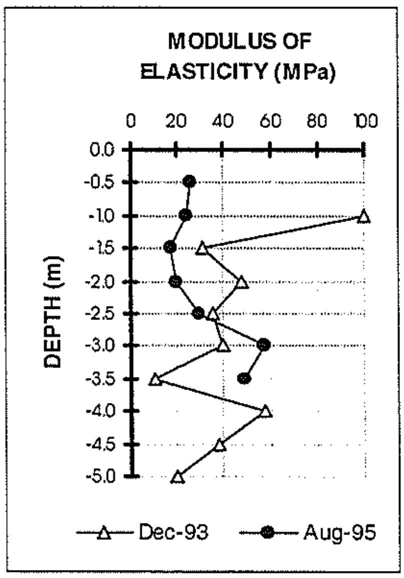

Figure 8 - Soil modulus profiles

Undrained shear strength parameters and Young's Modulus for the change in moisture conditions between December 1993 and August 1995 are shown in Figures 6, 7 and 8. The soil properties are more consistent when the suction values are lower (that is, August 1995). Triaxial testing was conducted on a single 38 mm diameter sample for each depth in a 3-stage test. The ratio of modulus of elasticity (E_u) to apparent undrained cohesion (c_u) equates to 430 for the August 1995 results. This correlates well with the suggested value for E_u/c_u of 400 for a 38mm diameter sample of stiff clay [2].

Strains

The strain gauge readings have been taken regularly, however the values obtained are spurious with readings of over 4000 microstrains ($\mu\varepsilon$) on one side of a pile at a specific depth and of less than 100 $\mu\varepsilon$ on the other side at the same depth. These high readings were also obtained before any significant soil heaving. Laboratory tests of concrete samples indicate that approximately 200 $\mu\varepsilon$ of tension is required to initiate cracking. No compressive values of strains have been recorded. The dummy gauges (surrounded by compressible rubber foam) have also all given readings of over 4000 $\mu\varepsilon$. The reasons for these inconsistent readings are still being investigated. Corrosion is one possibility, however, the gauges, leads and connections used were designed to be embedded in concrete. Also, a dummy gauge that has been exhumed shows no sign of corrosion.

Modelling

The research aims to interpret the experimental observations and provide comparisons with the theoretical simulations. Elastic modelling, that is, no pile-soil slip, was executed using the finite element package STRAND6 [1]. This involved a simple two dimensional, axi-symmetric, heat model. The boundary element analysis program PIES - Piles In Expansive Soils [3], was used to simulate pile-soil slip and non-linear soil properties. The output shows forces induced in the pile by the soil movements and the resultant pile movements. The parameters used in these models are listed in Appendix A. The results of these models are shown in Figure 9 for the straight pile case (pile 4). The fine line indicates the predicted pile movement for each model corresponding the soil movements at August 1995. The non-linear value of 3.1 mm corresponds closely with the experimental results. Further parametric studies are to be undertaken in order to assess the sensitivity of each of the parameters.

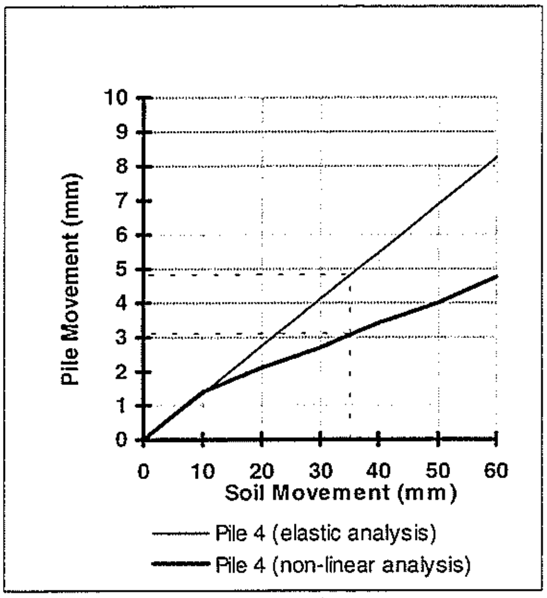

Figure 9 - Theoretical pile movements

It is envisaged that thermo-mechanical, axi-symmetric, finite element modelling will also be undertaken to further understand the relationships between soil and pile behaviours.

CONCLUSION

More field and modelling data is required in order to reach strong conclusions. Another six months of monitoring should result in further soil heave and therefore pile movements. However, as the soil suction values decrease, the soil absorbs moisture at a decreasing rate, therefore requiring greater time for measurable results. Increased movements will enable further data to compare with the computer modelling.

The finite element and the boundary element computer simulations will be used to demonstrate and facilitate possible design methods with higher precision than the currently available design charts.

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APPENDIX A

Table A1 - Parameters for elastic finite element model, STRAND6

Pile properties			Soil properties		
Young's Modulus	(E_{pier})	- 24 000 MPa	Modulus	(E_{soil})	- 20 MPa
Poisson's Ratio	(μ_{pier})	- 0.25	Poisson's Ratio	(μ_{soil})	- 0.30
Density	(ρ_{pier})	- 2500 kg/m^3	Density	(ρ_{soil})	- 2500 kg/m^3
Coeff of Expansion	(α_{pier})	- 0.00	Coeff of Expansion	(α_{soil})	- 0.01
Length	(L_{pier})	- 5 m	Soil movement:		
Diameter	(d_{pier})	- 0.5 m	triangular distribution (from 0 at 2.5 m		
No. of elements		- 10	deep to swelling movement at surface)		

Table A2 - Parameters for non-liner boundary element model, PIES

Pile properties			Soil properties		
Young's Modulus	(E_{pier})	- 25 000 MPa	Modulus	($E_{soil\ at\ tip}$)	- 20 MPa
Length	(L_{pier})	- 5 m	Modulus	($E_{soil\ below\ tip}$)	- 100 MPa
Diameter	(d_{pier})	- 0.5 m	Poisson's Ratio	(μ_{soil})	- 0.30
No. of elements		- 10	Shaft resistance:		
			σ_{peak} = 60 kPa,	$\sigma_{residual}$ = 45 kPa,	
			displacement$_{peak\text{-}to\ residual}$ = 10 mm		
			Soil movement:		
			triangular distribution (from 0 at 2.5 m		
			deep to swelling movement at surface)		
			Hyperbolic soil response:		
			shaft (R_{fs}) = 0.5, base (R_{fb}) = 0.9		