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Performance of the hard–soft piling system at A12 Hackney to M11 Link Road

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CONTENTS

Executive Summary

At the A12 Hackney to M11 Link Road the construction procedure, which was adopted as being the most cost effective, used a hard-soft piling system. Initially relatively weak cement-bentonite piles were installed from ground level. Once these had cured, the main reinforced concrete piles were installed by boring through their softer neighbours. The secant bored pile wall formed in this way typically had a retained height of between 4.5m and 7.5m and used a variety of hard and soft pile diameters.

However concerns about the permeability of the cement-bentonite ‘soft’ piles and their long term durability existed and for this reason a sprayed concrete facing was employed over the soft piles. Further cost savings, in addition to those resulting from the use of the hard-soft system, would have accrued if the soft piles could have been relied on to provide an adequate water seal as the sprayed concrete facing could then have been omitted.

A unique opportunity therefore existed at this scheme to make *in situ* measurements and install instrumentation to monitor the performance of the hard-soft piling system so that more information is available to assist in the design of future similar schemes.

Field instrumentation was installed at two locations along the length of the wall, at chainages 3540 and 3370. Piezometers were installed in the retained ground, the soft pile itself, and the ground beneath the new carriageway so that the flow of ground water could be clearly established in the vicinity of the wall. These measurements were supported by *in situ* tests to establish the permeability of the retained ground and the soft piles. In addition the permeability of the soft pile material was separately evaluated by laboratory testing.

An assessment of the pattern of water seepage was also carried out using a finite element program for analysing two-dimensional ground water flow. The findings from the analyses were compared with the site measurements and design recommendations made for future construction of the same type.

1 Introduction

At the A12 Hackney to M11 Link Road the construction procedure, which was adopted as being the most cost effective, used a hard-soft piling system. Initially relatively weak cement-bentonite piles were installed from ground level. Once these had cured, the main reinforced concrete piles were installed by boring through their softer neighbours. The secant bored pile wall formed in this way typically had a retained height of between 4.5m and 7.5m and used a variety of hard and soft pile diameters.

However concerns about the permeability of the cement-bentonite soft piles and their long term durability existed and for this reason a sprayed concrete facing was employed over the soft piles. Further cost savings, in addition to those resulting from the use of the hard-soft system, would have accrued if the soft piles could have been relied on to provide an adequate water seal as the sprayed concrete facing could then have been omitted.

A unique opportunity therefore existed at this scheme to make *in situ* measurements and install instrumentation to monitor the performance of the hard-soft piling system so that more information was available to assist in the design of future similar schemes. For this purpose piezometers were installed in the retained ground, the soft pile itself, and the ground beneath the new carriageway so that the flow of ground water could be clearly established in the vicinity of the wall. These measurements were supported by *in situ* tests to establish the permeability of the retained ground and the soft piles. In addition the permeability of the soft pile material was separately evaluated by laboratory testing.

An assessment of the pattern of water seepage in the vicinity of the retaining wall was also carried out using a finite element program for analysing two-dimensional ground water flow. The findings from the analyses were compared with the site measurements and design recommendations made.

2 Site location

The central section of the A12 Hackney to M11 Link Road was constructed under the second of the four main scheme contracts (Contract 2). The 2.4km long section included construction of the dual three lane carriageway adjacent to the western side of the London Underground Central Line railway. For the purpose of the TRL study on the performance of hard-soft pile system, two areas were selected as suitable for instrumentation. These areas were centred on the south wall at contract chainages 3540 and 3370 as shown in Figure 1.

3 Soil properties

A summary of the soil profiles established from the TRL boreholes in the instrumented areas at chainages 3540 and 3370 is shown in Figure 2. At chainage 3540, a brown clayey sand and gravel layer (Taplow gravel) which was up to 5m thick overlaid the London Clay. The London Clay comprised a thin layer (< 0.5m) of weathered but firm brown clay with stiff grey unweathered clay beneath this. Inspection of the exposed face of the bored piles confirmed this soil profile, as gravel could be seen adhering to the concrete to a level of 16.6mAOD.

Figure 2b shows the soil profile at chainage 3370 and at this location the depth to the grey unweathered London Clay was between 6.5m to 7m. The clay was overlaid with clayey sand and gravel (Taplow gravel), although at 5.7m depth there was a 200mm thick band of soft brown clay.

4 Details of construction

At chainage 3540 the secant piled retaining wall was constructed by installing ‘soft’ cement-bentonite piles of 900mm diameter at a spacing of 2000mm between centres and then installing ‘hard’ reinforced concrete piles of 1800mm diameter between them. The depth of the hard piles was 20.7m whilst the soft piles only extended to 12.7m; there was considered to be no purpose in extending the soft piles to a greater depth as they were already founded in impermeable London Clay and had achieved water cut-off. Similarly at chainage 3370, 900mm diameter soft piles were installed at 1700mm centres with 1500mm diameter hard piles between them. Depths of the hard and

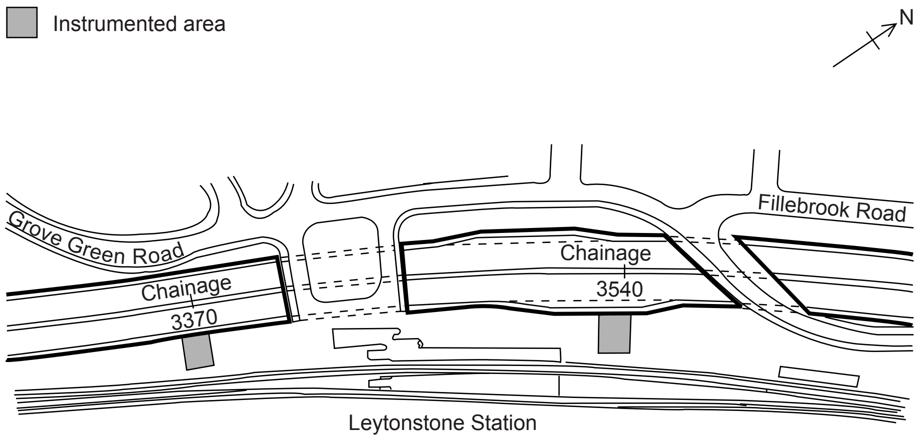


Figure 1 Location of TRL instrumented areas

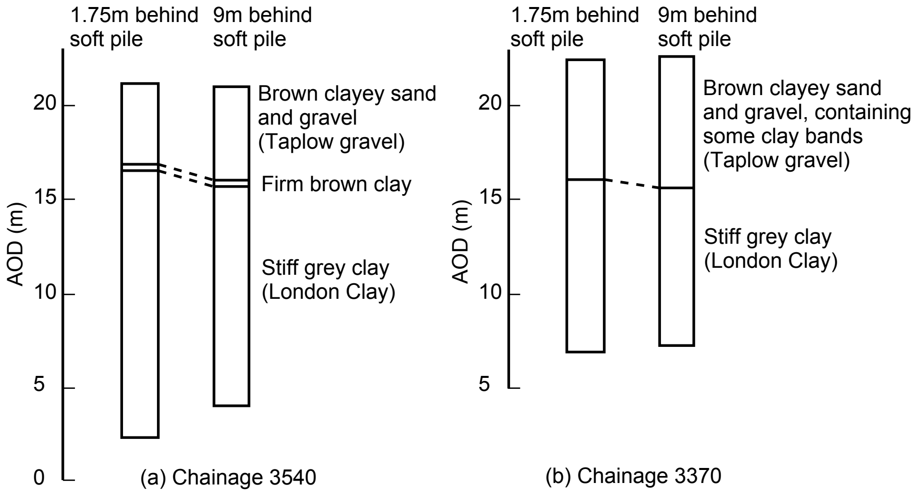


Figure 2 Soil profile established from TRL boreholes

soft piles at this latter location were 15.1m and 10.6m respectively. At both locations the centres of the soft piles were offset by 0.3m towards the retained ground from the line between adjacent hard pile centres.

At chainage 3540 the retained height of the wall was 9.8m, whilst at chainage 3370 the retained height was 6.5m. After excavation, sprayed concrete was applied to the exposed face of the soft piles at both locations to restore the watertightness to that of a hard-hard pile wall. A photograph of the completed wall at chainage 3540 is shown in Plate 1.

5 Field instrumentation

Field instrumentation was installed at two locations along the length of the wall, at chainages 3540 and 3370. At both locations, the instrumentation layouts were similar and these are shown in Figures 3 and 4 respectively.

5.1 Piezometers in the ground

The distribution of pore water pressure in the retained ground at each location was measured using an array of piezometers. The depths and distances from the wall of the 26 piezometers installed in the ground at chainage 3540 are shown in Table 1 and Figure 3. Generally piezometers were of the pneumatic type although two hydraulic piezometers were installed in one of the boreholes to enable *in situ* permeability tests to be undertaken. It was not possible to install any piezometers in front of the wall at this chainage as the reinforced concrete slab at carriageway level had already been cast by the time of instrument installation.

Table 2 and Figure 4 show the depths and distances from the wall of the 23 piezometers installed in the retained ground at chainage 3370. At this second instrumented section, three piezometers were also installed

