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Case Histories Back Analyses for the Application of the Observational Method under Eurocodes for the Scout Project

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ABSTRACT

A European Commission (EC) funded *Sustainable Construction of Underground Infrastructure* (SCOUT) project introduces a breakthrough for the construction of "cut-and-cover" tunnel using a horizontal diaphragm walling equipment with the implementation of the Observational Method for the construction of the underground structure. Design optimisation options such as the use of different structural forms, new construction material and even the design approaches were explored in an attempt to provide a sustainable design for the underground structure.

This paper will briefly describe the project background, design optimisation approach and present back analyses undertaken of retaining wall case histories to develop a methodology to derive design parameters appropriate for the implementation of the Observation Method under the framework of Eurocodes.

INTRODUCTION

The development of the Trans-European Transport Network requires the construction of many new railways or highways or waterborne connections. Underground transport infrastructures are in many cases the best option in urban environment to avoid congestion at the surface and minimise environmental impact during and post construction, and in many projects the only possible option to build intermodal connections such as links between underground stations and airports, parking lots, pedestrian access etc.

A three year EC funded SCOUT project was initiated in early 2005 to introduce a breakthrough in the technique of constructing "cut-and-cover" tunnel, design optimisation and the implementation of the Observational Method in an attempt to provide a more sustainable approach to the design and construction of these structure. This paper forms part of the dissemination activity of the this project.

EUROCODE IN RETAINING WALL DESIGN

Under Eurocode 7 (EC7) design consideration, it is required that the design be *verified that no relevant limit state, as defined in EN1990:2002, is exceeded (Clause 2.1 (1)P).* The limit states under consideration are the **ultimate** (ULS) or **serviceability** (SLS) limit states.

The concept of partial factors has been used in the design. These partial factors are applied to the actions or effects of actions (A), soil parameters (M) or the resistance (R). The values of the partial factors differ dependent on the three Design Approaches (DA) used and these are summarised below.

Design Approach 1 (DA1)

For DA 1, the following combinations of sets of partial factors are to be considered and the worst of the two combinations will form the ULS design of the retaining structure.

Combination 1: A1 + M1 + R1
Combination 2: A2 + M2 + R1

Details of the partial factors used are listed in Table 1.

Table 1: Partial factors for Design Approach 1 (DA1)

DA1 Comb	Actions or effects of actions, **A**				Soil para, **M**			Res^t, **R**
	Permanent		Variable					
	Unf	Fav	Unf	Fav	tanϕ'	c'	c_u	
1	1.35	1	1.5	0	1	1	1	1
2	1	1	1.3	0	1.25	1.25	1.4	1

Note: Comb – Combination, Res^t – Resistance, Para – Parameters, Unf – Unfavourable and Fav – Favourable

In the above table, a permanent action is defined as action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value.

A variable action is the action for which the variation in magnitude with time is neither negligible nor monotonic.

For retaining wall design using DA1 Combination 1, no partial factor is applied to the soil parameters. A partial factor of 1.5 is applied to the unfavourable variable load and no factor is considered for the favourable variable load. The effect of loading, i.e. resulting forces in the retaining structure, is then multiply with a factor of 1.35 for ULS design.

For DA1 Combination 2, partial factors are applied to the soil parameters and unfavourable variable load only. The effect of loading, resulting forces in the retaining structure, is treated as the forces for ULS design without any further partial factor applied to it.

There is no other design requirement to manipulate the resistance calculated from the design.

Design Approach 2 (DA2)

For DA2, the partial factors for actions and the effects of actions are the same as for Combination 1 of DA1 but these are combined with a partial factor of 1.4 for the resistance (R) in the ULS design of the retaining structure or:

Combination: A1 + M1 + R2 *(R2=1.4 for embedded retaining structure)*

This is summarised in Table 2.

Table 2: Partial factors for Design Approach 2 (DA2)

DA2	Actions or effects of actions, **A**				Soil para, **M**			Res[t], **R**
	Permanent		Variable		$\tan\phi'$	c'	c_u	
	Unf	Fav	Unf	Fav				
-	1.35	1	1.5	0	1	1	1	1.4

See notes above for DA1

For retaining wall design, DA2 is similar to DA1 Combination 1 with the exception that a further check is needed by ensuring the resistance has an adequate partial factor of safety of at least 1.4.

Design Approach 3 (DA3)

DA3 uses the following combination of sets of partial factors for the ULS design:

Combination: (A1* or A2#) + M2 + R3 (R3=1.0 for embedded retaining structure)

* on structural actions

\# on geotechnical actions

In effect, this approach requires two sets of design assessments for the requirements on the actions or the effects of actions. One of these sets of design assessments is the Combination 2 of the DA1. This is summarised in Table 3.

DA3 requires partial factors be applied to the soil strength and as well as the effects of the action. Such design approach is therefore not likely to produce a sustainable design where it allows significantly more conservatism in the design compared with the first two design approaches.

Table 3: Partial factors for Design Approach 3 (DA3)

DA3	Actions or effects of actions, **A**				Soil para, **M**			Res[t], **R**
	Permanent		Variable		$\tan\phi'$	c'	c_u	
	Unf	Fav	Unf	Fav				
Struct	1.35	1	1.5	0	1.25	1.25	1.4	1
Geo	1	1	1.3	0	1.25	1.25	1.4	1

See note above for DA1, Struct – Structural Actions and Geo – Geotechnical Actions

DESIGN OPTIMISATION

Optimisation is the main drive of the SCOUT project, which was to achieve sustainable construction through more efficient design hence more effective use of construction resources.

The following aspects of design were considered in the project, some were based on qualitative assessment.

- the analytical model;
- existing crack width requirements, especially in respect to their influence on cost, water ingress and the corrosion of both bar and steel fibre reinforcement;
- the benefits and disadvantages of including non-metallic fibres, water-stops, crack-inducers and drains;
- structural form which allows bending moment continuity between walls and slabs, provision of haunches to aid arch action in slabs, beneficial effects of axial load in the slabs and horizontal bending moment continuity.

Analytical model

The use of different design approaches was found to influence the computed structural forces. For some soils there was a reduction in the computed maximum bending moment when a more complex FE design approach was used compared to a beam spring approach with simplified design assumptions.

The effective stiffness of a reinforced concrete wall is approximately constant until it cracks, at which point there is a significant reduction in stiffness. Further increases in moment lead to further, smaller stiffness reductions. The relationship is very dependent on the wall thickness, reinforcement area and effective elastic modulus, which reduces with creep over time, but typical relationships are shown in Figure 1.

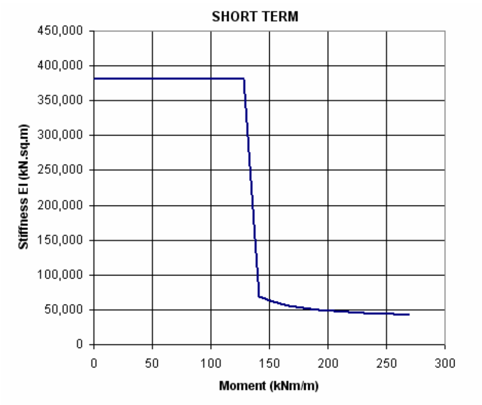


Fig. 1 Variation of bending stiffness with bending moment

Significant reductions in the computed bending moments were obtained using a variable bending stiffness wall. Such an approach may require an iterative process to ensure that the bending stiffness and moment are consistent.

Crack width consideration

Although the quantity of reinforcement in major infrastructure projects is often governed by crack width considerations, the need to control crack widths for durability purposes is controversial. Therefore, careful consideration of crack width allowance in the design of cut-and-cover structure could lead to potential design optimisation.

