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The effect of wall stiffness on bending moments

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ABSTRACT: Many deep excavations in urban areas are currently being constructed using stiff concrete diaphragm or secant pile walls. Research has shown that as wall flexibility increases the stresses imposed by the soil redistribute and reduce the structural forces in the wall. Unfortunately this beneficial effect is accompanied by greater wall and soil movements. In this paper the results of numerical analyses are used to quantify these effects. Three typical excavations in stiff clay are analysed. For each case, analyses using a range of wall stiffness from a Frodingham 1N to a 1m thick concrete wall have been performed. Large reductions in maximum bending moment occurs as the flexibility is increased and this is accompanied by increased wall and soil movements. The magnitude of the increased movement depends on the number of propping levels. It is concluded that in many situations a flexible sheet pile wall solution can be both viable and more economic.

1 INTRODUCTION

Deep excavations in stiff clay are often constructed using relatively stiff concrete diaphragm or secant pile walls. This is particularly true in urban areas such as London were site restrictions often make other support systems impractical. Accepted methods of design (Padfield & Mair 1984, British Standards Association 1951) or finite element analysis indicate that stiff walls must be designed to withstand large bending moments.

Experimental work carried out by Tschebotarioff (1948) and Rowe (1952) and recent numerical studies by Potts and Fourie (1984,1985,1986) clearly indicate that under the same operating conditions stiffer walls attract larger bending moments than more flexible walls. As wall flexibility increases there is increased freedom for stresses imposed by the soil to redistribute and reduce the structural forces imposed in the wall. While this is clearly beneficial it occurs at the expense of larger soil and wall movements. There is therefore a compromise between reduced bending moments and increased movements as the flexibility of the wall is increased.

Whether or not flexible sheet piles can provide a viable alternative to concrete diaphragm or secant pile walls for retaining deep excavations depends on quantifying the magnitudes of the moment reduction and increase in movements. A more economic solution may be available in situations where moment reduction leads to the use of lower modulus wall sections and the increased movements can be tolerated.

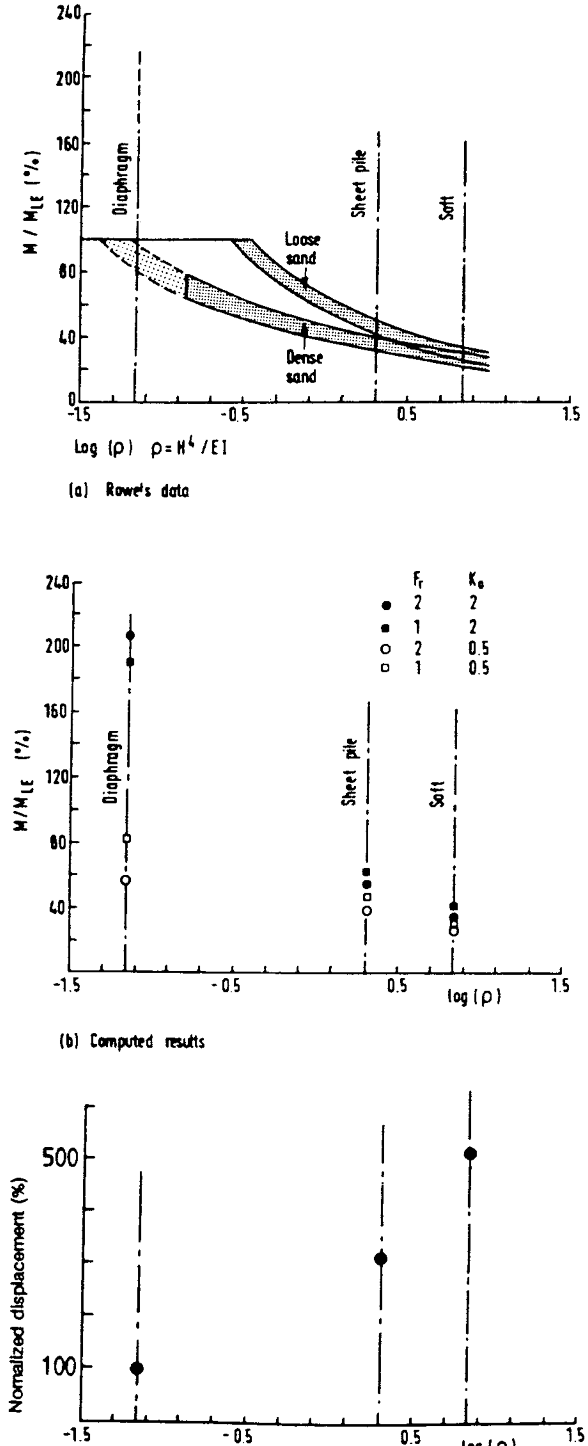
To assess the potential for use of sheet piles in deep excavations in stiff clay, finite element analyses have been performed for three excavations in the London area. For each case the results of four analyses using a range of wall stiffness covering three orders of magnitude are presented. The three cases chosen for investigation are typical of deep excavations in the London area. They differ mainly in the degree of support provided to the walls. Results from the analyses quantify both the moment reduction and the increased movements associated with reducing the stiffness of the wall. Relative costs of some concrete and sheet pile alternatives are also given.

