Modelling Viscous Effects during and after Construction in London Clay

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ABSTRACT: A new approach to allow the modelling of the viscous behaviour of clay soils has recently been developed (Clarke & Hird, 2012) based on the BRICK constitutive model (Simpson, 1992). In this approach viscous effects, such as creep and stress relaxation, and the effects of strain history on soil stiffness are modelled within a single conceptual framework. The developed model, SRD (strain-rate dependent) BRICK, has been incorporated in a finite element program, allowing boundary value problems to be analysed. In this paper two case histories involving London Clay, where viscous effects possibly had an influence on the observed displacements, are back-analysed. These cases are the Jubilee Line extension at St James's Park (Standing *et al*., 1996, Nyren *et al*., 2001) and a deep basement at Horseferry Road (May, 1975, Chapman, 1999). The results of the numerical modelling show that, in each case, the SRD BRICK model is able to achieve closer agreement with the recorded displacements when compared with the un-modified BRICK model.

Key words: viscous effects, tunneling, excavations, creep, numerical modelling

1. INTRODUCTION

Finite element modelling is an important tool to aid the prediction of ground movements arising from construction work, especially in congested urban areas. The accuracy of the finite element output is determined by a number of factors, including the quality of information from the site investigation, assumptions made during the modelling, the validity of input parameters and critically, the ability of the underlying constitutive model to represent the actual behaviour of the ground. For clay soils, with their naturally complex behaviour, the latter represents a considerable challenge.

Numerous advanced constitutive models have been developed in recent years (e.g. Al-Tabbaa & Wood, 1989, Jardine, 1992, Simpson, 1992, Whittle, 1993, Bolton *et al.* 1994, Stallebrass & Taylor, 1997, Grammatikopoulou *et al*. 2008) which allow the highly non-linear and mainly inelastic deformation behaviour of over consolidated clays to be modelled. Amongst these, the BRICK model devised by Simpson (1992) is unique in being developed within strain space and does not rely on classical plasticity concepts such as a plastic potential or a flow rule. Attempts have also been made (e.g. Kutter & Sathialingam, 1992, Yin *et al*., 2002, Wheeler *et al*., 2003, Hinchberger & Rowe, 2005, Kelln *et al.*, 2009) to allow the modelling of viscous effects in soft clays, such as creep and stress relaxation, but in general these have not simultaneously allowed the same level of refinement of deformation properties as achieved in the advanced models mentioned above. However, an exception is a modification of the BRICK model proposed by Clarke & Hird (2012) and termed Strain Rate Dependent (SRD) BRICK. It was shown in Clarke & Hird (2012) that the SRD BRICK model is capable of simulating the correct soil response in a number of different laboratory tests, including step change in strain rate tests done under both one dimensional (Leroueil *et al*., 1985) and triaxial conditions (Graham *et al*., 1983, Sorensen *et al*., 2007) and also the influence of creep, resulting from recent stress history, on soil stiffness seen during advanced triaxial tests (Gasparre *et al*., 2007).

SRD BRICK has been incorporated in a finite element program (SAFE (Oasys, 2006)) and therefore the point has been reached where the model can also be tested by applying it to field problems. In this paper back-analyses of two case histories in London are reported, namely the Jubilee Line extension at St James's Park (Standing *et al.*, 1996, Nyren *et al*., 2001) and a deep basement at Horseferry Road (May, 1975, Chapman, 1999). In the case of the Jubilee Line extension, due to the rapid rate of advance of the tunnel heading, the strain rate in the surrounding soil was thought to be sufficiently high to generate immediate viscous effects. In contrast, the heave of the deep basement excavation at Horseferry Road took place over a very long period of time with creep contributing to the measured displacements. Before the analyses are described, the SRD BRICK model and the required input parameters will first be briefly outlined.

2. THE SRD BRICK MODEL

The SRD BRICK model, or its parent BRICK, can be regarded as a multiple kinematic yield surface model with the yield surfaces being defined in terms of strain rather than stress. With monotonic strain from an initially elastic state there is a progressive mobilisation of plasticity leading to the loss of stiffness shown schematically in Figure 1, approximated in stepwise fashion. Thus, the height of each step represents a proportion of the soil which is given its own identity, allowing its strain state and history to be tracked. If the overall strain, generally comprising both shear and volumetric components, reverses or alters, the model is able to adjust the soil stiffness in an appropriate manner, as originally shown by Simpson (1992). Strain history is tracked right back to the formation of the soil, including the effect of deposition and erosion of any overlying strata. Since it was first published, the BRICK model has been generalised to facilitate full 3D analyses by the inclusion of three extra components of shear strain (additional to the two shear components and one volumetric strain component required in the original model). Details of the 3D BRICK model may be found in Ellison *et al.* (2012).