Durability. The durability resistance is determined by cover and the mix design. Even though Eurocode 2 (EC2) Table 7.1N and its notes link crack width with durability, many experts now believe that crack width control is not a measure of resistance to corrosion. The key issue is the presence of a crack, not its width. It should be noted that cracks of any size which are in line with the reinforcement can cause corrosion, whereas cracks transverse to the reinforcement are unlikely to. Research shows that crack widths up to 0.5mm are likely to be satisfactory for durability and the underlying research considers that cracks up to 1mm width are unlikely to cause problems. *(Beeby 1978, 1983; Schiessl and Raupach 1997, Schiessl 1988).*
For the SCOUT project, crack widths of up to 0.5mm have been considered satisfactory for durability.

Watertightness. Any size crack that passes through the section may let in water. However flexure, provided it is not combined with too much tension, will normally cause a compression zone that will prevent water passage. The rate of leakage depends on, amongst other things, the size of crack. EC2 gives guidance on how to keep the crack size small, which is, of course, important for structures below the water table. High leakage is often the result of bad workmanship, leading to grout leakage and honeycombing of the concrete and misplaced water bars.

It is essential that the client understands and designer agree on the strategy for watertightness and how to split the budget between attempting to prevent leaks in the first place and the long term maintenance of the structure.

Aesthetics. Smaller crack widths are usually less obtrusive. However there are situations where leaks occur with smaller crack sizes where the salts and deposits leach through and cause very unsightly marking. So it is unlikely that a specification of crack size alone will satisfy the client with this concern.

Crack width formulae in EC2. The calculation of crack widths to EC2 is given in section 7.3.4. There are uncertainties with using expression 7.9 for long-term loads. Following discussion with members of the code committee, it has been assumed that:

- the calculation of σ_s for long term loads should allow for creep, that is, in the cracked section calculation, the concrete modulus should be taken as $E_{cm}/(1+\varphi)$, where φ is the creep coefficient;
- the expression for α_e should be remain E_s/E_{cm} for long term loads.

Expression 7.11 in EC2 gives large crack spacings and hence crack widths for big covers. For structures in contact with soil, such as diaphragm walls, a cover of 75mm is usually specified, even though the cover required for durability maybe as low as half this figure. It has been suggested that since the increase to 75 mm is not to do directly with durability but to allow for deviations in the cover due to unevenness in the soil, it could be argued that it would be reasonable to calculate the crack width using the cover given in Table 4.4N and that any additional cover will result in an improvement in durability above that required by the code. This assumes that leakage and appearance are not critical.

Design for crack width in accordance with EC2. Design calculations in accordance with EC2 were made to compare the difference between reinforced concrete design with and without crack width control. Variations of the amount of reinforcement with the size of crack width are shown in Figure 2. From this and similar comparisons, it was clear that significant savings could be made in the design if careful thought and consideration is given to the allowance of crack width in the design. Such savings could be achieved without compromising the durability of the reinforced structure, which instead would be governed by watertightness and aesthetic considerations.

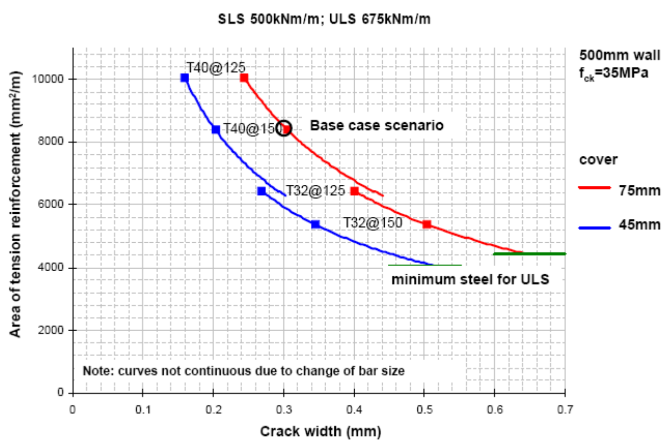
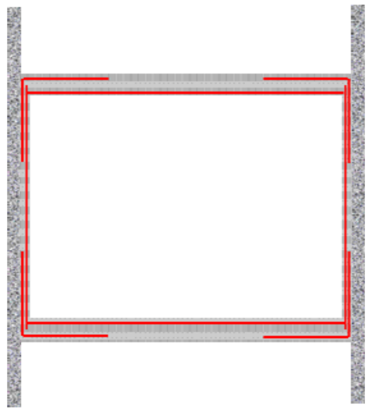


Fig. 2. Variations of quantity of reinforcement with size of crack width

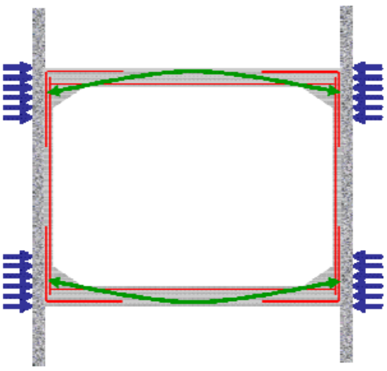
Structural form

Since the floor and roof slabs act as props, these elements must be designed for both axial force and bending moment; both upper and lower bound estimates of the propping force should be considered. If space permits haunches, consideration should be given to forming the permanent structure so that arch action can develop. This is a more efficient way to carry transverse loads and can reduce wall spans. The thrust from arch action will also tend to increase the soil loads acting around the prop height, reducing the span moments. These are illustrated in Figure 3.

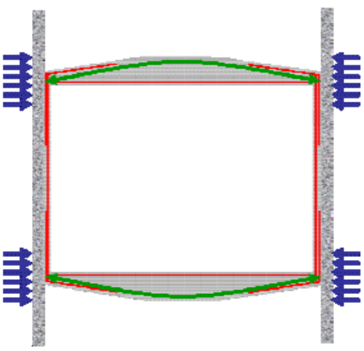


Conventional double wall structure

Lateral loads onto walls and slabs can only be taken by bending moments. Moment continuity at corners results in high quantities of reinforcement and congestion.



Arching reduces required reinforcement Providing haunches at the connection between roof and wall would allow beneficial arching effects to reduce reinforcement needed in these highly stressed areas. The solution with haunches seems better than profiled slab below (but haunches may not be acceptable).



Arching reduces required reinforcement Providing shaped/profiled slabs produces similar effects to that of haunches at the connection between roof and wall; this would allow beneficial arching effects to reduce reinforcement needed in these highly stressed areas.

Fig. 3. Performance of difference structural forms

DESIGN PARAMETERS FOR OM APPROACH

In a conventional design **characteristic** geotechnical parameters are used. In Eurocode this is defined as "cautious estimate of the value affecting the occurrence of the limit state" *(Cl 2.4.5.2 (2))*. "If statistical methods are used, the characteristic value should be derived such that the calculated probability of a **worse value governing the occurrence of the limit state** under consideration is not greater than 5%." *(Cl 2.4.5.2 (11))*.

The OM design approach differs from a conventional design where **most probable** and **characteristic** geotechnical design parameters are used to establish the range of behaviour of the structure. **Most probable** design produces a design close to the likely performance of the geotechnical structure while **characteristic** design generally has some built-in safety margin over and above the normal factors of safety allowed in geotechnical design. In essence, the OM approach allows the use of stringent construction control to tap into the potential benefits between **most probable** and **characteristic** designs.

The OM approach is entirely different to the conventional design approach, which relies on a generally conservative design with monitoring, if at all carried out, playing a very much **passive** role to check original predictions are still valid. In OM the monitoring of instruments plays an **active** role in both design and construction, allowing planned modifications to be carried out within an agreed contractual framework that involves all main Parties.