2 REVIEW

As early as 1923 the Danish Rules (Danish Engineering Society 1952) recognised that the maximum bending moment in anchored sheet pile walls could be considerably less than that calculated using classical triangular earth pressure distributions. This conclusion was reached following observations of many existing quay walls and is discussed qualitatively by Stroyer (1928,1935).

Comprehensive model tests by Tschebotarioff (1948) and Rowe (1952) have confirmed these observations and lead to the first quantitative evaluation of the effect of wall flexibility on bending moments. Theoretical studies by Baumann (1934), Hansen (1953), Terzaghi (1953), Rowe (1955) and Richart (1957), and more recent finite element analyses by Bjerrium (1972), Smith & Boorman (1974) and Potts & Fourie (1984,1985,1986), have demonstrated that the maximum bending moment in anchored sheet pile retaining walls is dependent on the stiffness on the wall. These studies have confirmed the results of the earlier model tests.

To illustrate the effect of wall flexibility on bending moment Figure 1a shows Rowe's (1952) experimental results for a propped retaining wall. The bending moment reduction factor, M/M_{LE}, (where M is the maximum observed bending moment and M_{LE} is the maximum theoretical bending moment given by a limit equilibrium "free earth" calculation) is plotted against the flexibility number ρ ($\rho = H^4/EI$, where H is the depth of wall and EI is the bending stiffness). The shaded zones represent Rowe's results for dense and loose sands and the vertical lines marked "diaphragm" and "sheet pile" indicate the values of ρ associated with a 1m thick concrete wall and a larssen 4B sheet pile section respectively. The line marked "soft" represents a lower extreme of wall stiffness. Figure 1a clearly shows that as the flexibility of the wall increases, the induced bending moment reduces. Figures 1b and 1c show the results of numerical predictions by Potts and Fourie (1985) for a propped retaining wall. Results are presented when the factor of safety, F_r (Burland et al 1981) equals 1 and 2, for the cases of high insitu horizontal stress ($K_o = 2$, where K_o is the ratio of horizontal to vertical effective stress) and for low insitu horizontal stress ($K_o = 0.5$). It can be seen that the theoretical results are similar in form to



c) Computed displacement (F_r=2;K_o=2)
Figure 1 Effect of flexibility on maximum bending moment

the experimental results and for flexible walls they are in good quantitative agreement. The theoretical moment reduction factors for K_o=2 lie above those for K_o=0.5 although for flexible walls the differences are much less than for stiffer walls. It is important to note that for K_o=2 the arbitrary cut-off at M/M_{LE}=1, shown in Figure 1a is theoretically invalid and for stiffer walls the bending moments can be larger than those given by the simple limit equilibrium calculation. Rowe did not perform tests on walls in soils with high K_o values. Figure 1c shows that there is a significant increase in wall displacement if the stiffness of the wall is reduced.