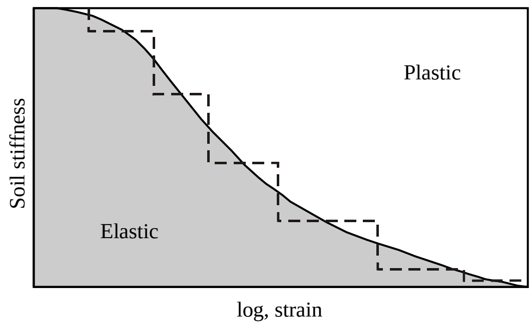

Figure 1 Stiffness degradation represented in stepwise fashion, after Simpson (1992)

In SRD BRICK viscous effects are modelled by effectively shifting the stepwise curve (Figure 1) to the right as the plastic strain rate increases. However, the curve does not simply translate. The plastic strain rate generally varies for each step (proportion of soil) and therefore the shift varies for each step. As this may bring the corresponding proportion of the soil back to an elastic state, iterative calculation of the plastic strain rate and its consequence is required. An increase in strain rate and the resulting increase in soil stiffness can be immediate. On the other hand, if the strain rate reduces, there is a limit to the rate at which the curve moves back to the left. Ultimately a lower bound, or reference, stiffness degradation curve is reached, linked to a reference strain rate. This approach permits the modelling of the effects of both the natural degradation of strain rate with time under constant stress (creep) and applied accelerations or decelerations of strain. Throughout the occurrence of strain, stress changes are calculated from the elastic strains only but there is provision for stress levels to increase due to consolidation involving full plasticity.

SRD BRICK requires the shape of the reference stiffness degradation curve to be defined for a given soil. Ideally, the curve would be based on experimental data from, for example, strain controlled triaxial tests with strain reversals, with allowance for the greater strain rate likely to be employed in tests. However, even for the extensively tested London Clay suitable data do not appear to exist. For modelling with BRICK the issue of strain rate does not arise and suitable parameters for London Clay were proposed by Simpson (1992). Later, Kanapathipillai (1996) proposed revised parameters based on back-analysis of the Heathrow Express trial tunnel and in this paper these have been adopted for analysis with BRICK. With SRD BRICK the reference curve has been derived by halving the strain coordinate for every step. Again, this was based on the work of Kanapathipillai (1996) who made an approximate calculation of the effect of creep by halving the strain coordinates. Equation 1 shows the strain rate-strain coordinate (SC) relationship which governs the variation in stiffness within the model.

$$SC = SC_{ref}\left[1+\beta\ln\left(\frac{|\dot{\varepsilon}|}{\dot{\varepsilon}_{ref}}+1\right)\right] \quad (1)$$

Where SC_{ref} are the default strain coordinates defined from stiffness degradation tests. Three additional parameters are required by SRD BRICK over the original BRICK model to describe viscous effects. These are: the reference strain rate ($\dot{\varepsilon}_{ref}$); a rate sensitivity coefficient (β), which governs the change of strain coordinates with strain rate; and a decay constant (m), which controls rates of creep and stress relaxation. In defining the reference strain rate, an upper time limit for the duration of creep, t_{max}, is also needed. Further details regarding model can be found in Clarke & Hird (2012).

Both the BRICK and SRD BRICK models require parameters, λ^* and κ^*, to define the slopes respectively of the compression and swelling lines when plotted with axes of volumetric strain and the logarithm of mean effective normal stress. Further parameters include, ι, which defines the elastic stiffness along with β_G and $\beta\varphi$ to control respectively the variation of stiffness and strength with over consolidation. A value of Poisson's ratio, ν, must also be specified.

3. BACK-ANALYSES

3.1 Jubilee Line Extension at St James's Park, London

For the Jubilee Line extension, new lines had to be constructed under sensitive existing buildings, justifying the use of a wide range of monitoring techniques. This not only enabled the monitoring of building settlements for safety reasons but was also intended to provide high quality data for research purposes through numerical modelling. One such data set was obtained during the construction of the twin running tunnels beneath St James's Park, a green field site where the surface displacements were carefully monitored (Nyren *et al.*, 2001). From previous tunnel constructions a supposedly conservative estimate of 2% was adopted for the design volume loss (Standing & Burland, 2006). However, while north of the lake in the Park the volume loss was below 2% as expected, south of the lake it was significantly higher, up to 3.3% (Nyren *et al.*, 2001). Volume loss, used to quantify tunnelling efficiency, is defined as the volume of the settlement trough measured at the surface divided by the volume of the tunnel (Mair, 2008):

$$V_l = \frac{100\,V_s}{\pi D^2/4} \quad (2)$$

where V_l is the volume loss (%), V_s is the volume of the transverse settlement trough per unit length of tunnel and D is the diameter of the tunnel.