Figure 4 shows the application of the OM after allowing modification in the design when this design approach is chosen before the initiation of the construction.

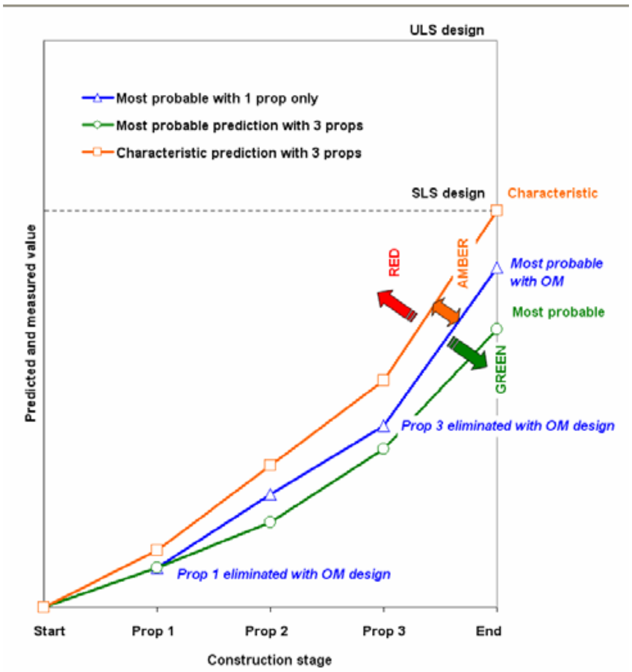


Fig. 4. Illustration of multi-stage construction measured and design values

In this figure, design is made with **characteristic** parameters with the original construction sequence (**RED** line). The behaviour of the structure under the same sequence using the

most probable parameters is shown for completeness (**GREEN** line), but this has no relevance in the context of the design using OM. Instead, a proposed sequence with a reduced number of prop is designed under the OM approach using the **most probable** parameters (**BLUE** line). Trigger values are set based on these predictions. The AMBER level was set at the **most probable** design line so to allow sufficient time to introduce contingency if the measured behaviour exceeds this limit.

BACK ANALYSIS OF CASE HISTORIES

Back analysis of case histories approach is used to derive the **most probable** geotechnical parameters for the design using the OM approach. In this paper case histories of excavation in stiff over-consolidated London Clay were back analysed to derive the **most probable** parameters.

In order to be reliable, the back analysis model must be developed in a systematic way. Key features of a reliable back analysis model are:

- It models all stages of the construction to date and matches the observed and measured behaviour of the structure at each and every significant construction stage;
- It models the construction sequence and geometry of the 'as-built' works (i.e. in the case of a multi-staged excavation, the actual excavation level of each stage is modelled rather than the planned excavation level);
- The soil conditions and material properties determined from the model are realistic and compatible (for example, the strength and stiffness of a given soil type determined from the model lie within the range of foreseeable parameters from the site investigation data and are compatible with empirical correlations for that soil type).

As previously indicated, it is important that the range of performance of retaining structure is established during the design stage so that a carefully controlled construction sequence can be implemented on site to monitor the progress of the excavation works.

Data Analysis

Statistical approach can be used to analyse the data in order to filter out irrelevant data points so that the parameters derived reflect true conditions of the ground. The soil parameters that govern the design should be established from the sensitivity design analyses. The **most probable** set of soil parameters is normally determined as the **mean** values of these parameters.

Sensitivity analyses were also performed in order to identify the governing parameters for the wall design. This is an essential part of the EC7 design approach whereby it is important to understand the importance of the governing parameter(s).

Project Lateral

Project Lateral is a Central London development comprising a three level basement supported by hard/soft mini pile wall with three levels of temporary props. The wall consists of a hard soft secant bored pile wall of 475mm diameter spaced at about 600mm centres. The depth of excavation is 12m deep with formation level at -5.4mOD. A cross section of the of the excavation is shown in Figure 5:

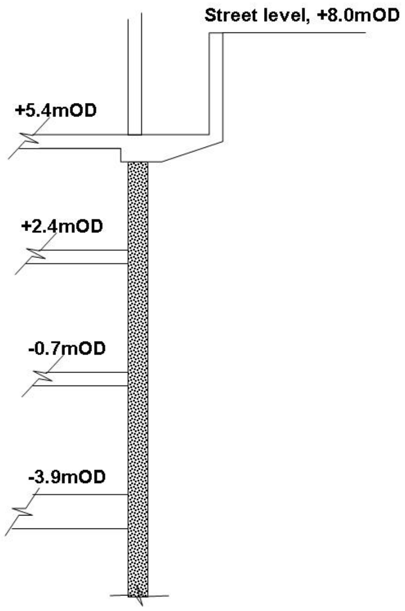


Fig. 5 Cross section of the retaining wall at Project Lateral

Derivation of input parameters. The stratigraphy for the Project Lateral site is summarised below:

Layer	Level (mOD)
Made Ground	+6.6 (Ground level)
Terrace Gravel	+4.5
London Clay	-0.2

For an excavation with a depth of 12m deep, the Made Ground and Terrace Gravel above the London Clay have a total depth of 6.6m. Sensitivity analysis undertaken in the back analysis with allowance within reasonable range of strength and stiffness parameters to investigate if these strata affect the estimate of the wall deflection, see next section.

The undrained shear strength profile, from laboratory and in-situ Standard Penetration Test (SPT), for this site is shown in Figure 6 and the best estimate design line is:

$c_u = 64 + 7.22z_1$ (kN/m²), where z_1 represents depth below -0.2mOD.

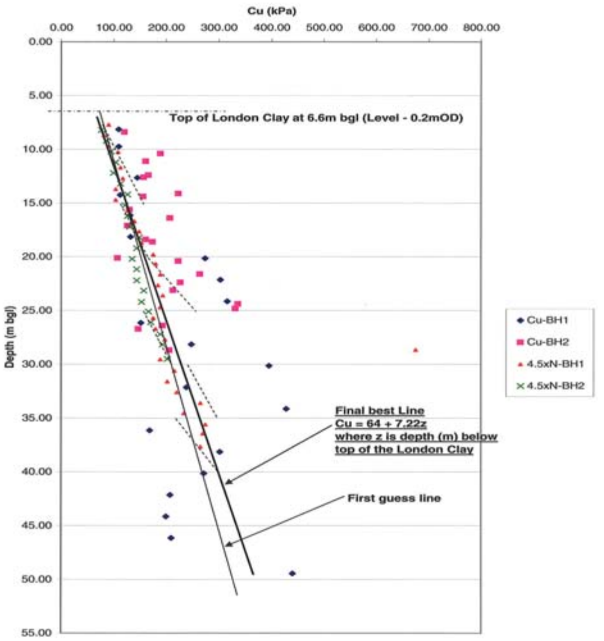


Fig. 6. Undrained shear strength profile for Project Lateral

Derivation of empirical correlation. The measured movements of the retaining wall are shown in Figure 7.