Plate 1 The completed wall at chainage 3540

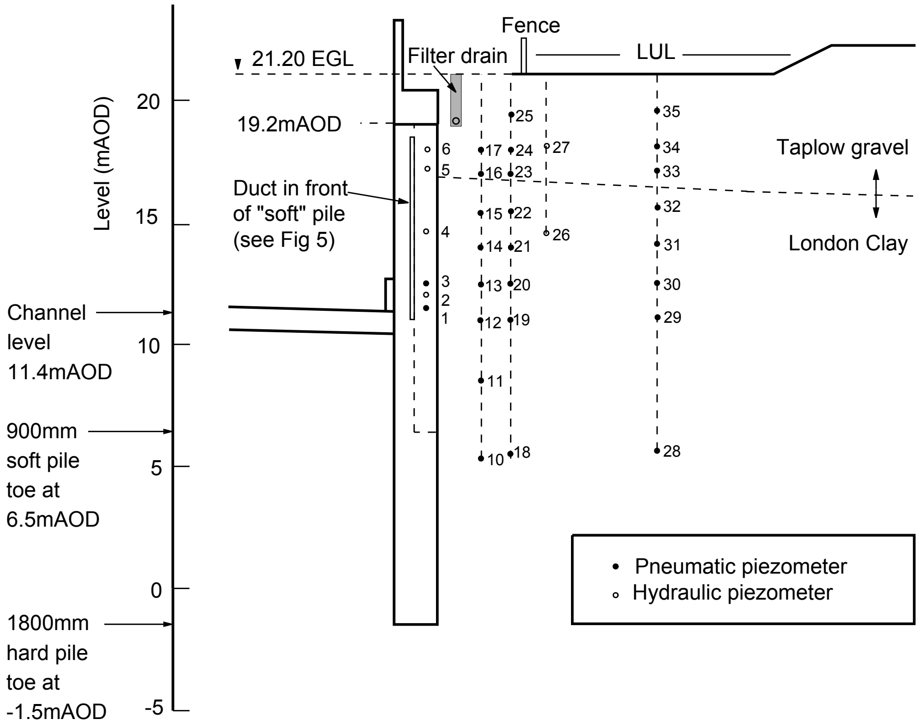


Figure 3 Instrumentation of south wall at chainage 3540

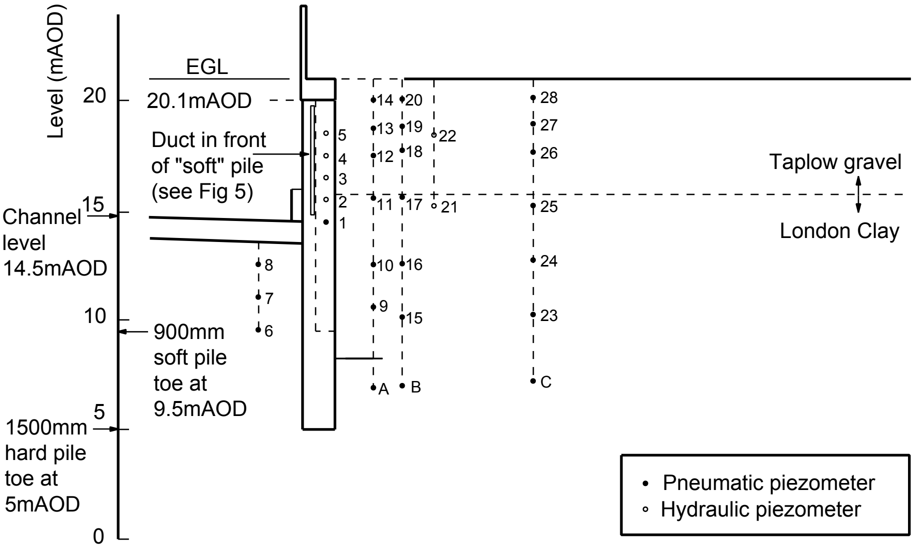


Figure 4 Instrumentation of south wall at chainage 3370

at depths of 2m, 3.5m and 5m below the reinforced concrete carriageway slab in a borehole at 2m in front of the wall.

Boreholes for the piezometers in the retained ground were drilled using a cable tool percussion drilling rig; whilst a portable rotary auger rig was used on the excavated side of the wall. In all cases the piezometers were installed in nominal 100mm long sand cells and the boreholes were backfilled with bentonite pellets to ensure an effective water seal. Piezometer tubes were terminated at instrument cabinets mounted against the wall parapet.

5.2 Piezometers in the soft piles

A number of piezometers were installed at different depths in the soft pile at each of the two locations. Each piezometer was positioned at the centre of the soft pile, ie. 450mm from its edge. The installation was carried out from the front face of the pile and comprised drilling a 50mm diameter and 300mm long hole, sloping downwards at an angle of about 10° to the horizontal, and then extending this hole to the required distance by removing a 38mm diameter core sample. A small quantity of a liquid cement-bentonite-sand mix was then poured into the hole

Table 1 **Locations of piezometers in the retained ground at chainage 3540**

1.75m behind soft pile		*3m behind soft pile*		*9m behind soft pile*	
Pneumatic piezometer no.	*Level (mAOD)*	*Pneumatic piezometer no.*	*Level (mAOD)*	*Pneumatic piezometer no.*	*Level (mAOD)*
10	5.28	18	5.56	28	5.71
11	8.48	19	11.06	29	11.21
12	11.00	20	12.59	30	12.60
13	12.48	21	14.06	31	14.21
14	14.00	22	15.59	32	15.70
15	15.48	23	17.06	33	17.21
16	17.00	24	18.09	34	18.20
17	17.98	25	19.56	35	19.71

Hydraulic piezometers 26 and 27 were installed at 14.64 and 18.24mAOD and at 4m behind the soft piles for the purpose of permeability testing.

Table 2 **Locations of piezometers in the retained ground at chainage 3370**

1.75m behind soft pile		*3m behind soft pile*		*9m behind soft pile*	
Pneumatic piezometer no.	*Level (mAOD)*	*Pneumatic piezometer no.*	*Level (mAOD)*	*Pneumatic piezometer no.*	*Level (mAOD)*
A	6.84	B	6.99	C	7.17
9	10.54	15	10.09	23	10.17
10	12.54	16	12.59	24	12.67
11	15.54	17	15.59	25	15.17
12	17.54	18	17.72	26	17.64
13	18.79	19	18.84	27	18.92
14	20.04	20	20.12	28	20.14

Hydraulic piezometers 21 and 22 were installed at 15.21 and 18.41mAOD and at 4m behind the soft piles for the purpose of permeability testing.

and the close fitting piezometer pushed in. A dryish cement-bentonite-sand mix was then compacted into the hole followed by a waterproof cement mortar to achieve the final sealing.

A combination of pneumatic and hydraulic piezometers were used and the location of each piezometer is given in Table 3.

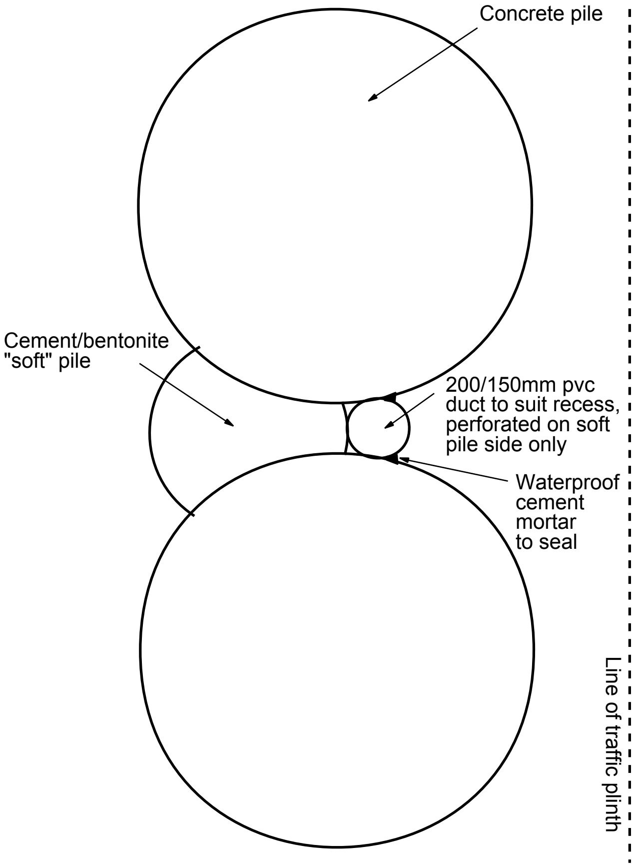
Table 3 Details of piezometers installed in the soft piles

Chainage ***3540***			***Chainage 3370***		
Piezo-meter type and no.	***Depth below top of wall capping (m)***	***AOD (m)***	***Piezo-meter type and no.***	***Depth below top of wall capping (m)***	***AOD (m)***
Hydraulic,6	2.15	18.05	Hydraulic,5	2.60	18.50
Hydraulic,5	3.00	17.20	Hydraulic,4	3.60	17.50
Hydraulic,4	5.55	14.65	Hydraulic,3	4.60	16.50
Pneumatic,3	7.68	12.52	Hydraulic,2	5.60	15.50
Hydraulic,2	8.15	12.05	Pneumatic,1	6.60	14.50
Pneumatic,1	8.70	11.50			

5.3 Water collection system

In order to determine whether significant quantities of water were passing through the soft piles from the retained ground, a collection system was installed on the excavated side of the retaining wall.

At each location, a vertical plastic pipe was installed which fitted closely between adjacent hard piles and so spanned the exposure of the intervening soft pile (Figure 5). This pipe extended up the face of the wall to near the underside of the capping beam. The joints between the pipe and the hard piles were made watertight using a waterproof cement mortar. Holes had been drilled in the back of the plastic pipe to allow any water permeating through the soft pile to enter and collect in the pipe. The head of water developed was monitored using a pneumatic piezometer installed at the bottom of the pipe and a bottom drain was also incorporated so that the volume of water collected could be measured.



Note: Shotcrete applied over TRL duct as elsewhere

Figure 5 Water collection system in front of soft pile

6 Properties of the soft piles

The mix design for the soft piles comprised dry batch weights of 150kg/m^3, 35kg/m^3 and 1230kg/m^3 of Portland cement, bentonite, and sand (from Angerstein Wharf) respectively. During the mixing, 460kg/m^3 of free water was added. The site requirements were that the soft piles should have a minimum shear strength of 150kPa at 28 days and a permeability not exceeding 5×10^{-8}m/s.

During installation of the piezometers in the soft piles (Section 5.2), 38mm diameter core samples were recovered for laboratory testing. Tests were carried out on these samples to provide further information on the permeability, undrained shear strength and particle size distribution of the cement-bentonite-sand mix used for the soft piles.

6.1 Permeability measurements

6.1.1 Laboratory permeability tests

The determinations of permeability of the samples from the soft piles were carried out in the laboratory using a triaxial cell. The test procedure followed that given in Clause 6 of BS1377: Part 6 (1990) although calibration for head loss was not necessary as pore water pressures were measured by transducers at both the top and bottom of the specimen. It must be noted that although 100mm diameter specimens are preferred for this test only 38mm diameter specimens were available to TRL in this case. BS1377 also states that the method is suitable for soils of low and intermediate permeability.

Initial tests were conducted using very low effective confining pressures in an attempt to reproduce *in situ* conditions. However it was found that this pressure was insufficient to prevent seepage between sample and membrane (Card, 1981) causing a threefold increase in permeability. Figure 6 shows that when effective pressures were raised to greater than 20kPa, the coefficient of permeability tended to a more constant value. For this reason all tests were conducted with cell pressures of above 500kPa, permitting use of back pressures in excess of 400kPa and thus ensuring that air dissolved within the specimen during saturation remained in solution. The results from permeability tests on samples recovered from different depths in the soft piles at the two locations are summarised in Table 4.

Table 4 Laboratory permeability results on soft pile material

Chainage 3540		*Chainage 3370*	
Depth (m)	*Permeability (m/s)*	*Depth (m)*	*Permeability (m/s)*
3.00	0.9×10^{-7}	2.60	n/a
5.55	1.4×10^{-7}	3.60	n/a
7.68	1.8×10^{-7}	4.60	1.7×10^{-7}
8.15	1.7×10^{-7}	5.60	1.3×10^{-7}
8.70	1.6×10^{-7}	6.60	0.6×10^{-7}
Average	1.5×10^{-7}	Average	1.2×10^{-7}

The permeability of the soft pile material measured in this way was very similar at the two locations and fell within the low permeability classification of 10^{-5} to 10^{-7}m/s suggested by Terzaghi and Peck (1948). Fissured and weathered clays or very fine/silty sands would also be expected to fall within this same permeability category.