The soil profile, shown in Figure 2, consists of four distinct beds, the uppermost of which is a 4.5 m thick layer of Sand, overlying a 2.7 m thick bed of Terrace Gravel. Underneath lies a 34 m thick layer of London Clay, overlying the very stiff Lambeth Group beds. The tunnels have an external diameter of 4.75m and a 200mm thick expanded precast concrete segment lining (Dimmock & Mair, 2007). As the tunnel lining has not been explicitly modelled the diameter of the tunnel was taken to be that of the tunnelling shield which was used during the construction, 4.85m. It was noted by Nyren *et al.* (2001), that one of the reasons for the high volume loss experienced south of the lake could have been the rapid rate of advance of the tunnels, reaching about 45m/day. The entire volume loss is assumed to be due to the extraction of material at the tunnel boundary.

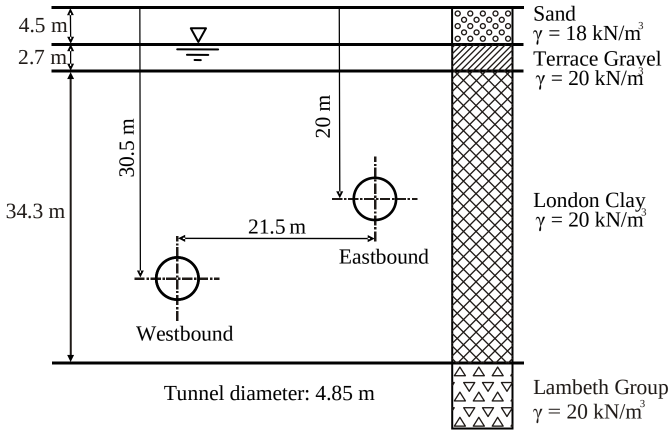

Figure 2 Soil profile and tunnel geometry at St James's Park, after Addenbrooke *et al*. (1997) and Mair & Dimmock (2007)

Previous studies have identified a number of possible factors contributing to the magnitude of settlement observed, including anisotropy (Addenbrooke *et al*., 1997) and the effects of historic creep on the soil stiffness (Grammatikopoulou *et al*., 2008). The work presented here aims to determine whether strain rate effects during construction could also have had an influence on the shape of the surface settlement trough.

Three analyses of the westbound running tunnel, which was the first to be constructed, will be presented: an initial analysis using a linear elastic / perfectly plastic model with a Mohr-Coulomb criterion governing plasticity (referred to simply as the Mohr-Coulomb model) for all strata, a second analysis changing the model to BRICK for the London Clay to demonstrate the benefit of employing a more advanced model and a final analysis using SRD BRICK for the London Clay to demonstrate the further benefit of including viscous modelling. As the behaviour of London Clay is well documented, the use of a complex soil model can be justified. The intention of this study is to vary the parameters for the London Clay layer while keeping the parameters for the other layers constant to isolate the influence of the modelled viscous effects.

3.1.1 Model Parameters

The parameters for the Sand, Terrace Gravel and Lambeth Group, as used by Addenbrooke *et al*. (1997), are given in Table 1. As the Lambeth Group was deemed to have a minimal effect on the surface displacements, a large cohesion of 200kPa was assumed for that layer. The Sand was assumed to have a dilation angle of 0° to represent the loose nature of the soil. All the layers in Table 1 were assumed to be fully drained for the purpose of this study. The pore water pressures were taken to be hydrostatic from the top of the Terrace Gravel, as indicated in Figure 2.

Table 1 Mohr Coulomb model parameters for strata other than London Clay at St James's Park, after Addenbrooke *et al*. (1997)

Strata	**Sand**	**Terrace Gravel**	**Lambeth Group**
Strength parameters	$c' = 0$ kPa $\varphi' = 35.0°$	$c' = 0$ kPa $\varphi' = 35.0°$	$c' = 200$ kPa $\varphi' = 27.0°$
Angle of dilation	0 °	17.5 °	13.5 °
Bulk unit weight (kN/m^3)	$\gamma_{dry} = 18$ $\gamma_{sat} = 20$	$\gamma_{sat} = 20$	$\gamma_{sat} = 20$
Young's modulus, E' (kPa)	5000	6000z	6000z
Poisson's ratio, ν	0.3	0.2	0.2
Earth pressure coefficient, K_0	0.5	0.5	1.5

Note: z is the distance below the ground surface in metres

The finite element mesh used for the analysis is shown in Figure 3. Although provision was made for eventual modelling the eastbound tunnel, this has not yet been attempted. In Addenbrooke *et al*. (1997) and Grammatikopoulou *et al*. (2008) the volume loss due to tunnelling was modelled using the 'volume loss control method' (Potts & Zdravkovic, 2001). In the present analyses another method for introducing volume loss has been employed. This method involved redefining the soil within the tunnel boundary to act as a linear elastic drained material with reduced stiffness and applying a negative pore water pressure to this material to force a volume reduction. It should be noted that the current study is only concerned with the immediate volume loss during the excavation, with further volume loss due to installation of the lining (Dimmock & Mair, 2007) and long term drainage (Wongsaroj *et al*., 2007) being ignored. During the introduction of the volume loss, the London Clay beyond the tunnel boundary was assumed to remain undrained. It was also assumed that volume changes in the drained layers above the London Clay would be negligible and this was confirmed by checking the volumetric strains.