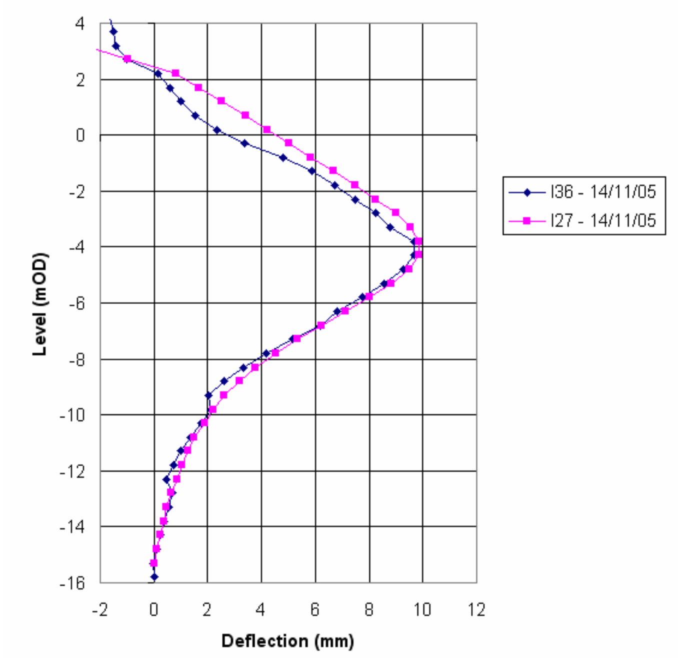


Fig. 7. Measure wall deflection at Project Lateral

With an excavation of 12m depth, the wall deflection is slightly less than 0.1% (0.083%).

The undrained Young's modulus has been calculated as $E_u = f_2 \times c_u$, where $f_2 = 1250$ as a starting point. The wall deflection plot shows $f_2 = 1250$ is a reasonable correlation, despite some movement at the toe of the wall which is not measured on site, see Figure 8.

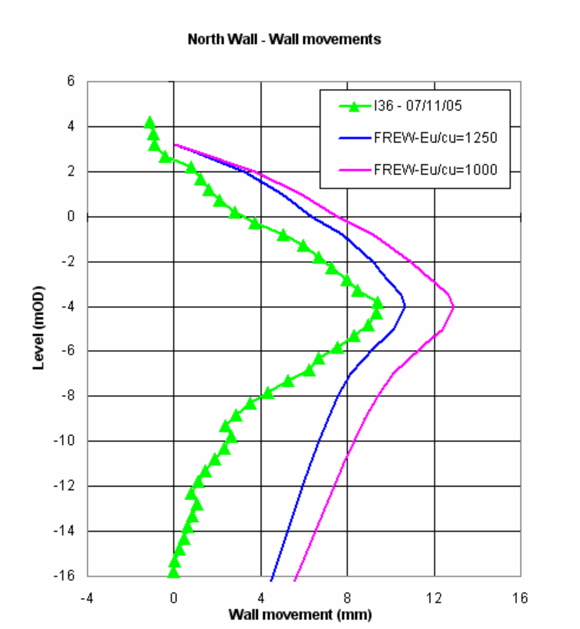


Fig. 8. Computed and measured wall deflection at Project Lateral

Sensitivity analyses undertaken using finite element program show the computed wall deflection is not affected by the strength and stiffness parameters in the River Terrace Deposits (RTD), see Figure 9. The figure also shows that the governing parameters are the soil stiffness of London Clay and to lesser extent the coefficient of earth pressure at rest, K_o.

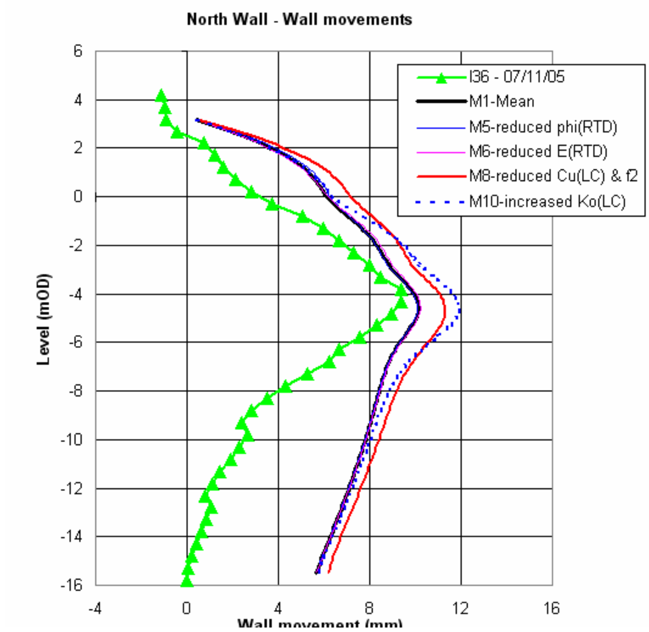


Fig. 9. Sensitivity analyses undertaken using finite element program

Kings Place Development

The Kings Place development comprises an eight storey superstructure and is constructed of a steel frame with reinforced concrete cores and varied facade containing both glass and masonry. The substructure comprises a grade 3 basement which is approximately 16m deep. The basement wall comprises a 1.0m thick reinforced concrete diaphragm wall constructed with panel lengths of up to 6.7m. The structure is founded on a 1m raft foundation which thickens to 1.5m beneath the building cores. The raft is underdrained by a 250mmm thick granular drainage layer. In the permanent case the basement wall structure is propped by the slabs at ground floor and all the three basement slabs.

The cross-section of the excavation is shown in Figure 10 below:

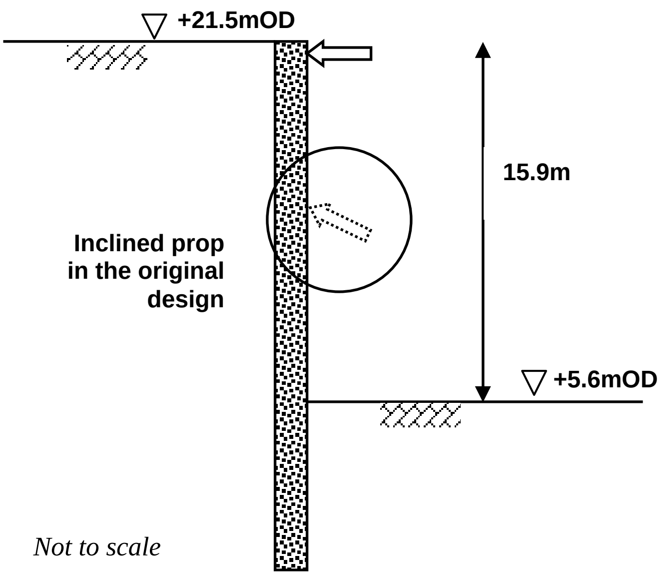


Fig. 10. Cross-section of a typical wall at Kings Place

The image of the excavation is shown in Figure 11 below:

Fig. 11. Excavation of the basement for Kings Place Development (courtesy of Sir Robert McAlpine)

Derivation of input parameters. The measured movements of the retaining wall is shown in Figure 12.

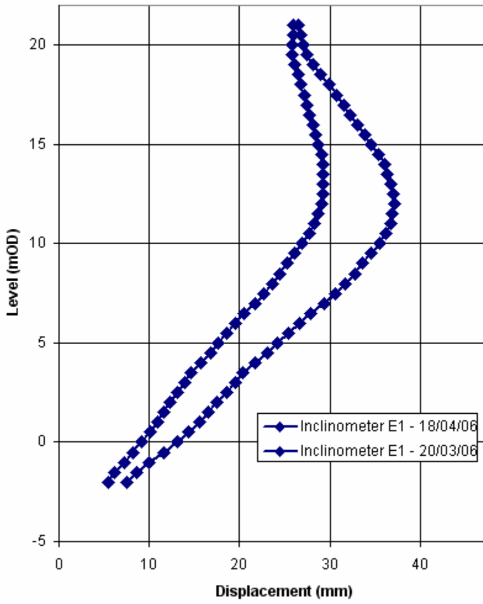


Fig. 12. Measured wall deflection immediately after excavation and when the base slab is cast

The stratigraphy for the Kings Place site is summarised below:

Layer	Level (mOD)
Made Ground Granular	+21.5 (Ground level)
Made Ground Cohesive	+20.5
London Clay	+18.5
Lambeth Clay	-7.0

The undrained shear strength obtained from laboratory tests and those correlated to SPT values are shown in Figure 13.