The results of this work has serious implications for the design of retaining walls in stiff clay (high K_o). As shown in Figure 1b and 1c increasing the flexibility of the wall can significantly reduce the induced bending moment and increase the wall displacement. Because of this effect it may be possible to provide a range of solutions from a stiff to a flexible wall. While a flexible wall might be cheaper it will also involve greater soil and wall movements. There is therefore a complex interaction between bending moment, soil and wall movements, and cost, as the flexibility of the wall changes. This paper investigates this in greater detail.

3 DETAILS OF ANALYSES

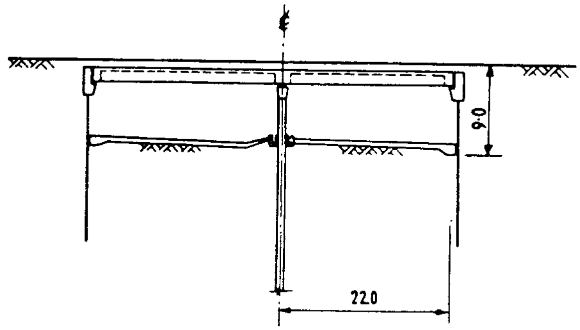
The Imperial College Finite Element Program (ICFEP) was used to perform analyses of three excavations in stiff clay. The excavations chosen for analysis were the Bell Common Tunnel (Potts & Burland 1983, Hubbard et al 1984, Tedd et al 1984, Higgins et al 1989), the George Green Tunnel, (Potts & Knights 1985) and the House of Commons car park (Burland & hancock 1977). These were selected because each was designed with roughly equal concrete wall stiffness but with different horizontal prop conditions. Both the Bell Common tunnel and the House of Commons car park have been constructed. Construction of the George Green tunnel is due to start in 1991. The final design was not available at the time the present work was carried out and therefore the analysis has been based on the available published details. The Bell Common and George Green tunnels consist of 1.2m diameter secant piles and the House of Commons car park is a 1m diaphragm wall. The Bell Common wall has a single horizontal prop near the top (Figure 2a). The George Green wall is propped by 1m deep concrete slabs with full moment connectivity to the wall at the top and at excavation level (Figure 2b). The House of Commons car park consists of 6 floor levels providing only horizontal support to the wall (Figure 2c).

3.1 Bell Common Tunnel

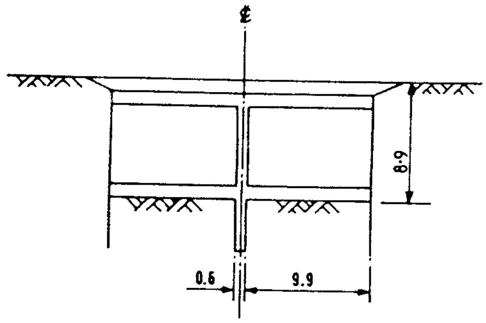
A description of the Bell Common Tunnel and its design is given by Hubbard et al (1984). The geometry of the wall and soil parameters used in this analysis are shown in Figure 3. Due to symmetry only half the tunnel cross section was analysed. The initial pore water pressure was assumed hydrostatic from the ground water position shown on Figure 3.

The construction sequence adopted for the analysis was based on that actually followed (Higgins et al (1989) and was:

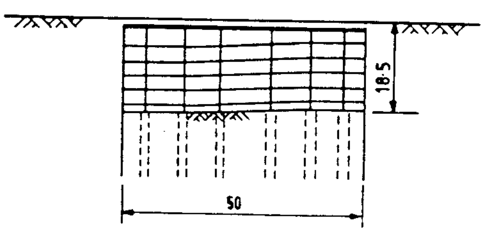
- Install main walls.
- Excavate to level 1 with temporary sheet pile wall propped against the main wall.
- Construct thrust wall and cill beam. Prop temporary wall to back of thrust wall.
- Excavate to level 2.
- Construct roof slab. (Install equivalent spring and apply vertical load to the cill beam and central piles).
- Excavate to level 3.
- Apply loads arising from protection of roof slab.
- Excavate to formation level.
- Backfill behind thrust wall and remove temporary sheet pile wall.
- Construct road slab.