3.1.2 Analyses

In the first analysis (the "Mohr-Coulomb" analysis), the problem was initialised with the parameters from Table 1 along with the drained London Clay parameters from Table 2, after which the displacements were reset to zero.

In the second analysis (the "BRICK" analysis), it was necessary, using BRICK, to account for the effect of the entire stress history of the London Clay. It was assumed that the original thickness of the London Clay layer was 200m (King, 1981).

As the remaining London Clay layer is 34m thick, application and removal of 166m of overburden was simulated, followed by the surcharge of the recent overlying deposits.

Table 2 Mohr Coulomb model parameters for London Clay at St James's Park

Strata	**London Clay**	
	Drained	**Undrained**
Strength parameters	$c' = 0$ kPa $\varphi' = 25.0°$	$c = 150$ kPa
Angle of dilation	12.5 °	-
Bulk unit weight (kN/m^3)	$\gamma_{sat} = 20$	$\gamma_{sat} = 20$
Young's modulus, E' (kPa)	6000z	6000z
Poisson's ratio, ν	0.2	0.498
Earth pressure coefficient, K_0	1.5	1.5

Note: z is the distance below the ground surface in metres

As already mentioned, the stiffness curve parameters presented in Kanapathipillai (1996) were employed. These are shown in Table 3, while Table 4 lists the remaining BRICK parameters. The predicted profile of the in-situ lateral earth pressure coefficient, K_0, can be seen in Figure 4.

The third analysis (the "SRD BRICK" analysis) simulated the same stress history as the BRICK analysis but used the modified set of stiffness curve parameters (strains associated with each step were halved to give the reference values). Because of the very long timescale of the geological stress history, viscous effects were disabled and the modified stiffness curve, taken to be the reference curve (applicable for extremely small strain rates), was assumed to apply continuously. The predicted K_0 profile was identical to that from the second analysis (Figure 4). Subsequently, viscous modelling was restored with an upper time limit for creep, t_{max}, of 10^9 seconds, a suitably large though otherwise arbitrary value. Based on test data for London Clay from Bishop (1966), the strain rate at this time would be 1×10^{-13} s^{-1} which was taken to equal the reference strain rate, $\dot{\varepsilon}_{ref}$. From the same data a value of m = 1.039 can be found, while Clarke & Hird (2012) showed that the value of β for London Clay was 0.23 based on the work of Sorensen *et al*. (2007). As the SRD BRICK model is rate dependent, the duration of the volume loss had to be specified. From Dimmock & Mair (2007) it is known that the volume loss was experienced during a tunnel face advance of 32m. With the rate of advance of the tunnel boring machine taken to be 45m/day, this equates to a volume loss duration of 0.71 days.

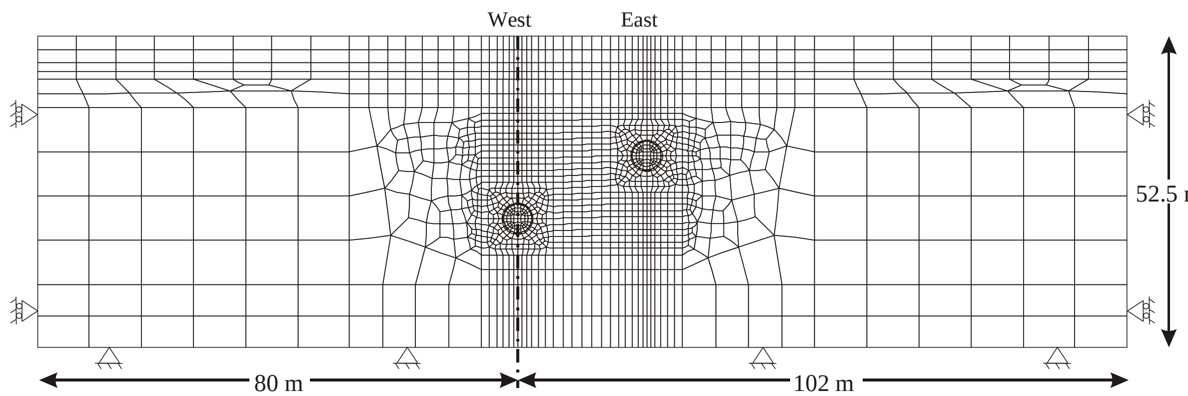

Figure 3 Finite element mesh, St James's Park twin tunnels

Table 3 BRICK stiffness curve parameters for London Clay, Kanapathipillai (1996)

Step	Strain	Soil proportion below step	Soil proportion represented by step
1	3.040×10^{-5}	0.92	0.08
2	6.086×10^{-5}	0.75	0.17
3	1.014×10^{-4}	0.53	0.22
4	1.211×10^{-4}	0.29	0.24
5	8.200×10^{-4}	0.13	0.16
6	0.00171	0.075	0.055
7	0.00352	0.044	0.031
8	0.00969	0.017	0.027
9	0.0222	0.0035	0.0135
10	0.0646	0	0.0035