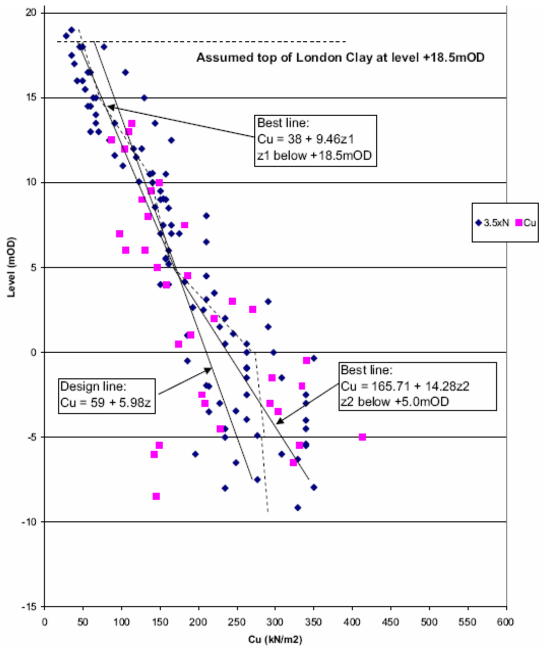


Fig. 13. Undrained shear strength profile for Kings Place Project

The best estimate line chosen for the back analysis is:

- $c_u = 38 + 9.46z_1$ (kN/m²), where z_1 represents depth below +18.5mOD and up to +5.0mOD.
- $c_u = 165.71 + 14.28z_2$ (kN/m²), where z_2 represents depth below +5.0mO.

Derivation of empirical correlation. The undrained Young's modulus has been calculated as $E_u = f_2 \times c_u$, where $f_2 = 750$ as a starting point. The value of f_2 was adjusted in the back analysis to best fit the displacement profile shown by the inclinometer readings. Sensitivity analyses with different at rest earth pressure at rest, K_o, is also undertaken.

Figure 14 above shows that the variation in K_o value between 1.2 and 1.35 produced a difference in wall deflection of about 3mm. The above plot shows an empirical correlation of E/c_u =1200 appropriate for the measurement at Kings Place site if immediate undrained conditions is considered. The one month duration needed to construct the base slab allowed some drainage to happen and a correlation of E_u/c_u =900 is more appropriate for situation where long period of construction is needed for the base slab.

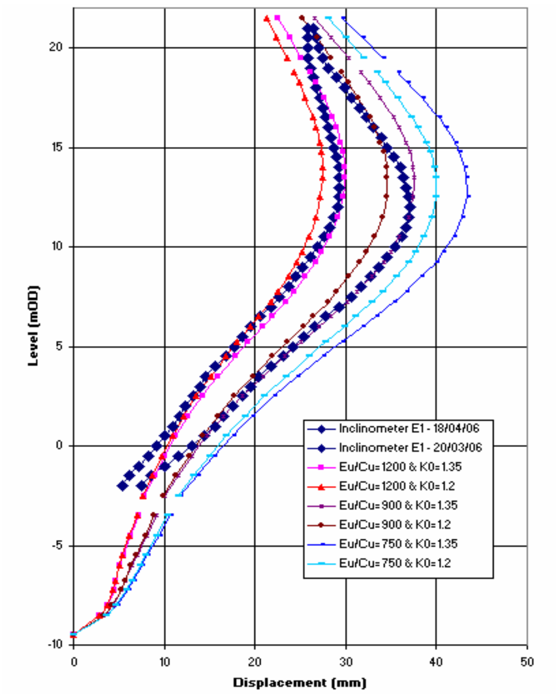


Fig. 14. Computed and measured wall deflections for Kings Place Project

Kings Cross Hub Shaft

Kings Cross hub shaft is part of the new Northern Ticket Hall for the redevelopment of Kings Cross underground station project to ease the congestion at Kings Cross's underground railway system and to provide the extra capacity needed for the passengers arriving from Europe when the new International Terminus of the Channel Tunnel Rail Link opens at St Pancras Station. It is designed to form the access shaft from the ticket hall to the deep underground stations of the Victoria, Piccadilly and Northern Lines of the London Underground system.

The shaft is supported on 1.2m contiguous bored pile wall spaced at 1.4m centres with three levels of temporary props. Total depth of the excavation is 21.3m. Figure 15 shows the cross section of the retaining wall and the permanent slab levels.

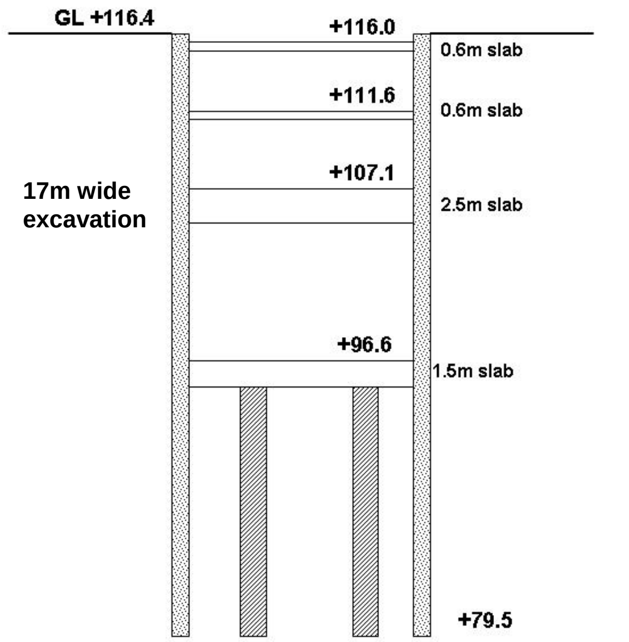


Fig. 15. Cross section of the Kings Cross Hub Shaft

Figure 16 shows the construction stage of the Hub Shaft with the top two levels of propping:

Fig. 16. Image of the excavation at Kings Cross Hub Shaft

Derivation of input parameters. The geological cross-section based on the site investigation holes is given below:

Stratum	Top of stratum, (mTD)	Thickness (m)
Made ground	+116.9	3.9
London Clay	+113.0	18.0
Lambeth Group	+95.0	17.0
Thanet Sands	+78.0	3.0
Chalk	+75.0	-

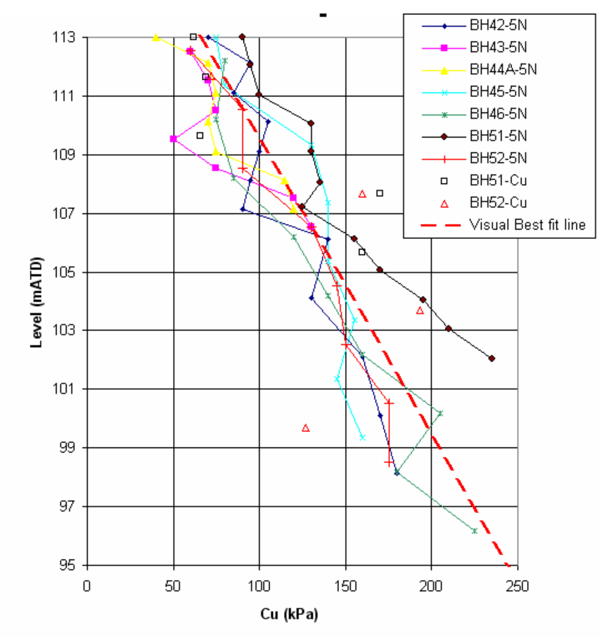


Fig. 17. Undrained shear strength profile at Hub Shaft

The best estimate for undrained shear strength (see Figure 17 above) chosen is the following:

c_u = 65+10z (kN/m²), where z represents the depth below top of the London Clay.