(A) Bell Common Tunnel



(B) George Green Tunnel



(C) House of Commons Car Park

Figure 2 General arrangement of walls

- Apply loads arising from backfill of 0.6m over roof slab.

The support provided by the roof slab was modelled by a spring with a linear stiffness of 10 MN/m per metre. Point loads were applied to the cill beam and the centre pile due to the loads from the roof slab. The road slab does not provide any support to the wall. The soil was assumed to remain undrained during construction. After construction the excess pore pressure was dissipated to the long term pore pressure distribution which was based on the flow net given in Higgins et al (1989).

3.2 George Green Tunnel

A description of George Green Tunnel is given by Potts & Knights (1985). The geometry of the wall, soil parameters and initial pore water pressure distribution used in this analysis are shown in Figure 4. The roof and road slabs provide a full moment connection to the wall. Both sides of the tunnel were assumed to be constructed simultaneously and therefore only half the tunnel was analysed. In practice this may not be the case and consequently it may be necessary to analyse the complete section.

The construction sequence assumed was:

- Install walls.
- Excavate to 3m below original ground level.
- Construct roof slab.
- Excavate beneath roof slab to formation level.
- Construct road slab.
- Backfill 1m of soil over tunnel roof and apply a traffic surcharge of 10.5 kN/m^2 to the fill.

The Boyn Hill Gravel and the fill were assumed to remain drained, and the London Clay undrained, during construction. After construction the excess pore pressure was dissipated back to the initial pore pressure distribution (Figure 4).

3.3 House of Commons car park

A description of the House of Commons car park is given by Burland and Hancock (1977). The geometry of the wall and soil parameters used in this analysis are shown in Figure 5. Again symmetry was assumed and only half the cross section was analysed. The initial pore water pressure was assumed hydrostatic from the ground water position shown on Figure 5.

The construction sequence assumed was based on that actually followed and was:

- Install walls.
- Excavate to soffit of roof slab.
- Construct roof slab.
- Excavate beneath roof slab to first floor level.
- Construct floor slab.
- Excavate beneath floor slab to next floor level.
- Construct floor slab.
- Continue procedure on successive floors except lowest floor.
- Excavate to final formation level.
- Construct lowest floor level leaving void beneath slab.

The roof slab and five floor slabs were assumed to be simply supported by the wall and thus provide horizontal support only. The layer of sand/gravel at the surface was assumed to be drained, and the London Clay undrained, during construction. After construction the excess pore pressure was dissipated to the long term pore pressure distribution given in Burland and Hancock (1977).

3.4 Finite element analysis

Plane strain conditions were assumed. Eight node isoparametric 2-D elements were used to model the soil and three node isoparametric 1-D beam elements were used to model the wall. Reduced integration was used for all elements. In all cases the analysis began after wall installation and closely followed the construction procedure. It was assumed that wall installation had no effect on the initial insitu soil stress. ie. the walls were assumed to be "wished in place".

Soil was assumed to behave as a linear elastic - perfectly plastic material obeying the Mohr-Coulomb yield criteria with an angle of shearing resistance ϕ'. The plastic potential, with the same shape as the yield surface, was defined by the angle of dilation, ν. For the George Green Tunnel and the House of Commons car park analyses, $\nu = \phi'/2$, whereas for the Bell Common analyses $\nu = 0$ to be consistent with Higgins et al (1989).

During the construction stages of the analysis the undrained soil layers were modelled using effective soil parameters (c', ϕ', E', μ', ν) combined with a bulk modulus