It must be noted that *in situ* permeability tests, together with laboratory tests on larger diameter samples, would be expected to indicate that the soft piles are less permeable than the tests on 38mm diameter samples primarily because of the effect of disturbance during sampling. This point was confirmed by tests carried out under the main contract in the TRL instrumented area on 100mm diameter triaxial samples which, when tested under a confining pressure of 50kPa and an applied head of 3m of water, gave a mean permeability of 1.2×10^{-8}m/s.

6.1.2 In situ permeability tests

Constant head permeability tests were carried out on the hydraulic piezometers installed in the soft piles at both

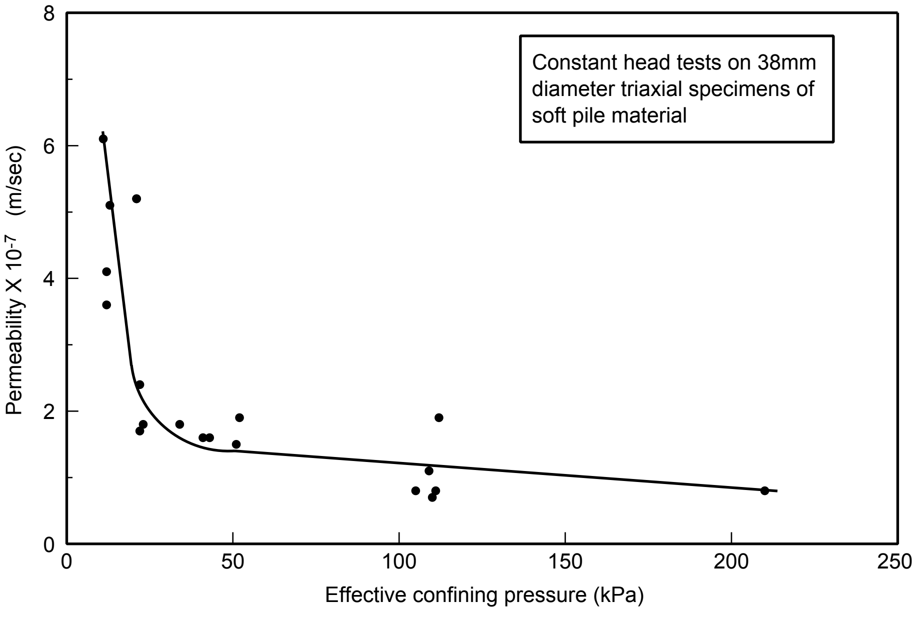


Figure 6 Dependence of permeability on confining stress (triaxial cell)

chainages 3540 and 3370. Initial tests indicated that no measurable flow of water was obtained at low heads over a test duration of a few hours and for this reason a high head (h) of about 18m of water was applied. Permeability (k) was then calculated using the formula:

$$k = Q/(4\pi rh)$$

where r is the radius of the equivalent sphere with the same surface area as that of the 38mm diameter cored hole into which the piezometer tip was installed and sealed. Even with this high head only very small flow rates (Q) were recorded over 6 hours.

The tests carried out at chainages 3540 and 3370 indicated that the *in situ* permeability of the soft pile material was of the order of 1.9×10^{-11}m/s and 1.7×10^{-11}m/s respectively.

6.1.3 Discussion of permeability results

A significant variation in permeability was recorded for the soft pile material depending on the test procedure employed. The effect of a small sample size and disturbance during coring gave a high permeability of 1.35×10^{-7}m/s from 38mm diameter triaxial tests and this result was considered to be unrepresentative of the actual properties. Tests carried out under the main contract on 100mm diameter triaxial samples gave a mean permeability of 1.2×10^{-8}m/s after about 28 days and provided a better indicator, although this value may be regarded as a conservative value for design purposes as the low confining pressures used in the triaxial cell may have permitted some seepage between the membrane and specimen. *In situ* constant head permeability tests on piezometers installed in the soft piles suggested an even lower value of 1.8×10^{-11}m/s.

It is interesting to note that Hazen (1892) carried out experimental work on fairly uniform sands and related permeability to effective particle size D_{10} (the size for which 10% of particles are finer) by the equation:

$$k = 0.01\ (D_{10})^2 \text{ m/s}$$

where D_{10} is expressed in metres. Given that D_{10} for the soft pile material is <0.06mm (see Section 6.3), this implies a permeability of $<3.6\times10^{-11}$m/s. This is of the same order of magnitude as that measured in the *in situ* permeability tests.

6.2 Undrained shear strength

A limited number of quick undrained triaxial tests were conducted on the 38mm diameter samples of soft pile material according to Clause 8 of BS1377: Part 7 (1990). Using a confining stress of 50kPa, undrained shear strengths (c_u) at depths of 3m and 5.55m were 250kPa and 366kPa respectively.

These values can be compared with those measured under the main contract in the TRL instrumented areas at 28 days when the undrained shear strengths of 100mm diameter samples were between 264kPa and 484kPa.

6.3 Particle size distribution

Trimmings from the permeability samples were pretreated with hydrochloric acid to break down the cementitious bonds and fully disaggregate the particles. The residue was then wet sieved to remove remaining fines before final dry sieving in accordance with Clause 9.2.4 of BS1377: Part 2 (1990). The resulting particle size distributions are shown in Figure 7 and indicate that over 66% of the original samples were retained on the 63ìm sieve, almost all derived from the natural sand aggregate.

A chemical analysis was carried out on a portion of the sample to determine the cement content following procedures given in BS1881: Part 124 (1988) for analysis of hardened concrete. Inspection of the results in conjunction

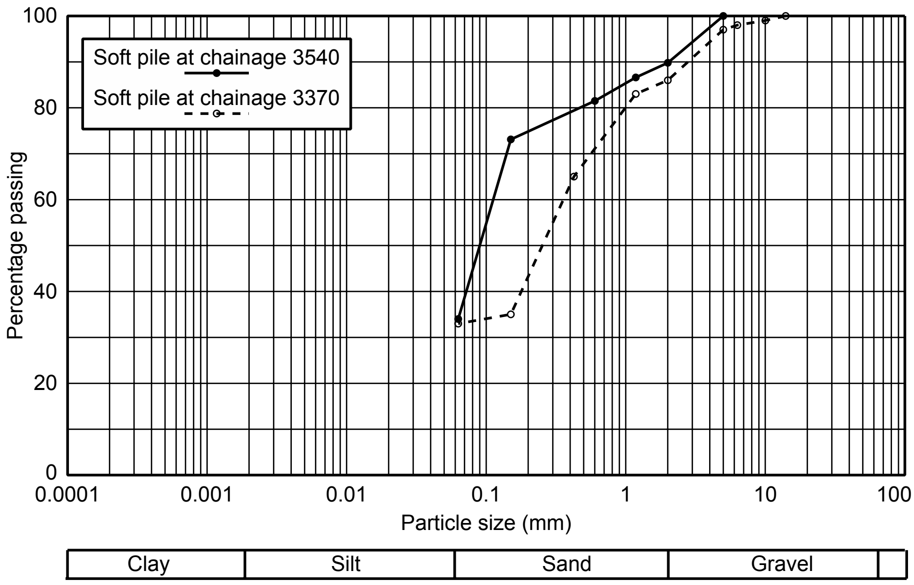


Figure 7 Particle size distribution of soft pile material

with a petrographic examination confirmed that the material comprised Portland cement and aggregate; bentonite could not be specifically identified using these test methods. The cement content calculated from the acid soluble calcium content was found to be 5.4% by weight after making a typical allowance for 10% CaO fraction in the aggregate. This cement content result should not be viewed as very reliable as there were shell fragments present in the aggregate which could affect the results. The design value for cement content was 10.6% by weight.

7 *In situ* permeability testing of the ground

Constant head permeability tests were carried on hydraulic piezometers 27 and 26 installed at 14.64mAOD and 18.24mAOD in the Taplow gravel and London Clay at the instrumented section at chainage 3540. Similar tests were performed on piezometers 22 and 21 at 15.21mAOD and 18.41mAOD in the instrumented section at chainage 3370.

In all cases water was passed into the piezometer using one of its hydraulic tubes and the test was continued until the rate of flow (Q) had stabilised. The head (h) at the tip was determined using a vibrating wire transducer installed at the control panel on the sealed end of the other hydraulic tube from the piezometer. The permeability was then calculated from the formula:

$$\text{Permeability} = Q/(4\pi rh)$$

where r is the radius of the equivalent sphere with the same surface area as that of the sand cell in which the piezometer was installed.

The results are summarised in Table 5. The average permeability of the London Clay was 2×10^{-10}m/s, this was similar to *in situ* permeabilities measured elsewhere in the unweathered over-consolidated formation (Cripps and Taylor, 1986). The measured permeabilities of the Taplow gravel were more variable and ranged between 3×10^{-7}m/s and 1×10^{-5}m/s at the two instrumented areas. Although higher permeabilities of about 10^{-4}m/s would be expected for clean sand-gravel mixtures (BS8004, 1986), the measured values at this site were lower because of the clayey nature of the Taplow gravel. The presence of clay bands and the variation in clay content were considered to account for the scatter in permeability.

Table 5 Permeability results in the retained ground

	Chainage 3540		*Chainage 3370*	
Soil type	*Piezometer no.*	*Permeability (m/s)*	*Piezometer no.*	*Permeability (m/s)*
Taplow gravel	27	3×10^{-7}	22	1×10^{-5}
London Clay	26	1×10^{-10}	21	3×10^{-10}

8 Observations

8.1 Measurements at chainage 3540

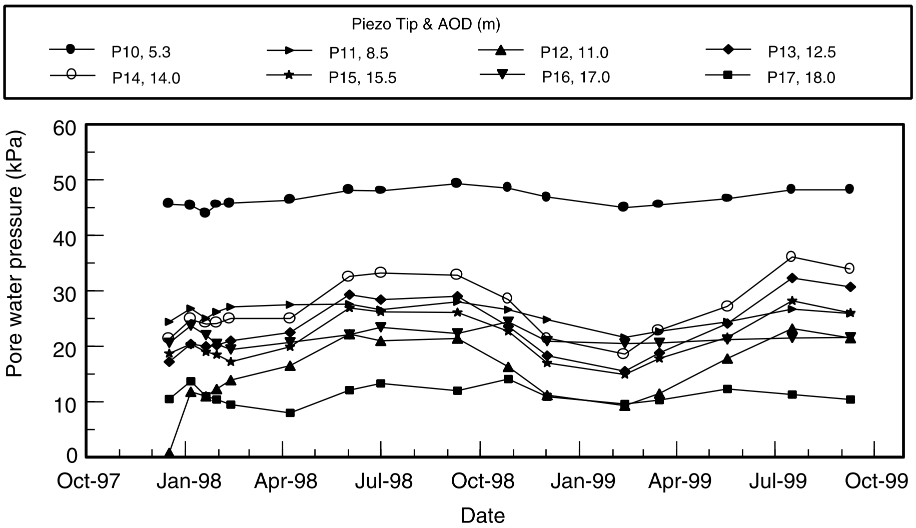
The seasonal variation of pore water pressures is plotted in Figure 8 for the piezometers at 1.75m, 3m and 9m behind the soft pile at chainage 3540. Generally greater fluctuations in pore water pressure were measured closer to the wall than at 9m away from it. These fluctuations were therefore considered to be associated with the behaviour of the wall rather than as a result of the seasonal rise and fall in the water table. This was particularly the case as maximum pore pressures tended to occur in the summer months when the water table would be expected to be at its lowest. These changes may well be associated with thermal expansion and contraction of the prop slab located immediately beneath the carriageway in front of the wall. Figure 9 shows measurements of lateral movements near the top of the wall, provided by WS Atkins Consultants Ltd, which are consistent with this proposed mechanism. Similar behaviour has been reported by Carder and Symons (1989) for a propped retaining wall embedded in stiff clay on the A3 where increases in both total stress and pore water pressure occurred during the summer months.