<u>Derivation of empirical correlation.</u> The measured wall movement is shown in Figure 18 with maximum wall deflection between 11 and 13mm:

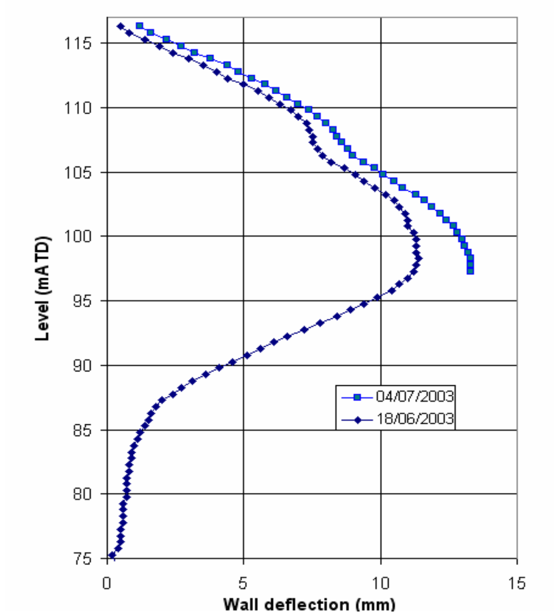
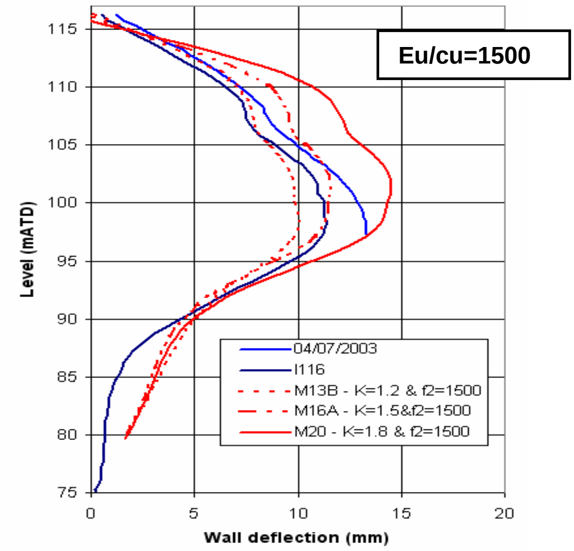


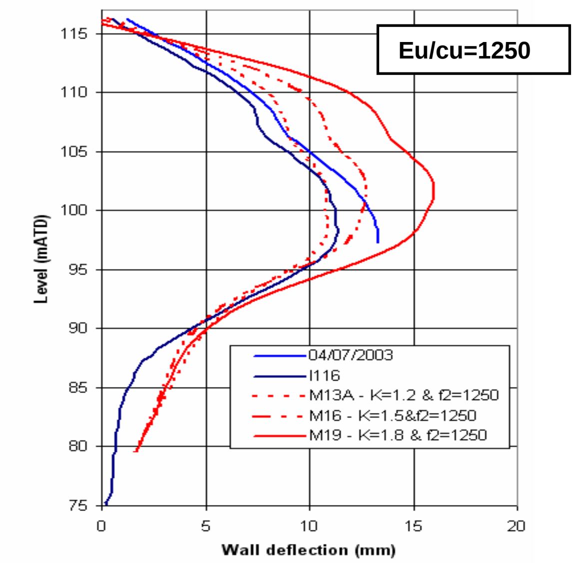
Fig. 18. Measured wall deflection for the Hub Shaft

The deflection is equivalent to 0.054% of the excavation depth.

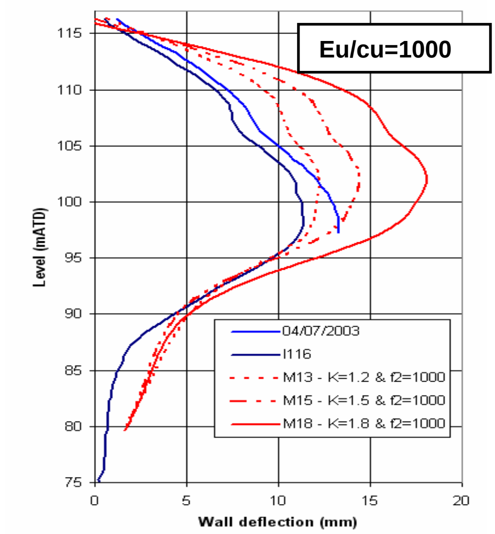
A change in K_o value from 1.2 to 1.8 causes about 4 to 5mm difference in wall movements, see Figures 19a to 19c. Changes in the stiffness value from E_u/c_u=1000, 1250 and 1500 are also shown in the figures below. These figures show that an E_u/c_u value best fit the measured value is between 1000 and 1250, for K_o value between 1.2 and 1.5.



a) Computed wall deflection for E_u/c_u=1500



b) Comparison of wall deflections for E_u/c_u=1250



c) Comparison of wall deflections for E_u/c_u=1000

Fig. 19. Computed and measured wall deflection at Kings Cross Hub Shaft

Thamelink Box

The main structure of the Thameslink Box consists of a new cut and cover station box built along the alignment of an existing Thameslink line tunnel. The station box is about 380m long and about 22m wide. The depth of excavation is about 11m deep with a single permanent prop near ground level which also formed part of the roof structure.

The retaining walls comprise of contiguous wall with 1.2m diameter piles at 1.35m centres, and hard/hard secant wall with 1.2m diameter piles at 1.9m centres with intermediate 1.2m diameter female piles. The retaining walls were installed outside the existing Thameslink masonry tunnel in advance of the construction work. A typical cross section of the box is shown in Figure 20.

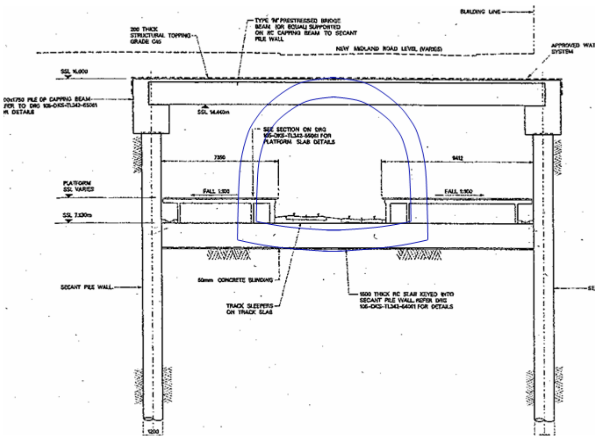


Fig. 20. Typical cross-section of the box

Derivation of input parameters. A typical stratigraphy comprising London Clay over Lambeth Beds, Upnor Formation, Thanet Sand, Upper Chalk, overlain by Made Ground has been defined by RLE based on their interpretation of ground conditions from a review of ground investigation at and near the site. Table 1 shows the stratigraphy used in the back analyses.