Table 4 BRICK parameters for London Clay

λ^*	κ^*	ι	ν	β_G	β_φ
0.1	0.02	0.0019	0.2	4	3

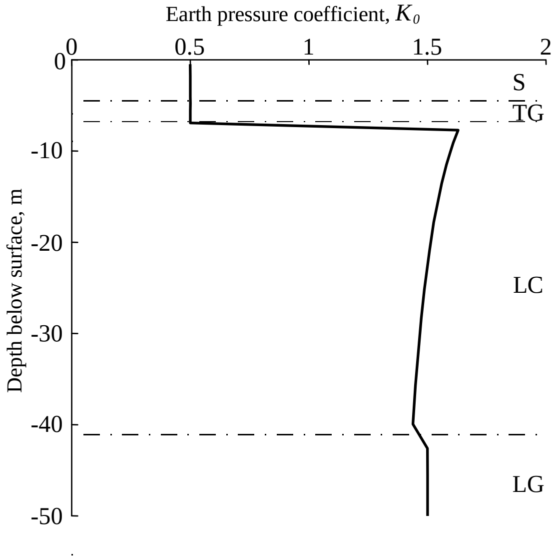

Figure 4 BRICK generated K_0 profile based on stress history

3.1.3 Results

In all three analyses the same volume loss of 3.3% was induced. Comparisons of the resulting contours of vertical and horizontal surface displacements are shown in Figure 5. It can be seen that the increasing complexity of the London Clay model, from Mohr-Coulomb through BRICK to SRD BRICK, leads to a progressive deepening and narrowing of the vertical settlement trough above the tunnel axis (Fig. 5a). This is accompanied by a progressive increase in horizontal displacement at the surface around the steepest part of the settlement trough (Fig. 5b). The improvement of the BRICK predictions over those of the Mohr-Coulomb model can be attributed, in part, to changes of initial stiffness with depth, generated by modelling the stress history. The SRD BRICK model was able to generate even larger beneficial stiffness changes due to the inclusion of rate effects. However, while SRD BRICK provided the best match with field settlement data, the settlement was still somewhat under-predicted for both horizontal and vertical displacements and the lateral extent of the settlements were over-predicted in the far-field. It must also be noted that the analyses presented here are a 2D representation of a much more complex 3D problem. This approach amalgamates the individual factors (face excavation, soil deformation etc.) into a single volume loss. The results therefore should be seen as an indication of the influence of viscous effects in 2D only.

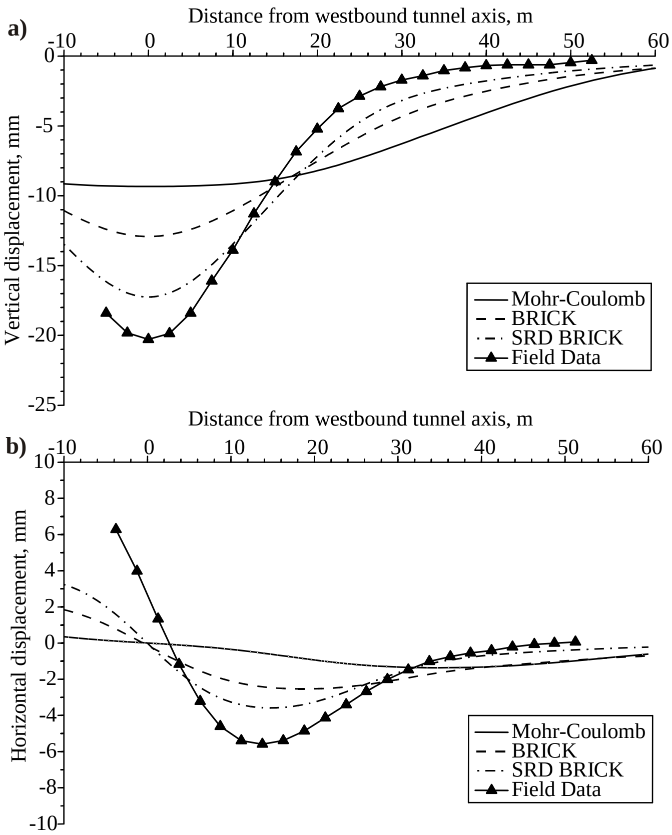

Figure 5 Comparison of predicted settlement troughs for a 3.3% volume loss with field data from Nyren *et al*. (2001)

3.2 Excavation at Horseferry Road, London

At Horseferry Road a deep basement was constructed in London Clay and the heave of the basement was measured for an exceptionally long period of 21 years (May, 1975, Chapman, 1999). Back-analysis of the excavation was undertaken to determine the relative influence of primary and secondary swelling.