Because of this seasonal variation, profiles of pore water pressure with depth were plotted in Figure 10 for July 1998 and February 1999. At these times the pore water pressures were at a seasonal minimum and maximum respectively. Inspection of the pore water pressure profile at 9m behind the soft pile and away from the influence of wall construction indicates that a perched water table exists in the overlying Taplow gravel which is consistent with that of a hydrostatic distribution with depth. The pore water pressure in the uppermost 5m of the underlying London Clay then remains fairly constant with depth with values between 15kPa and 30kPa. Below 10mAOD the pore water pressure then increases again with depth as indicated in Figure 10c.

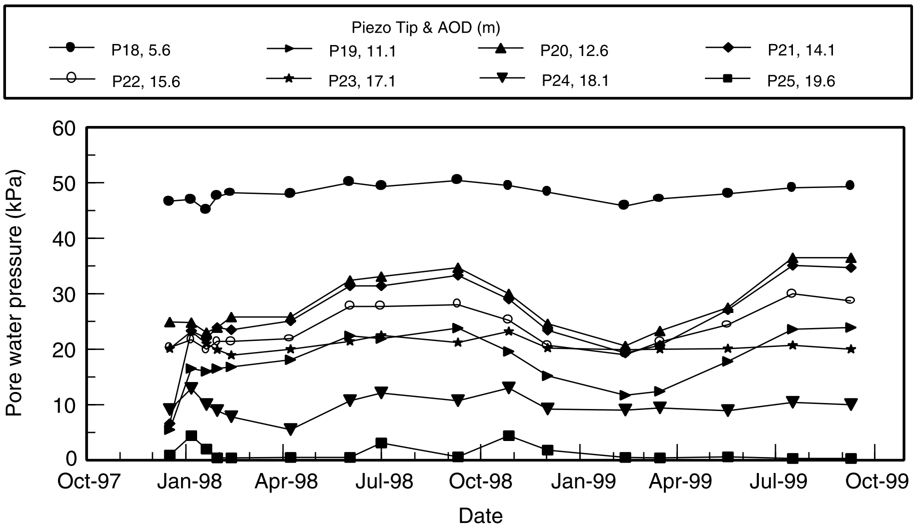
Comparison of the piezometer measurements at 1.75m, 3m and 9m behind the soft pile given in Figure 10 shows a very similar pattern in the pore water pressure distributions although the seasonal variations are larger nearer to the wall. As previously mentioned these variations are probably associated with small wall movements induced by thermal expansion and contraction of the carriageway prop slab.

Generally the similarity of the pore water pressure distributions in Figure 10 indicates that the retained height of the soft pile wall (which is finished with sprayed concrete) is tending to act as an impermeable barrier. However further evaluation of the results is necessary as it could be argued that the slight dip in pore pressures at about 11mAOD is coincident with the excavation level in front of the wall and may indicate some seepage through the soft piles below this level. Alternatively the dip in pressures could be accounted for solely by the existence of the perched water table in the overlying gravel at this site. A seepage analysis was therefore undertaken to investigate these possibilities and this is described in Section 9.

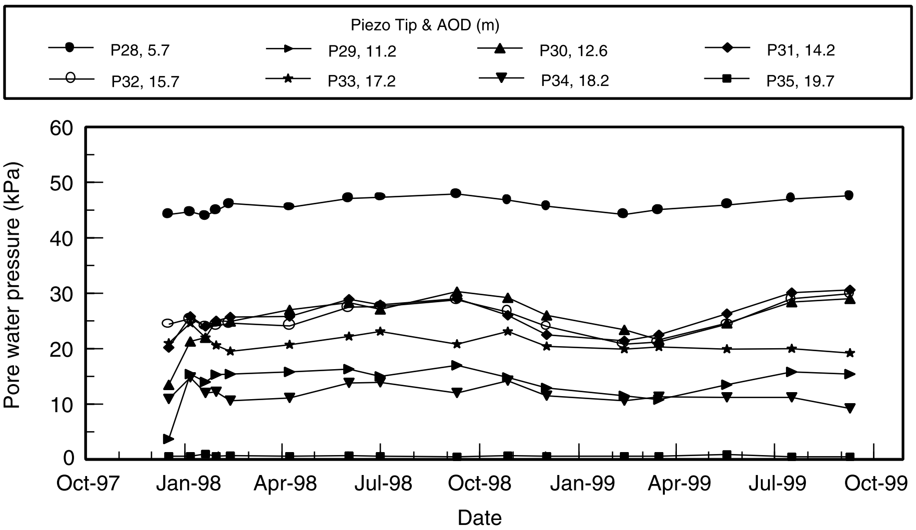
Pore water pressures measured using the piezometers installed in the soft piles did not give meaningful results



a) 1.75m behind soft pile

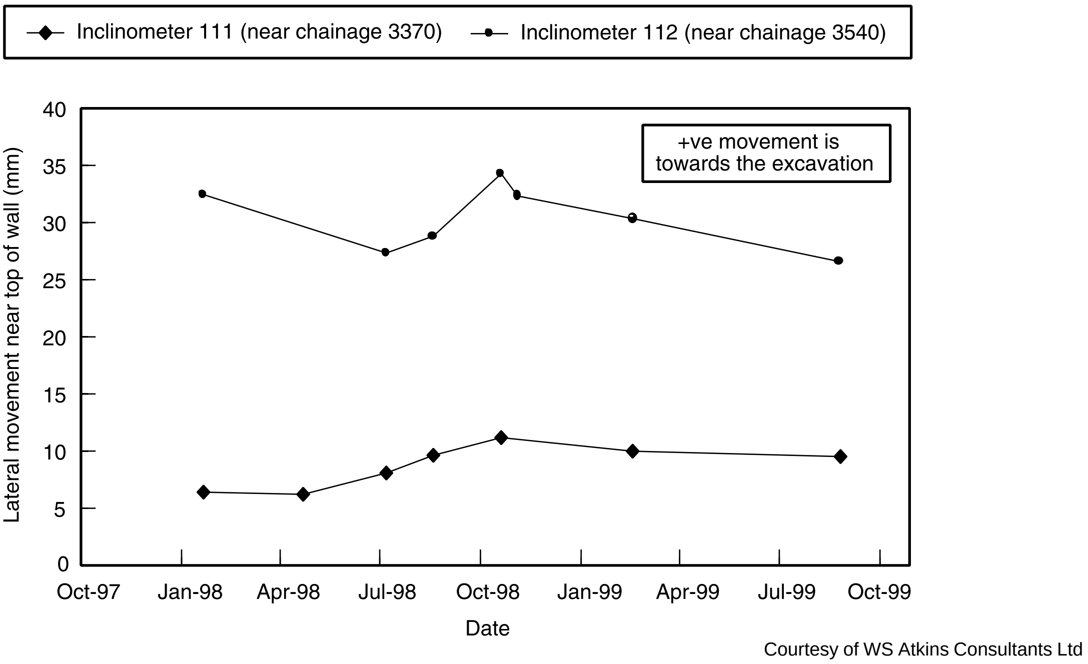


b) 3m behind soft pile



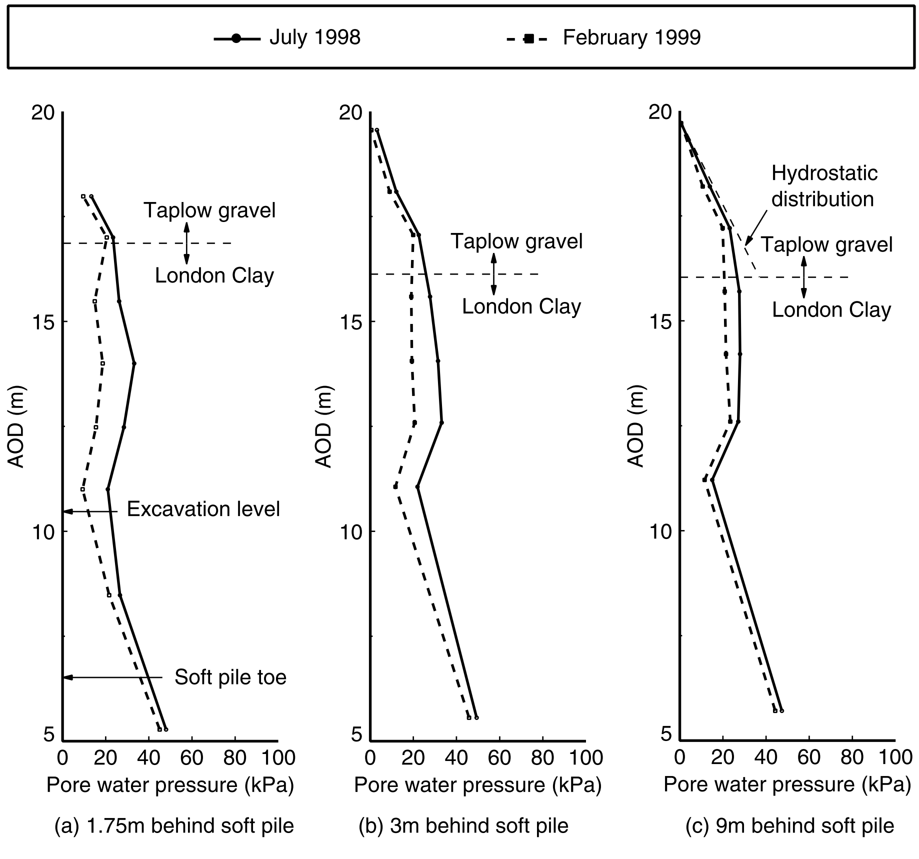
c) 9m behind soft pile

Figure 8 Seasonal variation of pore water pressure at chainage 3540



Courtesy of WS Atkins Consultants Ltd

Figure 9 Seasonal variation of lateral wall movement



(a) 1.75m behind soft pile

(b) 3m behind soft pile

(c) 9m behind soft pile

Figure 10 Distribution of pore water pressure at chainage 3540 because of the impermeable nature of the piles. For this reason, the readings tended to reflect the water pressures last used in de-airing the piezometer tips rather than a true pore pressure. These results are therefore not presented.