Stratum	Level (mOD)
Made Ground	+17.5
London Clay	+16.5
Lambeth Beds	-3.0

It has to be highlighted that at design stage, the London Clay was divided in two layers, the upper defined as Weather London Clay, and the lower layer as London Clay. In the data analysis carried out for the SCOUT project, one layer is considered appropriate. The best estimate for undrained shear strength is:

$c_u = 54 + 7.83z$ (kN/m²), where z represents the depth below top of the London Clay, see Figure 21.

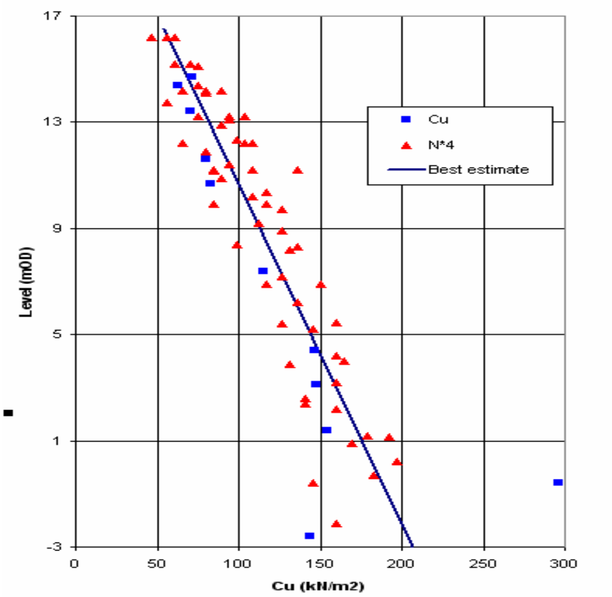


Fig. 21. Undrained shear strength for Thames Link Box

Derivation of empirical correlation. The construction sequence of this station box consisted a shallow depth cantilever excavation to install the precast roof beams before excavation and demolition of the existing masonry tunnel to the formation level where the base slab will be cast. Measured wall movements was only about 5mm, as shown in Figure 22.

For an excavation of about 11m deep, this represents about 0.05% of wall deflection, a very small measured value compared with other case histories.

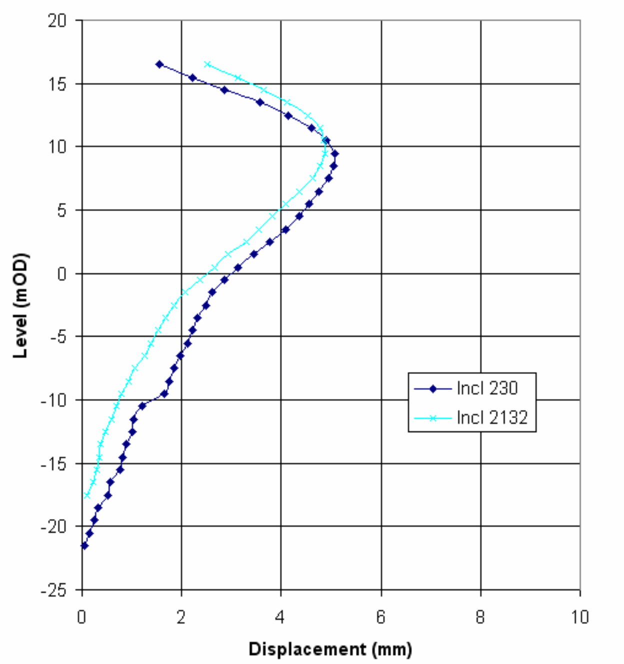


Fig. 22. Measured wall deflection at Thames Link Box

Using a E_u / c_u correlation, the following Figure 23 shows the computed wall deflection of the station box:

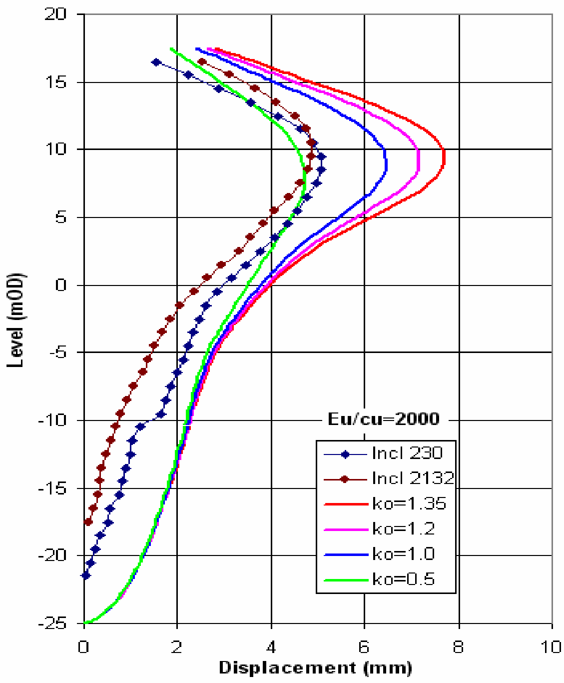


Fig. 23. Measured and computed wall deflection for Thames Link Box

The figure shows, in order to obtain a good match to the measured wall movement, a low value of K_o =0.5 would be necessary. This is inconsistent with the coefficient of earth pressure at rest of a stiff heavily consolidated London Clay. However, if we consider the effects of construction of the existing masonry tunnel and assuming that the coefficient of earth pressure at rest has not been reinstated over years, it is plausible that the earth pressure responsible for the performance of the retaining wall is the active pressure instead of its virgin at rest earth pressure. For London Clay with ϕ'=22°, the active earth pressure coefficient is about 0.6.

With K_o =0.5, sensitivity analyses were undertaken to derive the best E_u /c_u correlation and the results are plotted in Figure 24.

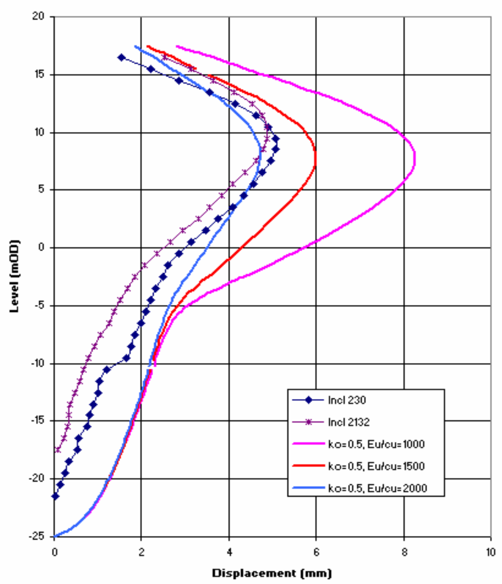


Fig. 24. Measured and computed wall deflection for Thames Link Box with K_o=0.5

The figure shows the most appropriate correlation is E_u /c_u =1500.

Ropemaker Project

The proposed development consists of a new 7 to 25 storey structure. The new structure will also have a two level basement formed in the southern part of the site and a three level basement formed in the northern part of the site below the existing basement.

The retaining wall consists of hard-hard secant pile of 1.2m diameter, evenly spaced at 1.05m centres. Male pile toe level is at -6.25mOD and female pile toe level at -0.5mOD. The depth of excavation varies between 6 and 7m deep. Figure 25 shows the excavation at its final depth.

Fig. 25. Excavation at formation level of the basement of the Ropemaker Project

Derivation of input parameters. Review of the geotechnical interpretive report indicates the following stratigraphy:

Stratum	Top Level (mOD)
Made Ground (MG)	+11 to +16.
London Clay (LC)	+9
Lambeth Group (Reading Formation)	-14
Lambeth Group (Upnor Formation):	-25
Thanet Sand	-30
Chalk	-42

The undrained shear strength is plotted in Figure 26.