Excavation for the basement began in June 1966 and was completed in November 1967. The basement was completed to ground level in May 1968 but due to unforeseen circumstances, the superstructure was never completed and the site lay derelict (but still monitored) until June 1989, when it was redeveloped. A ground investigation to support the redevelopment was carried out in February 1989, which showed a scour hollow in the London Clay to lie partially beneath the site (which had been missed in the original investigation). The inferred extent of the scour hollow, in-filled with Terrace Gravel, can be seen on a cross-section in Figure 6. This section also shows that above the Terrace Gravel a layer of Made Ground about 3m thick and, in places, a layer of Alluvium about 2m thick were present. Initial pore water pressures were assumed to be hydrostatic from a level near the top of the Terrace Gravel (-2.5m).

3.2.1 Model Parameters

In plan the excavation had an irregular but roughly square shape and measured at least 70m in each direction. Figure 7 shows the finite element mesh which was based closely on the cross-section of Figure 6. Plane strain conditions were assumed to apply. As shown in the modelling of the Jubilee Line extension, the Mohr-Coulomb model is unable to predict the behaviour of London Clay accurately and it was therefore not used to model the London Clay layer at Horseferry Road.

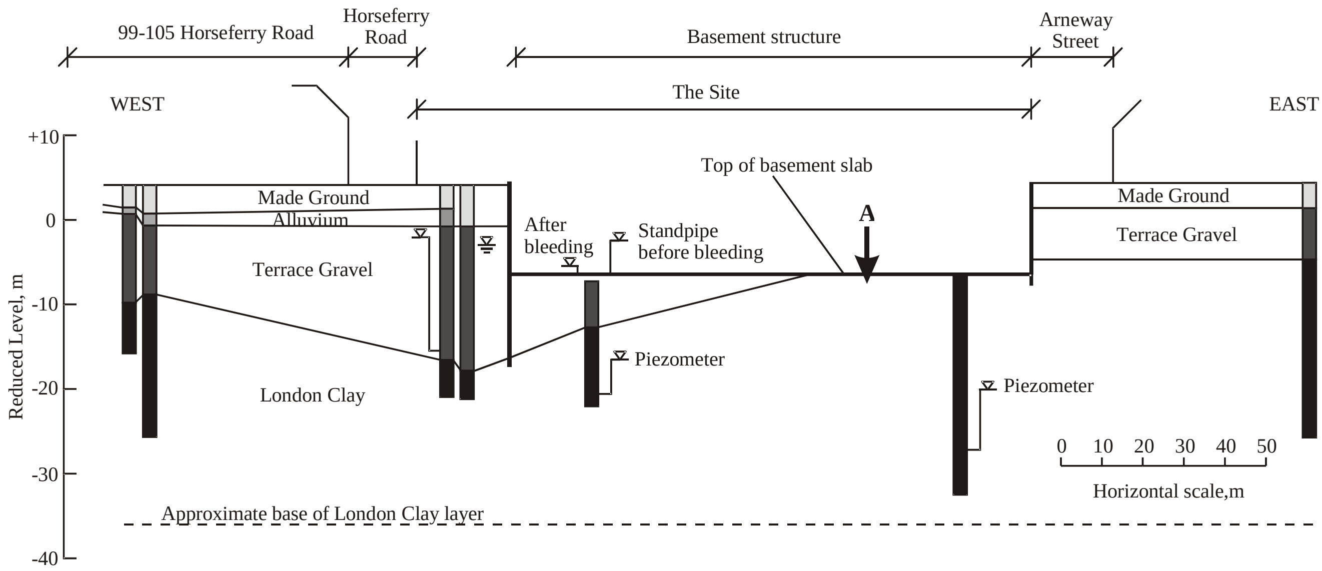

Figure 6 Site section showing relevant borehole information, after Chapman (1999)

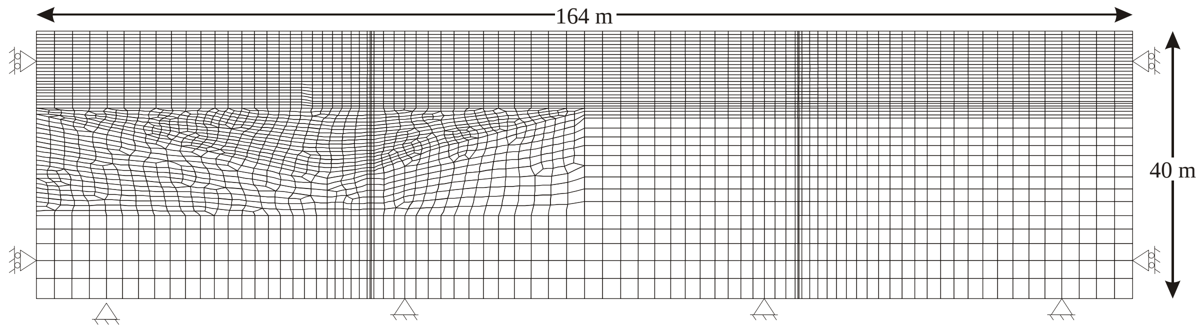

Figure 7 Finite element mesh, Horseferry Road

Mohr-Coulomb model parameters for layers other than the London Clay are given in Table 5. By comparison with Table 1, it can be seen that some changes were made to the values adopted for the Jubilee Line extension analysis. The elastic modulus and dilation angle of the Terrace Gravel were reduced because it was considered likely that the relative density of this soil in the potentially important region of the scour hollow would be lower. The parameters for the Alluvium and Made Ground were based on judgement of the borehole records.