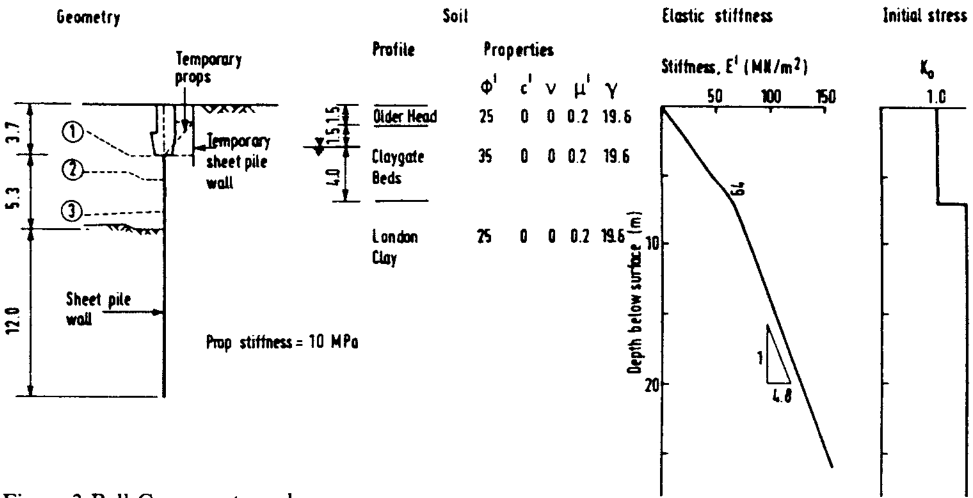


Figure 3 Bell Common tunnel

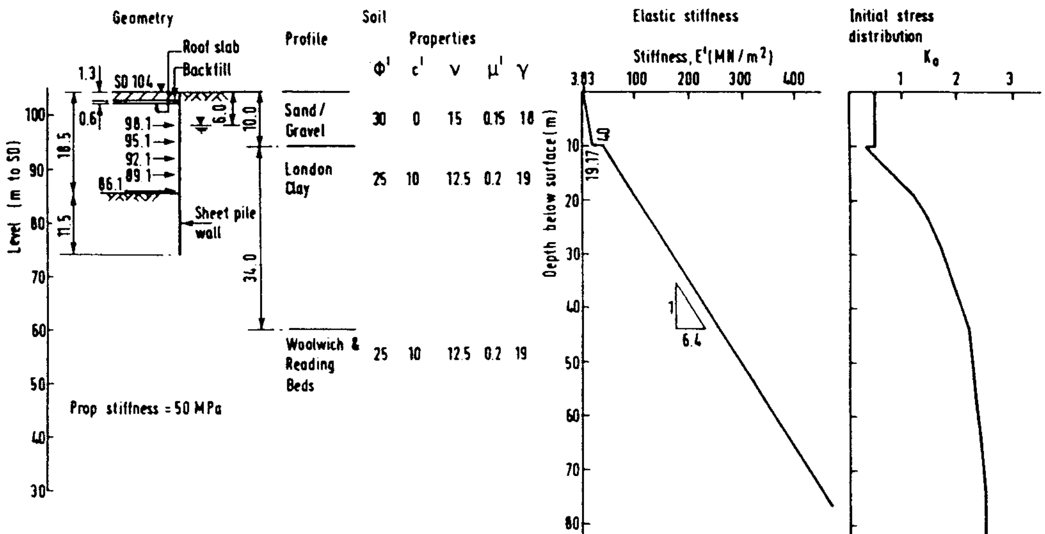


Figure 4 George Green tunnel

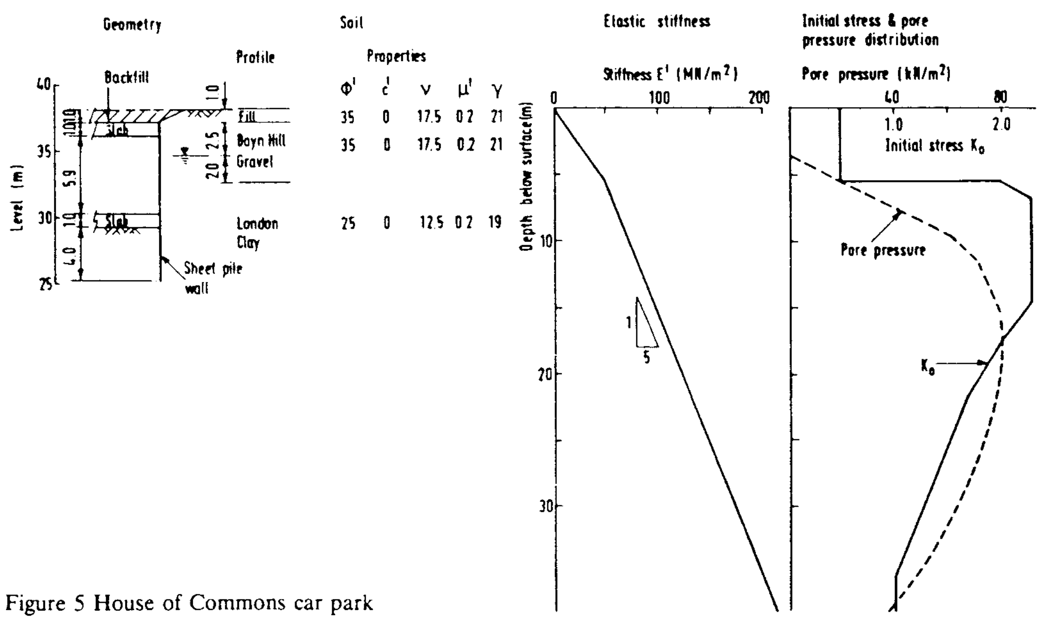


Figure 5 House of Commons car park

for the pore water (Naylor et al 1981). In the present analyses the bulk modulus of pore water was set to be 100 times greater than the effective bulk modulus of the soil skeleton. As a result the behaviour of the soil is nearly incompressible. It also enables the calculation of the undrained changes in pore water pressure.

For the dissipation stages of the analysis the pore pressures after construction are compared with the final long term values. The differences between these are then used in the finite element calculations to evaluate the long term displacements and changes in effective stress. During the dissipation stage of the analysis the bulk modulus of the pore water is set to zero.

The sheet pile wall was assumed to be linear elastic. Four pile sections, representing the range from a very light sheet pile section to a very stiff high modulus sheet pile, were used in the analyses. The same four pile sections were used in each wall studied (Table 1). Youngs modulus, E, equal to $2.1\text{x}10^8$ kN/m^2 and poissons ratio, μ, equal to 0.2 were used.

The bending stiffness of the High Modulus 4N section is equivalent to that of a 1m thick uncracked concrete wall. Analyses in which a 1m thick wall was modelled with eight node 2-D elements gave results similar to the High Modulus 4N section and are not presented.

4 RESULTS OF ANALYSES

4.1 Wall behaviour