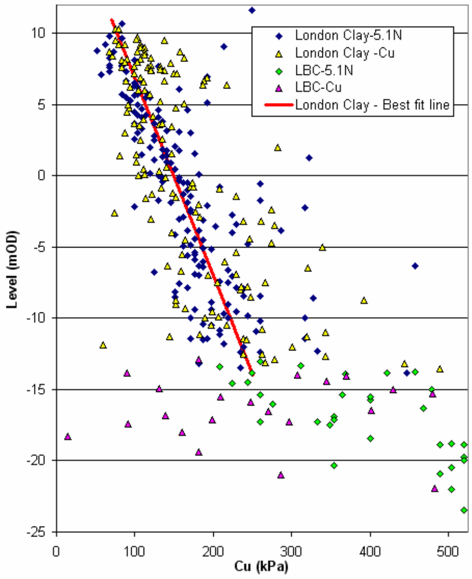


Fig. 26. Undrained shear strength profile at Ropemaker site

The best fit line is chosen as:

$c_u = 70 + 7.2z$, z is the depth below top of London Clay

<u>Derivation of empirical correlation.</u> The measured wall movement ranges between 6 and 12mm, as shown in Figure 27. This is another case of very small movement of 0.1% to 0.2% of the excavated depth.

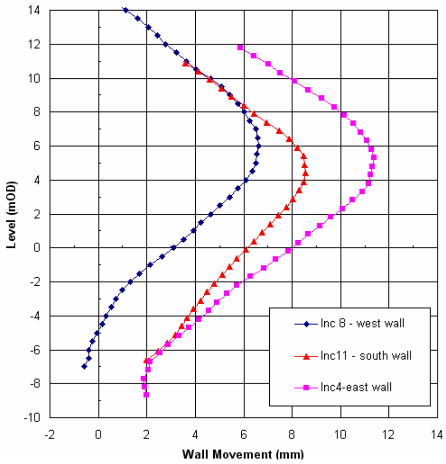
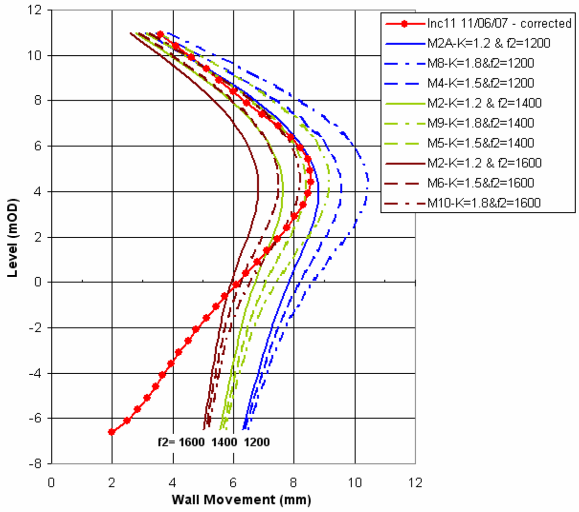


Fig. 27. Measured wall deflections at Ropemaker Project

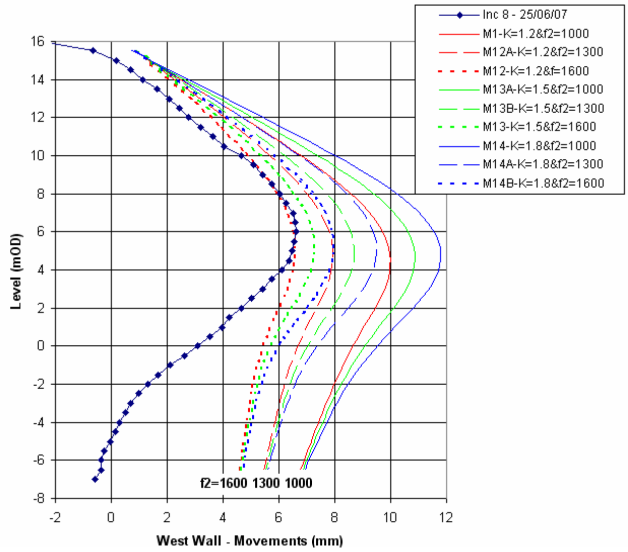
The back analyses produced the following match to the measured wall movements.

The sensitivity analyses show that varying the K_o values beyween 1.2 and 1.8 produced less than 2mm difference in wall movements, see Figure 28a. The most appropriate empirical correlation between strength and stiffness is about E_u /c_u =1200, for a K_o value of 1.2.

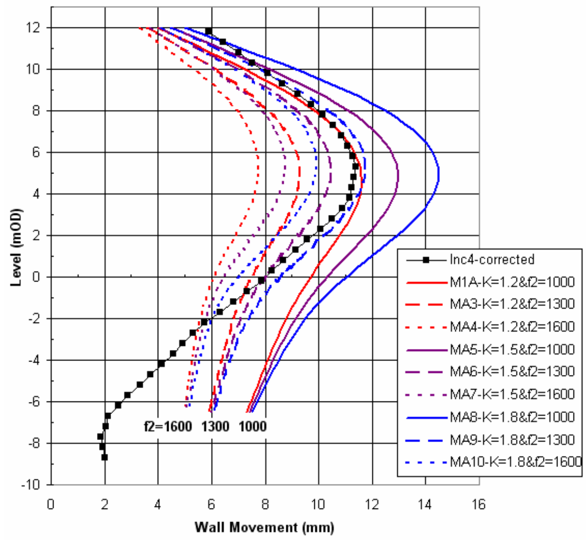
Again Figure 28b shows varying K_o does not change the computed wall movement by much. In this case the empirical correlation between strength and stiffness is E_u /c_u =1600 for a K_o value of 1.2.



a) Comparison of wall deflections - South Wall



b) Comparison of wall deflections - West Wall



c) Comparison of wall deflections - East Wall

Fig. 28. Comparison of computed and measured wall deflections for Ropemaker Project

The variation in K_o produced about 3mm difference in wall displacement. In this case the empirical correlation between strength and stiffness is about $E_u / c_u = 1000$ for a K_o value of 1.2, see Figure 28c.

Summary of the Case Histories back analyses

The results of the case histories back analyses are summarised in the table below:

Project name	Excavation depth, H (m)	Max measured deflection, δ (mm)	δ/H (%)	E_u/c_u ratio
Project Lateral	12	10	0.08	1250
Kings Place	16	29-40	0.2-0.25	900-1200
KX Hub Shaft	21.3	11-13	0.05-0.06	1250
Thames Link Box	11	5	0.045	1500*
Ropemaker	6-7	6-12	0.1-0.2	1000-1600

CONCLUSIONS

The design optimisation assessments undertaken in the analysis of the cut-and-cover retaining structure have shown that substantial savings can be obtained when one or a combination of the following design approach is adopted:

- The use of more complex finite element design approach with the consideration of change in stiffness with curvature of the retaining wall;
- Use a variable bending stiffness approach instead of constant stiffness approach in the design of the retaining wall;
- Allow less stringent or even no crack width consideration and properly advise the owner /client of the watertightness and aesthetic issues;
- Use more efficient structural form to encourage arching to reduce the amount of reinforcement.

The optimisation assessments undertaken in the SCOUT project show that a saving of material cost of more than 15% could be achieved based on design comparison using conventional reinforced concrete.

Back analyses of five case histories of measurements taken from construction site form the core of the design using the OM approach in the SCOUT project. The measured wall movements were used to identify **most probable** parameters for design using the OM approach in stiff London Clay. Rigorous sensitivity analyses were performed to identify the governing parameter(s) for retaining structure, which is essential in the design framework under the Eurocode.

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