Table 5 Mohr Coulomb model parameters for strata other than London Clay at Horseferry Road

Strata	**Made Ground**	**Alluvium**	**Terrace Gravel**
Strength parameters	c' = 0 kPa φ' = 25.0 °	c' = 0 kPa φ' = 25.0 °	c' = 200 kPa φ' = 38.0 °
Angle of dilation	0 °	0 °	0 °
Bulk unit weight (kN/m^3)	γ_{dry} = 18	γ_{sat} = 20	γ_{dry} = 20 γ_{sat} = 20
Young's modulus, E' (kPa)	1500	4500*z*	4500*z*
Poisson's ratio, ν	0.2	0.2	0.2
Earth pressure coefficient, K_0	0.561	0.561	0.384

Note: *z* is the distance below the ground surface in metres

The London Clay layer was modelled using BRICK or SRD BRICK with the parameters given in Tables 3 & 4. The permeability of the soil was modelled using Equation 3 (Potts & Zdravkovic 1999).

$$k = k_0 e^{(-ap')} \tag{3}$$

where *k* is the predicted permeability of the soil, k_0 is the minimum soil permeability, *a* is a material constant and *p′* is the mean normal effective stress. By analysis of the permeability values used in Addenbrooke *et al*. (1997), the parameter *a* was calculated to be 0.0104 for a k_0 value of $1e^{-8}$ m/s.

The concrete diaphragm walls and basement slab were modelled as linear elastic materials with a Young's modulus of 16000kN/m² and a Poisson's ratio of 0.2. Temporary props were added at the top of the diaphragm walls after the first excavation stage. The concrete was modelled as a no-flow drainage boundary. The load applied to the soil by the finished basement construction was 48.2kN/m² as stated in May (1975).

3.2.2 Analyses

The construction of the deep basement at Horseferry Road lasted for 17 months and could not be assumed to take place under undrained conditions. Thus, for the London Clay layer a coupled consolidation analysis was performed for the construction phase, as well as the subsequent monitoring period. The overlying soil layers were taken to be drained throughout the analysis. The various stages of the analysis are given in Table 6. During the initialisation stage the stress history of the London Clay was simulated in the same manner as that described for the site at St James's Park but with an assumed removal of 168m of overburden following deposition.

Table 6 Horseferry Road finite element analysis stages

	Stage description	Duration (days)	Steps	Cumulative time (days)
	Initialisation	0	1	0
Construction	Installation of basement walls	120	4	120
	Excavation stage 1, +0.5m	120	4	240
	Installation of temporary props			
	Excavation stage 2, -3.0m	120	4	360
	Excavation stage 3, -5.0m	60	3	420
	Excavation stage 4, -6.5m	60	3	480
	Cast base slab	30	3	510
Monitoring	March 1968 heave	180	6	690
	March 1969 heave	360	6	1050
	March 1973 heave	1460	10	2510
	March 1980 heave	2555	7	5065
	March 1990 heave	3650	10	8715

3.2.3 Results

As the basement was excavated the vertical stress on the underlying soil decreased and the London Clay layer began to heave. Both BRICK and SRD BRICK predicted the development of a region of negative pore pressure below the base of the excavation. The maximum predicted changes of pore pressure with (the square root of) time, at Point A (Figure 6), are shown in Figure 8. Unfortunately, no measurements of pore pressure were made so the accuracy of these predictions is unknown. Once the basement slab was completed and the load of the basement substructure was applied, the (negative) excess pore pressures dissipated as the soil continued to heave. The BRICK model predicted a rapid increase in pore pressure over the first 6 months of the monitoring period. The rate of increase then decayed until pore pressures reached preconstruction levels after about 100 months. The SRD BRICK model predicted a similarly sharp increase in pressure during the first 6 months, but the following approach to preconstruction levels was noticeably slower due to viscous effects, primarily creep. The viscous, or secondary, swelling was sufficient to generate small excess pore pressures and this is why the predicted excess pore pressures did not dissipate completely during the period of interest.