Results from the water collection system in front of one of the soft piles (see Section 5.3) showed little change in level over the period of monitoring. Generally the level of the water remained within the cavity formed by the concrete traffic plinth in front of the wall and the plastic pipe used for its collection. It was therefore concluded that any quantities of water seeping through the soft piles into this system were small.

8.2 Measurements at chainage 3370

Figure 11 shows the seasonal variation of pore water pressure recorded by the piezometers in the retained ground at chainage 3370. The pattern of the changes is not quite so well established as at chainage 3540 as the piezometers were installed at a later date and monitoring was not commenced until the end of May 1998. A preliminary assessment however suggests that a similar pattern of behaviour occurred with less fluctuation in pore water pressure further away from the wall and also little change in pressure at depth.

At chainage 3370, three piezometers were also installed at depths of 2m, 3.5m and 5m below the carriageway prop slab in a borehole at 2m in front of the wall. The variation of these piezometer readings with time is plotted in Figure 12. It is worth noting that in Figure 12 the seasonal maxima and minima occurred during October and March and were slightly out-of-phase with those on the retained side of the wall (Figure 11).

A further assessment of the seasonal changes is given in Figure 13 by comparing readings taken in June 1998 and February 1999. A perched water table again existed in the Taplow gravel which overlay the London Clay at this site. At this chainage the depth to the London Clay was slightly greater than at chainage 3540 and the retained height of the wall was less. Examination of the results in Figures 13b and 13c confirmed the seasonal trend of higher pore water pressures during late summer and slightly lower pressures in the winter. Once again this may be attributable to the undrained response of the retained ground to the thermal loading generated in the carriageway prop slab. Any seasonal variation at 9m behind the wall (Figure 13d) was small.

As reported at the other instrumented location, no meaningful pore water pressures were recorded by the piezometers installed in the soft piles because of their impermeability. Whereas small quantities of water were collected in the pipe in front of the soft pile at chainage 3540, the pipe at chainage 3370 remained nearly dry throughout.

9 Predictions from seepage analyses

A preliminary assessment of the pattern of water seepage in the vicinity of the retaining wall has been carried out using the program SEFTRANS which is a finite element code for analysing two-dimensional ground water flow problems. The major assumption of the flow equation is that flow is governed by Darcy’s Law.

Two boundary conditions were employed to investigate the following distributions of water pressure in the retained ground remote from the wall. They were:

a a hydrostatic distribution of pore water pressure with depth;

b the measured pore water pressure distribution, ie. with a perched water table in the Taplow gravel.

Throughout the analyses the pore water pressure at the interface between the clay and the reinforced concrete prop slab was controlled to zero to model the presence of the drainage layer beneath the slab. In each case the sensitivity of the findings to different soft pile permeabilities was investigated and, for the purpose of the analysis, no account was taken of the fact that the exposed area of soft pile was finished with sprayed concrete. This enabled an evaluation to be made on the basis of the likely scenario if sprayed concrete had not been employed.

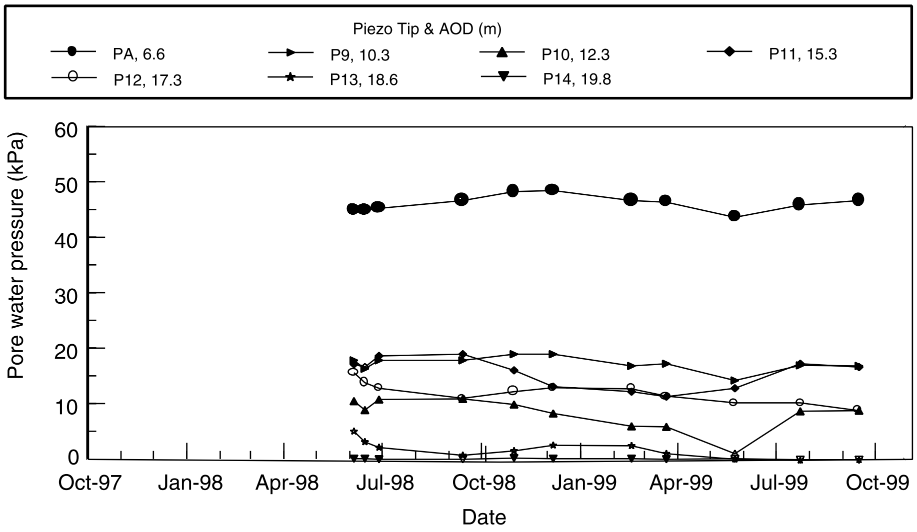
Pore water pressures were predicted at the same distances from the wall as those at which the piezometers were installed, so that a comparison between the results was readily achieved.

9.1 Results at chainage 3540

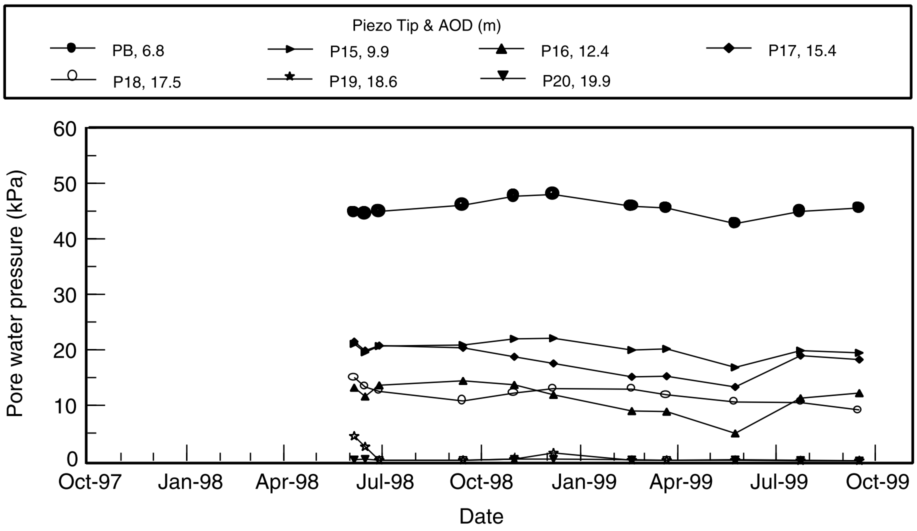
The geometry adopted for the numerical modelling followed the construction details given in Figure 3. The permeabilities of the Taplow gravel and the London Clay were as given in Table 5.

At this chainage the bored piles were of 1.8m diameter and at 2m centres. Given that the reinforced concrete piles can be deemed nearly impermeable, the wall permeability was mainly dependent on that of the material in the 0.2m gap between piles. Because of the ratio of widths of pile to gap, the equivalent permeability of the wall in the two-dimensional analysis will actually be approximately ten times less than that of the material in the gap. This means that the equivalent wall permeability where clay is present between piles has been taken as 10^{-11}m/s. Where the gap between hard piles is filled by a soft pile, equivalent wall permeabilities have been similarly calculated to investigate the permeability range measured in 100mm diameter triaxial and *in situ* tests on the soft pile material (Section 6.1.3).

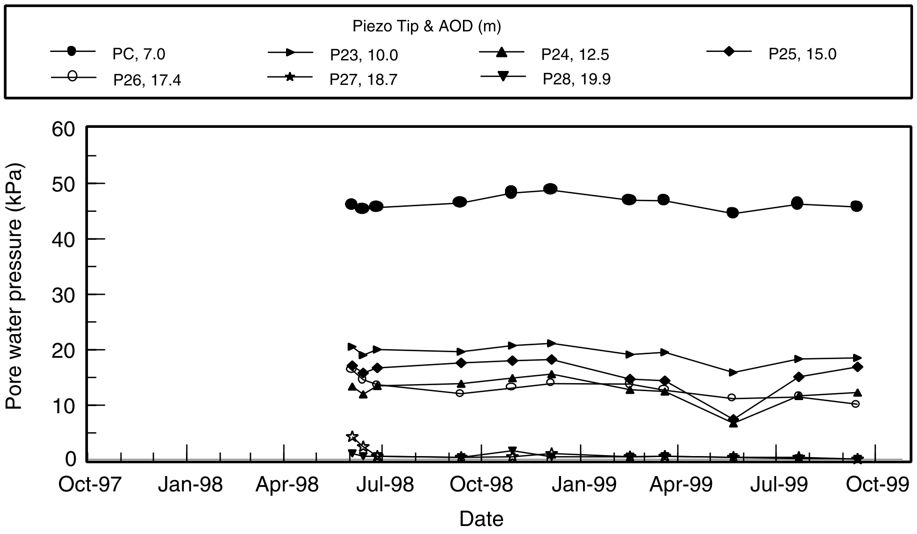
Figure 14 shows the distributions of pore water pressure at 1.75m, 3m and 9m behind the wall predicted on the basis of an idealised hydrostatic distribution of pressure with depth at a free field location, ie. remote from the wall. Comparison of the predictions close to the retaining wall (Figure 14a and 14b) with the measurements in Figure 10 shows some similarities particularly when the wall is more permeable. Close to the wall the predicted profiles of pore pressure resemble that expected if a perched water table existed, although the mechanism is quite different in so far as the shape of the profiles is produced by flow through the wall in addition to steady state seepage around its toe. This point is further demonstrated by examination of the predicted and measured pore pressure profiles at 9m behind the wall which are shown in Figures 14c and 10c respectively and are, as expected, very different. At this distance, the predicted pore water pressures increase near linearly with depth as would be anticipated because of the



a) 1.75m behind soft pile



b) 3m behind soft pile



c) 9m behind soft pile

Figure 11 Seasonal variation of pore water pressure at chainage 3370

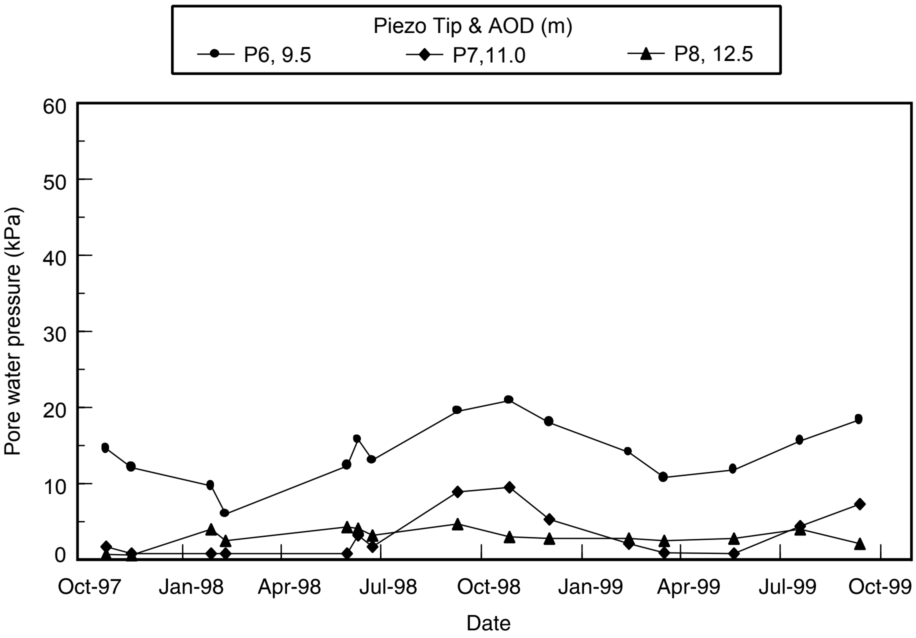
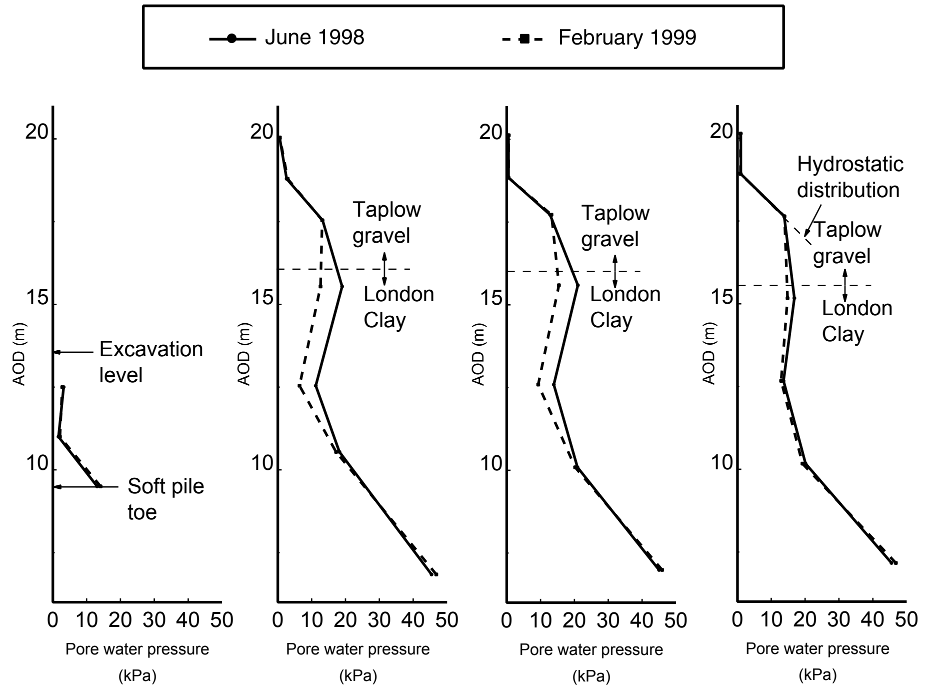


Figure 12 Seasonal variation of pore water pressure in front of wall at chainage 3370



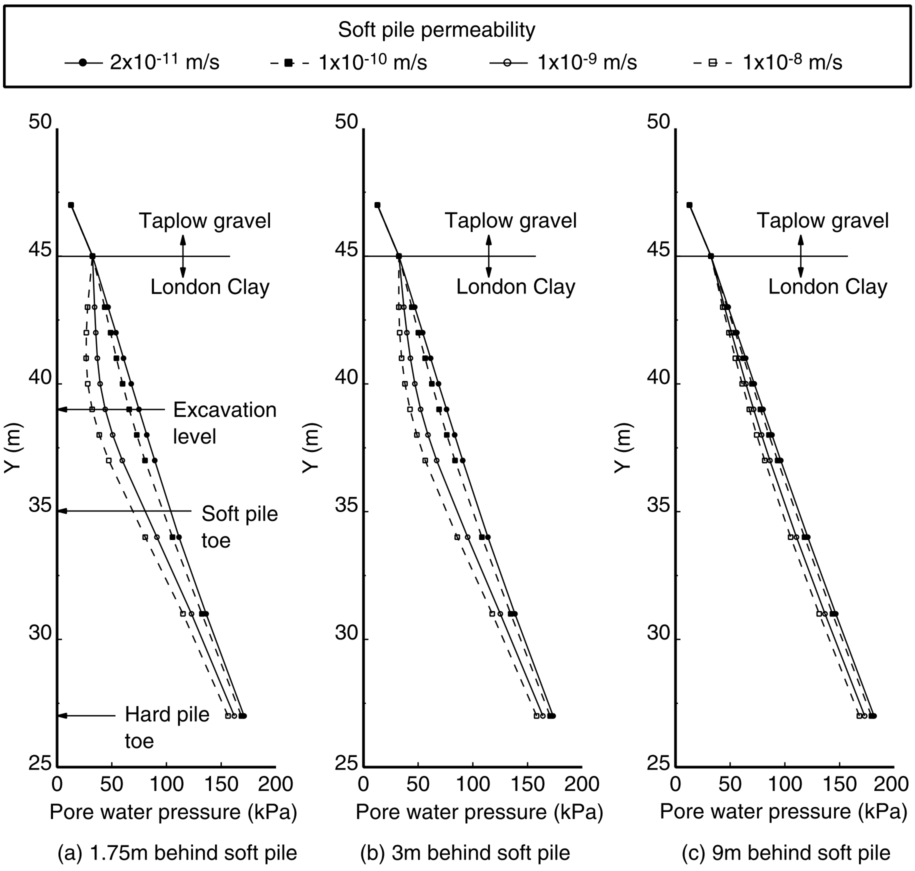
(a) 2m in front of wall

(b) 1.75m behind soft pile

(c) 3m behind soft pile

(d) 9m behind soft pile

Figure 13 Distribution of pore water pressure at chainage 3370



(a) 1.75m behind soft pile (b) 3m behind soft pile (c) 9m behind soft pile

Figure 14 Predicted distribution of pore water pressure at chainage 3540 assuming *in situ* hydrostatic distribution

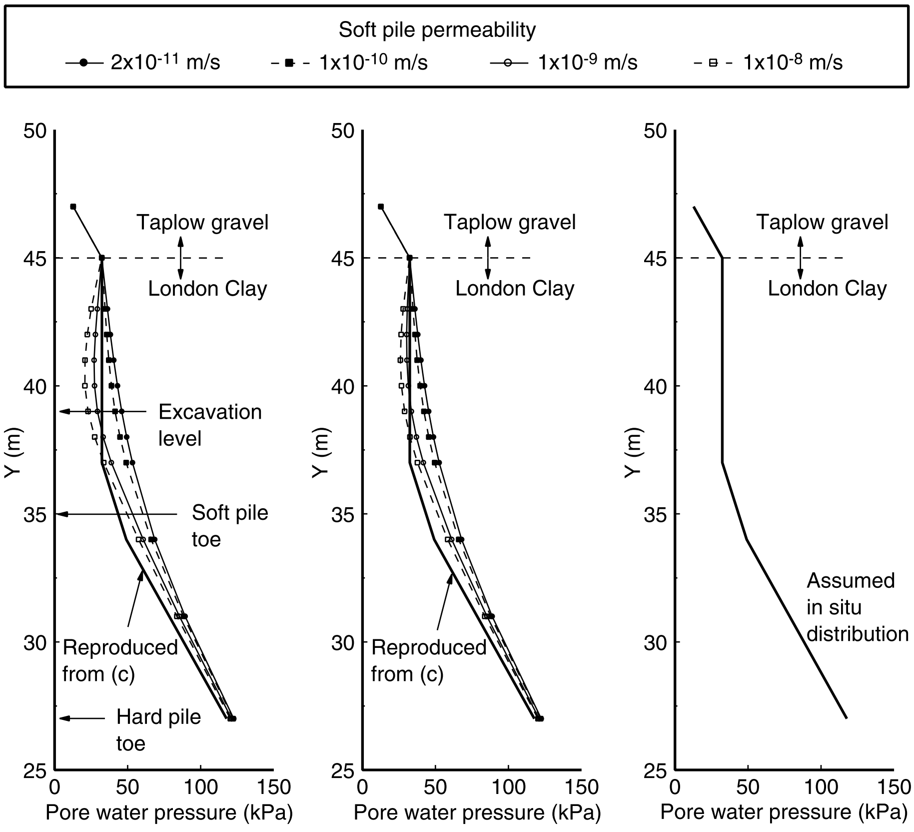
prescribed hydrostatic distribution remote from the wall. However the measured pressures confirm the existence of a perched wall table at this site and a further analysis which is considered to be more realistic was therefore carried out on this basis.

The results of this analysis assuming a perched water table as boundary condition are shown in Figure 15. The distribution of pore water pressure with depth in the retained ground at 9m away from the wall (Figure 15c) is based on the site measurements. However predicted pressure profiles at closer distances to the wall (Figures 15a and 15b) for soft pile permeabilities of 1×10^{-8}m/s and 1×10^{-9}m/s indicate that some flow then occurs through the exposed part of the wall which causes an associated fall in pressures below the *in situ* values. When the soft piles are more impermeable the water pressures slightly exceed the *in situ* values because there is no significant flow through the wall and recharging occurs from the head in the Taplow gravel. Based on the measured *in situ* permeability of 2×10^{-11}m/s for the soft piles, the analysis would suggest that any significant water seepage through the wall would have been unlikely even if the sprayed concrete finish had been omitted.

The effects of recharging from the head in the Taplow gravel were separately investigated by assuming anisotropic permeability for the London Clay. The value for clay permeability used in the previous analyses was isotropic and based on *in situ* values from permeability tests carried out on the hydraulic piezometers. As these piezometers were installed in boreholes with impermeable bentonite seals above and below their tips, they were considered to be more indicative of the horizontal permeability of the clay. In practice because of the presence of sand lenses in the clay, London Clay is commonly considered to be about 5 times more permeable in the horizontal than in the vertical direction. The analyses in Figure 15 were therefore repeated with a revised vertical permeability and the results are shown in Figure 16. These results confirm that the effect of water pressures exceeding the *in situ* values, because of recharging, has then been largely eliminated from Figures 16a and 16b. Comparison of the pore pressure predictions close to the wall (Figure 16a) with the measured values in Figure 10a, confirms that closest agreement occurs when the wall is modelled as being relatively impermeable.