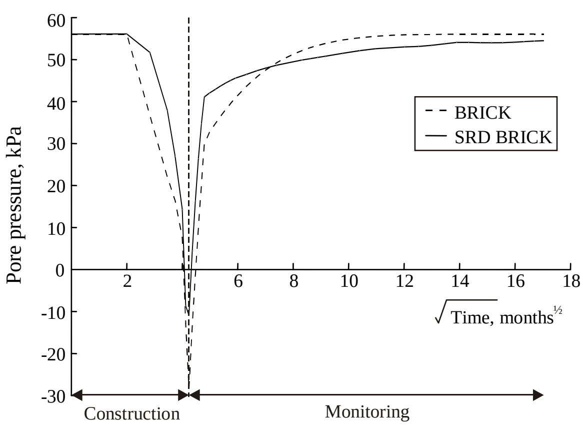

Figure 8 Change in pore pressure with time at Point A (Fig. 6)

Figure 9 shows a comparison of the contours of vertical displacement generated during the 21 year post-construction period as predicted by BRICK and SRD BRICK. Both models predicted the same location of the maximum displacement, which logically coincided with the location of the maximum excess pore water pressure (Point A). While the displacement patterns are similar, the SRD BRICK model predicted more heave and a larger gradient of heave across the basement slab.

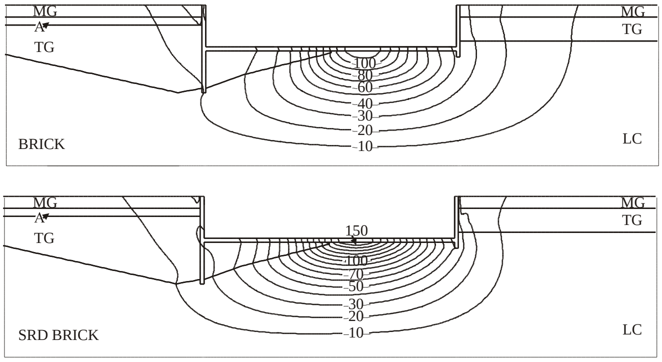

Figure 9 Predicted contours of resultant displacement for March 1990

In Figure 10 the field data are compared with the predicted heave at Point A which was close to the location of the field measurements. The monitoring of heave started in March 1968, 6 months after the basement slab was completed. Thus, to enable a comparison between the finite element predictions and the field data, the predicted displacements were reset to zero after 6 months. In the BRICK analysis the heave levels off after about 100 months, when the excess pore pressures were negligible (Figure 8), and the overall result is rather poor. With SRD BRICK the heave continues for longer, due to the inclusion of viscous behaviour, and there is much better agreement with the field data.

In the case of the BRICK analysis, the match to the field data could probably be improved by reducing the assumed permeability of the London Clay. However, the required reduction, by a factor in an estimated range of 5 to 8, would not be supported by the available data (Addenbrooke *et al.*, 1997).

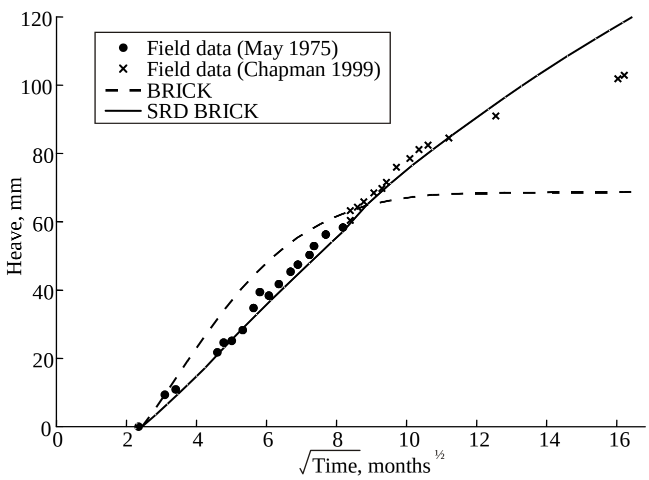

Figure 10 Comparison of heave predictions with field measurements

4. CONCLUSIONS

An advanced model for simulating the behaviour of clay soils, incorporating viscous behaviour, has been used in finite element back-analyses of two case histories, each involving London Clay as the principal stratum.

The analyses conducted were not designed to take into account all the construction phenomena. Instead the analyses concentrated on increasing the complexity of the constitutive model for only the London Clay layer to assess the possible benefits of including viscous effects, while simplifying the numerical modelling.

The purpose of this was to use a single constitutive model to model both short-term high-rate movements due to tunnelling and long-term slow-rate movements due to basement construction. In both cases the predictions of ground surface displacements were improved by incorporating viscous effects.

The use of the SRD BRICK model requires good quality data on stiffness degradation and viscous effects on which to base the input parameters. Unfortunately the extensive research on London Clay will not generally be matched for other deposits. However, for major projects in relatively uniform and extensive strata suitable testing could be justified. Potentially, the model could be valuable for projects involving clays which are less over consolidated than London Clay, where viscous effects are likely to be more pronounced. The exploration of this potential, perhaps initially through further back-analysis of case histories, would be welcomed.

5. ACKNOWLEDGEMENTS

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