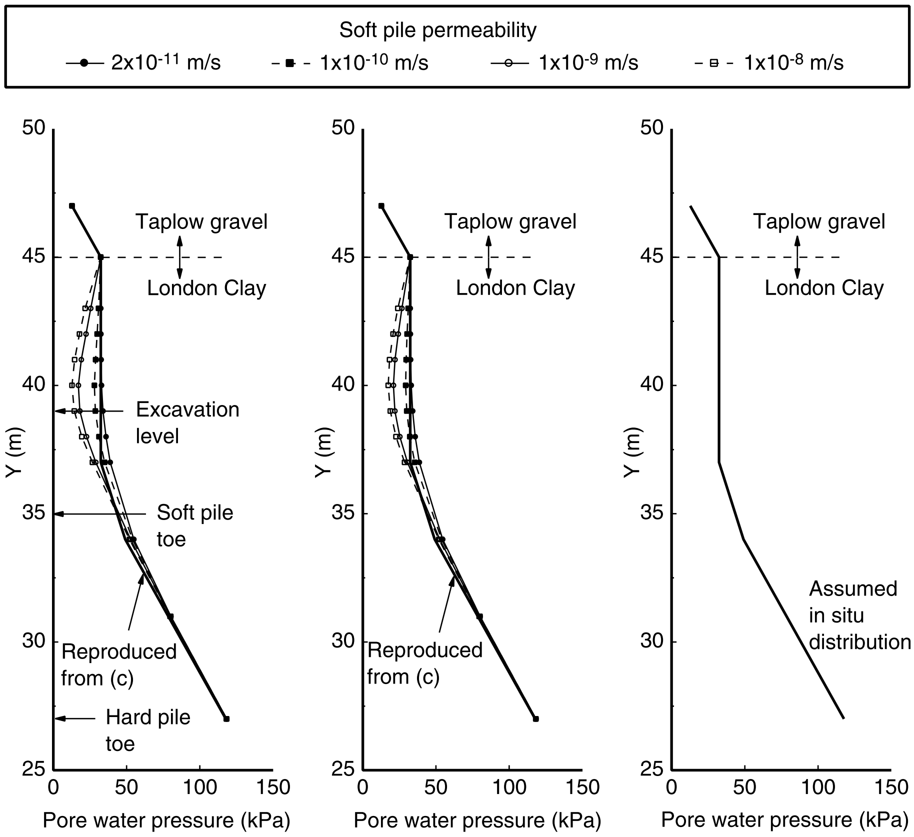
9.2 Results at chainage 3370

The wall dimensions and soils data used in the numerical modelling were as given in Figure 4 and Table 5 respectively. At chainage 3370, the hard piles were 1.5m in diameter and installed at 1.7m centres. The wall permeability was again considered to be mainly dependent on that of the soft pile material in the 0.2m gap between piles. A similar approach to that described in Section 9.1 for chainage 3540 for determining the effective wall permeabilities (in both the hard-soft and the hard-clay parts) was therefore adopted.



(a) 1.75m behind soft pile (b) 3m behind soft pile (c) 9m behind soft pile

Figure 15 Predicted distribution of pore water pressure at chainage 3540 assuming *in situ* perched water table



(a) 1.75m behind soft pile (b) 3m behind soft pile (c) 9m behind soft pile

Figure 16 Predicted distribution of pore water pressure at chainage 3540 assuming *in situ* perched water table and anisotropic clay permeability

Figures 17 and 18 show the respective distributions of pore water pressure with depth calculated when a hydrostatic distribution and the measured distribution of pressure were assumed for the boundary condition on the retained side of the wall. The results in Figure 17c indicate that pore pressures would still be expected to be near hydrostatic at 9m behind the wall if a hydrostatic distribution is prescribed remotely. However measured values in Figure 13c indicate that a perched water table existed at 9m behind the wall. Better agreement with the measured pressure distributions is therefore obtained in Figure 18 when the perched water table is input as boundary condition for the numerical modelling.

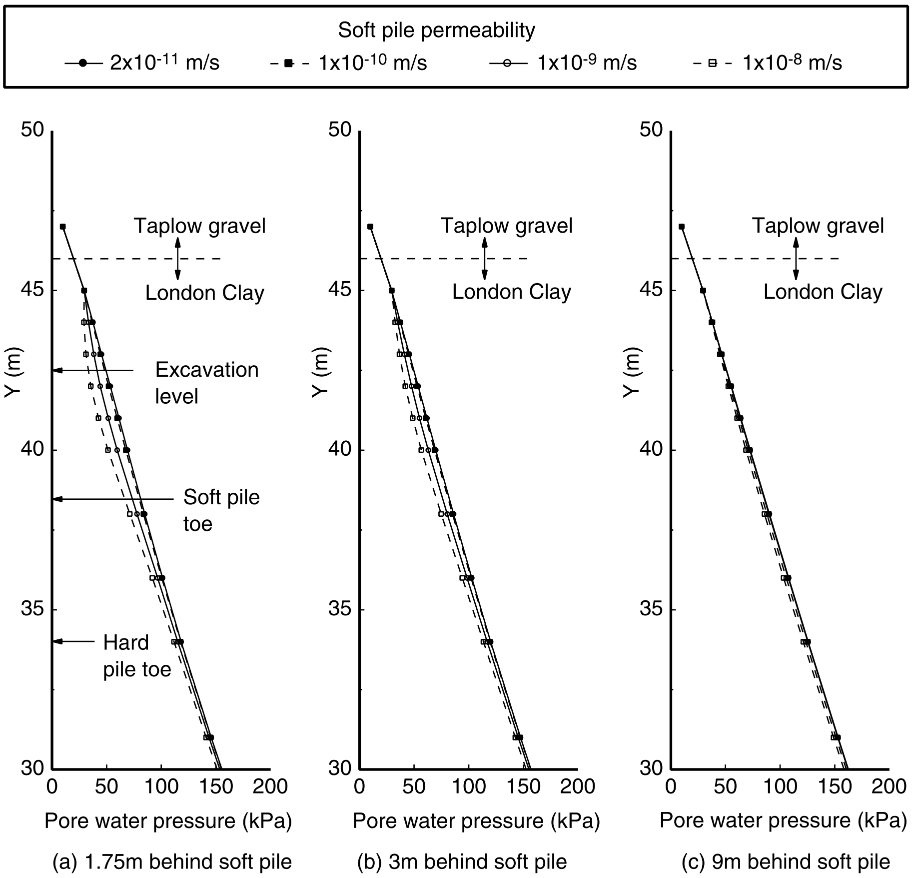
When the soil parameters are further refined using a lower permeability in the vertical direction for the London Clay together with a perched water table at the boundary, the calculated pore pressure distributions are shown in Figure 19. Generally, as the effects of recharge from the head in the Taplow gravel are then minimised, pore pressures are lower than those given in Figure 18 and correlate better with the piezometer measurements. At chainage 3370, only small differences in pore pressure were predicted as the soft pile permeability changed from 2×10^{-11}m/s to 1×10^{-8}m/s. This was because the retained height of the wall was significantly less than at chainage 3540 and seepage effects were therefore of smaller magnitude.

9.3 Predicted water efflux rates

The predicted efflux rates of water through the exposed faces of the retaining wall were also obtained from mass balance calculations available in the numerical seepage routine. The results from the final analyses using lower vertical than horizontal permeabilities for the London Clay are given in Table 6 for the two locations.

Table 6 Predicted flow of water through a metre run of wall

Chainage	*Soft pile permeability (m/s)*	*Efflux rate through boundary (m^3/s)*	*Equivalent flow (litres per year)*
3540	1×10^{-8}	2.2×10^{-9}	70.8
	1×10^{-9}	8.5×10^{-10}	26.8
	1×10^{-10}	6.7×10^{-10}	21.1
	2×10^{-11}	6.5×10^{-10}	20.6
3370	1×10^{-8}	3.1×10^{-9}	96.5
	1×10^{-9}	1.7×10^{-9}	55.0
	1×10^{-10}	1.6×10^{-9}	50.2
	2×10^{-11}	1.6×10^{-9}	50.0



(a) 1.75m behind soft pile (b) 3m behind soft pile (c) 9m behind soft pile

Figure 17 Predicted distribution of pore water pressure at chainage 3370 assuming *in situ* hydrostatic distribution

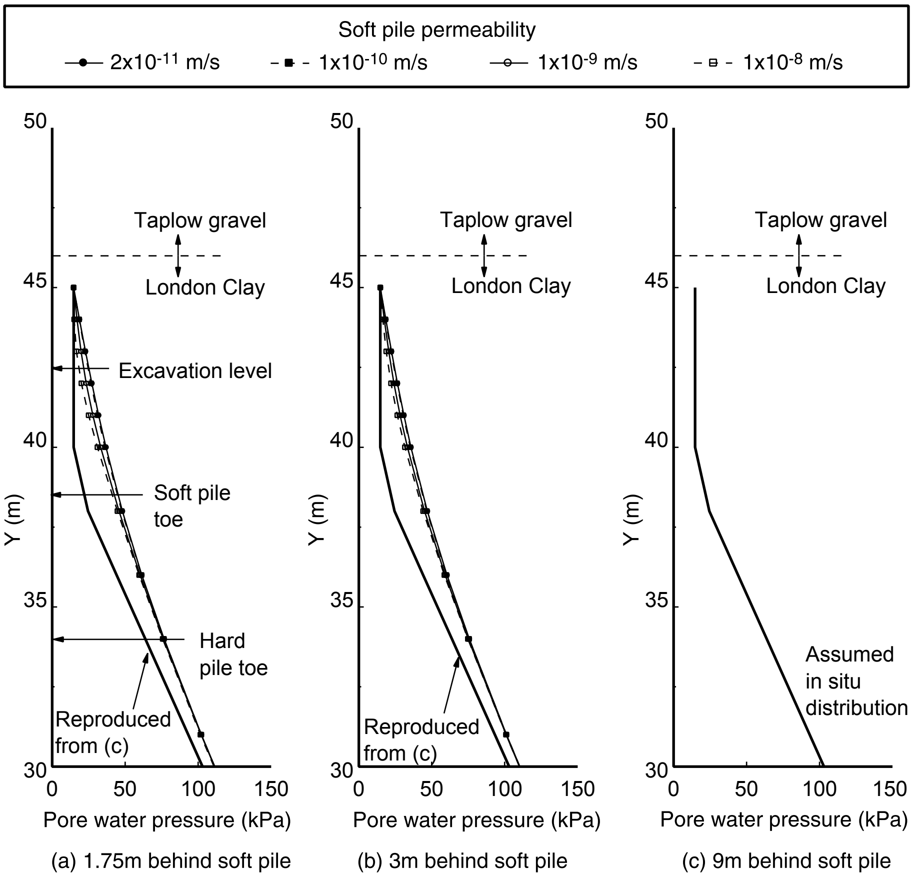


Figure 18 Predicted distribution of pore water pressure at chainage 3370 assuming *in situ* perched water table

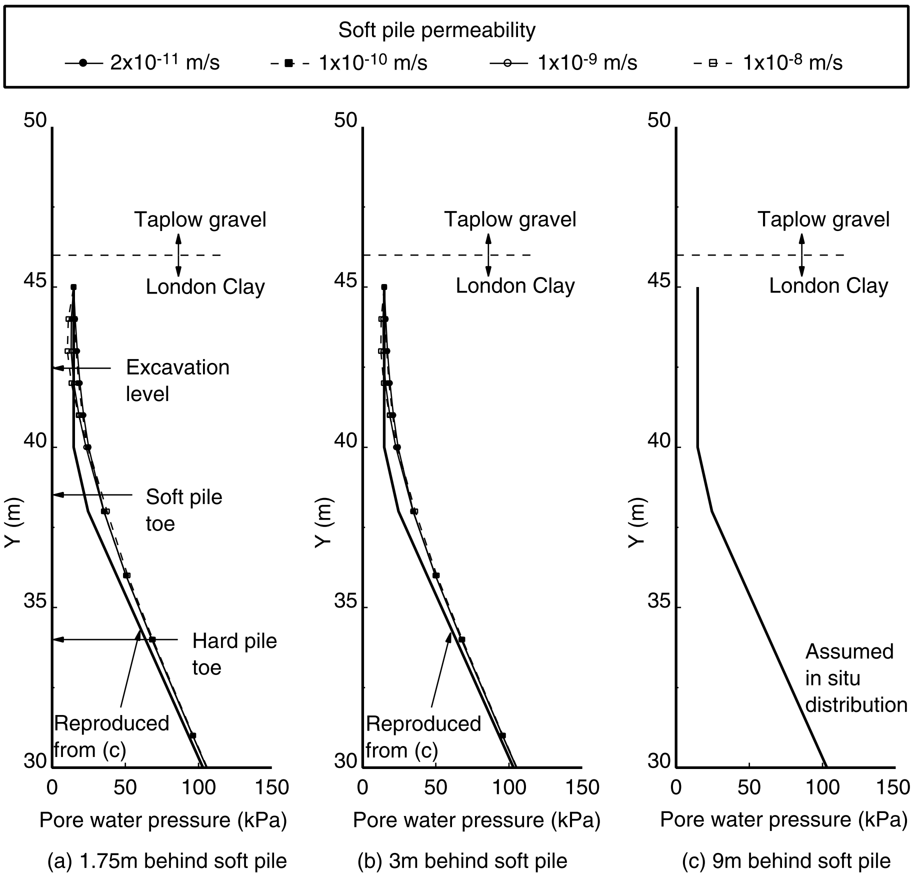


Figure 19 Predicted distribution of pore water pressure at chainage 3370 assuming in situ perched water table and anisotropic clay permeability

As would be anticipated the flow of water through the wall increased as its soft pile component became more permeable. However even when the soft material was at its most permeable value of 1×10^{-8}m/s, the flow of water over the retained height per metre run of wall was only 70.8 and 96.5 litres/year at chainages 3540 and 3370 respectively. Even if the soft pile had not been protected by the sprayed concrete facing at this site, it is envisaged that this quantity of water would probably have been lost through evaporation with any excess being readily coped with by the drainage system between the structural wall and its cladding.

It is worth noting that there is an apparent anomaly in Table 6 when comparing flow through the wall at chainages 3540 and 3370. At first sight it would be anticipated that a larger flow would be predicted where the retained height was larger at chainage 3540. The reason this does not occur is probably because the measured permeabilities used for the Taplow gravel and London Clay (Table 5) are marginally different at the two locations.

10 Durability issues

Concerns also exist about the durability of the soft pile material as any significant flow of groundwater through the piles might result in migration of cement or bentonite from the pile. In theory this could lead to either the soft material losing strength or becoming more permeable. For this reason, the water collection system described in Section 5.3 was installed in front of one of the soft piles at each of the two instrumented areas. This enabled some assessment of the rate of water flow and also permitted a chemical analysis of the water collected.

10.1 Measured flow of water through the soft piles

Throughout the period of monitoring, no measurable flow of water into the collection system could be detected at chainage 3370.

However some flow of water into the collection system at chainage 3540 did occur. With the exception of periods immediately after emptying the collection system, the water level in the collection duct remained consistent with that of the top of the traffic plinth in front of the wall (Figure 3). On occasions when the collection duct was emptied, the duct took about 4 months to fill up again. This corresponded to a flow rate of about 46 litres/year per metre run of wall, which was of the same order of magnitude as that predicted from the seepage analysis.

10.2 Chemical analysis of the collected water

Two water samples were retrieved, one at the beginning and the other at the end of 1998, from the collection system at chainage 3540. Their suspended solids contents were prepared for examination by analytical electron microscopy and X-ray diffraction. Results from the two samples were similar.

The examination by electron microscopy did not provide any evidence of the migration of bentonite from the soft piles. Although the samples were silica rich, particle sizes suggested that the silica was associated with the London Clay rather than with bentonite.

Chemical analysis of the solids residue indicated that there were calcium components (about 40% of the residue) which may have been derived from the groundwater, the cement in the soft pile, or the adjoining structural concrete. However diffractometer results suggested that a crystalline component corresponding to calcium carbonate was present, whereas calcium silicate (related to cement) was not detectable. The calcium component was therefore most likely to have resulted from the groundwater.

It was therefore concluded that there was no significant evidence to suggest migration of either the bentonite or cement from the soft piles, which was hardly surprising in view of the low water seepage rate.

11 Site specific conclusions

The following conclusions have been made from field monitoring of the performance of the soft cement-bentonite piles at the A12 Hackney to M11 Link Road:

i A significant variation in permeability was recorded for the soft pile material depending on the test procedure employed. The effect of a small sample size and disturbance during coring gave a high permeability of 1.35×10^{-7}m/s from 38mm diameter triaxial tests and this result was considered to be unrepresentative of the actual properties. Tests carried out under the main contract on 100mm diameter triaxial samples gave a mean permeability of 1.2×10^{-8}m/s after about 28 days and provided a better indicator, although this may be regarded as a conservative value for design purposes as the low confining pressures used in the triaxial cell may have permitted some seepage between the membrane and specimen. *In situ* constant head permeability tests on piezometers installed in the soft piles indicated an even lower value of 1.8×10^{-11}m/s which was consistent with that expected from the effective particle size.

ii The undrained shear strength of the cement-bentonite piles exceeded the minimum strength of 150kPa at 28 days which was specified in the construction contract.

iii Measurements of the pore water pressure distribution in the retained ground at both chainages 3540 and 3370 confirmed the existence of a perched water table in the Taplow gravel overlying the London Clay. Very little difference in the pore water pressure distributions was measured in the retained ground at 1.75m, 3m and 9m behind the soft pile. This confirmed that the soft piles (whose exposed faces were finished with sprayed concrete) were tending to act as an impermeable barrier.

iv Generally greater fluctuations in pore water pressure were measured closer to the wall than at 9m away from it. These fluctuations were therefore considered to be associated with the behaviour of the wall rather than the result of the seasonal rise and fall in the water table. This was particularly the case as maximum pore pressures tended to occur in the summer months when the water table would be expected to be at its lowest. These changes may well be associated with thermal expansion and contraction of the prop slab located immediately beneath the carriageway in front of the wall. Similar behaviour has been reported by Carder and Symons (1989) for a propped retaining wall embedded in stiff clay on the A3 where increases in both total stress and pore water pressure occurred during the summer months.

v A numerical seepage analysis was carried out to investigate the likely scenario if a sprayed concrete finish to the soft piles at chainages 3540 and 3370 had not been used. When the presence of a perched water table was modelled and used in conjunction with measured permeabilities of the ground and soft pile, the analysis suggested that any significant water seepage through the soft piles would have been unlikely even if the sprayed concrete finish had been omitted. A further analysis based on a hydrostatic distribution of pore water pressure with depth demonstrated that the reduction in measured pore pressures at the top of the clay strata was a consequence of the perched water table rather than the effect of water seepage through the soft piles.

vi Only small efflux rates of water through the exposed faces of the retaining wall were predicted from mass balance calculations available in the numerical seepage routine. Even if the soft pile had not been protected by the sprayed concrete facing at this site, it was therefore envisaged that this water would probably have been lost through evaporation with any excess being readily coped with by the drainage system between the structural wall and its cladding. The measured flow of water through an exposed soft pile at chainage 3540 was about 46 litres/year per metre run of wall, which was of the same order of magnitude as that predicted from the seepage analysis. At chainage 3370, there was no measurable flow of water into the collection system in front of the soft pile whereas some had been predicted.

vi The suspended solids contents from water samples retrieved from the collection system at chainage 3540 were examined by analytical electron microscopy and X-ray diffraction. Although the samples were silica and calcium rich, this was considered to be related to the chemical constituents in the groundwater rather than any migration of either bentonite or cement from the soft pile material.

12 General design recommendations

On the basis of the site study carried out at the A12 Hackney to M11 Link Road, the following general design recommendations can be made. These recommendations should be regarded as provisional until additional instrumented case history studies are available to further substantiate the findings:

i Ground investigation should include provision for measurement of the *in situ* distribution of pore water pressures and for *in situ* permeability testing in different soil strata.

ii In secant piling where alternative ‘hard’ reinforced concrete piles and ‘soft’ cement-bentonite piles are used, careful consideration needs to be given to pile diameters and spacings. Designs where the diameters of the soft piles are significantly greater than the gaps they are filling between adjoining hard piles are likely to provide better water retention.

iii Mix design for the soft piles needs to be such that strength and permeability requirements are met. In both cases, laboratory testing of trial mixes and site acceptance would normally be carried out on samples of minimum 100mm diameter. Samples of smaller diameter are likely to be less representative and lead to difficulties in compliance particularly with permeability requirements. The following requirements for the soft pile material have proved satisfactory for a wall retaining stiff impermeable clay:

  a minimum undrained shear strength of 150kPa at 28 days;

  b permeability not exceeding 5×10^{-8}m/s.

The long term durability of the soft piles also needs consideration although there is no evidence of migration of either the bentonite or cement from one case history study, further studies are needed.

iv For walls retaining impermeable clay, a visual examination of the exposed face of the soft pile should be continued over a period of about 6 months. If there is no evidence of water seepage, a concrete facing for the soft piles is probably not required. If there is evidence of water seepage, a facing should however be employed in the specific areas affected.

v For walls retaining non-homogeneous ground conditions, heavily fissured soils, or water bearing strata, it is generally advisable to employ a concrete facing to the exposed soft pile surface to improve water tightness. Control of high water tables by drainage measures may also need consideration.

vi Short term design water pressures would normally be evaluated from the measured *in situ* water pressures before construction: these would be hydrostatic from the water table in homogeneous soils. Evaluation of steady state seepage around and through the wall can be carried out by numerical analysis to determine longer term design water pressures. A sensitivity analysis can then be readily performed to assess the influence of different soft pile permeabilities and cut-off depths. Alternatively, where perched water tables do not exist, long term design pressures can be determined from simple linear seepage calculations assuming the head is dissipated uniformly along the flow path down the back of the wall to the hard pile toe and then up the front. This latter method although safe is likely to be conservative.

13 Acknowledgements

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Abstract

The secant bored pile wall at the A12 Hackney to M11 Link Road used alternate ‘hard’ reinforced concrete piles and ‘soft’ cement-bentonite piles. A sprayed concrete facing was employed over the facing of the soft piles because of concerns about their permeability. This report describes performance monitoring of the hard-soft piling system at the site. Piezometers were installed in the retained ground, the soft pile itself, and the ground beneath the new carriageway so that the flow of ground water could be clearly established in the vicinity of the wall. These measurements were supported by laboratory and *in situ* tests to establish the permeability of the retained ground and the soft piles. The results from the field monitoring were compared with finite element predictions of the pattern of water seepage in the vicinity of the wall.

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