Figure 6 shows the bending moment distributions in each wall in the long term after all excess pore water pressures have dissipated. It is clear that the high stiffness walls attract much larger bending moments than the flexible walls. In Figure 7 the normalised maximum bending moment, defined as the maximum bending moment divided by the maximum bending moment in the High Modulus 4N wall, is plotted against wall stiffness. Plotting the data in this way allows comparison of the three cases. As the wall stiffness is reduced over the range analysed there is a similar reduction of a factor of approximately five in the maximum bending moment for all three cases. This similarity occurs despite the different prop conditions and initial stress in the soil in each of the walls.

Table 1 Sheet pile stiffness per metre of wall

	A ($\text{x}10^{-2}$m^2)	I ($\text{x}10^{-5}$m^4)	ln(EI)
Frodingham 1N	1.26	6.048	9.45
Frodingham 4N	2.18	39.83	11.33
High Modulus 1BXN (610x229x113 US with Frodingham 1BXN wings)	3.17	208.2	12.99
High Modulus 4N (914x305x289 US with Frodingham 4N wings)	6.00	1109.8	14.66

The maximum calculated bending moment and the maximum allowable bending moment for each case are compared in Table 2. In calculating the allowable bending moment, the conservative assumption that the axial force reduces both the maximum permissable compressive and tensile stresses by equal amounts, has been made. For the George Green tunnel the most critical stage is immediately after construction. Therefore 225 MPa was used as the maximum allowable stress in the sheet pile walls. For the other two sites a maximum allowable stress of 180 MPa was used. Table 2 indicates that a pile section as light as a Frodingham 4N would be sufficient to carry the structural forces in each case.

The deflected shapes of the walls in the long term are presented in Figure 8. These correspond to the bending moment distributions given in Figure 6. Clearly the deflected shape of the wall is also dependent on the wall

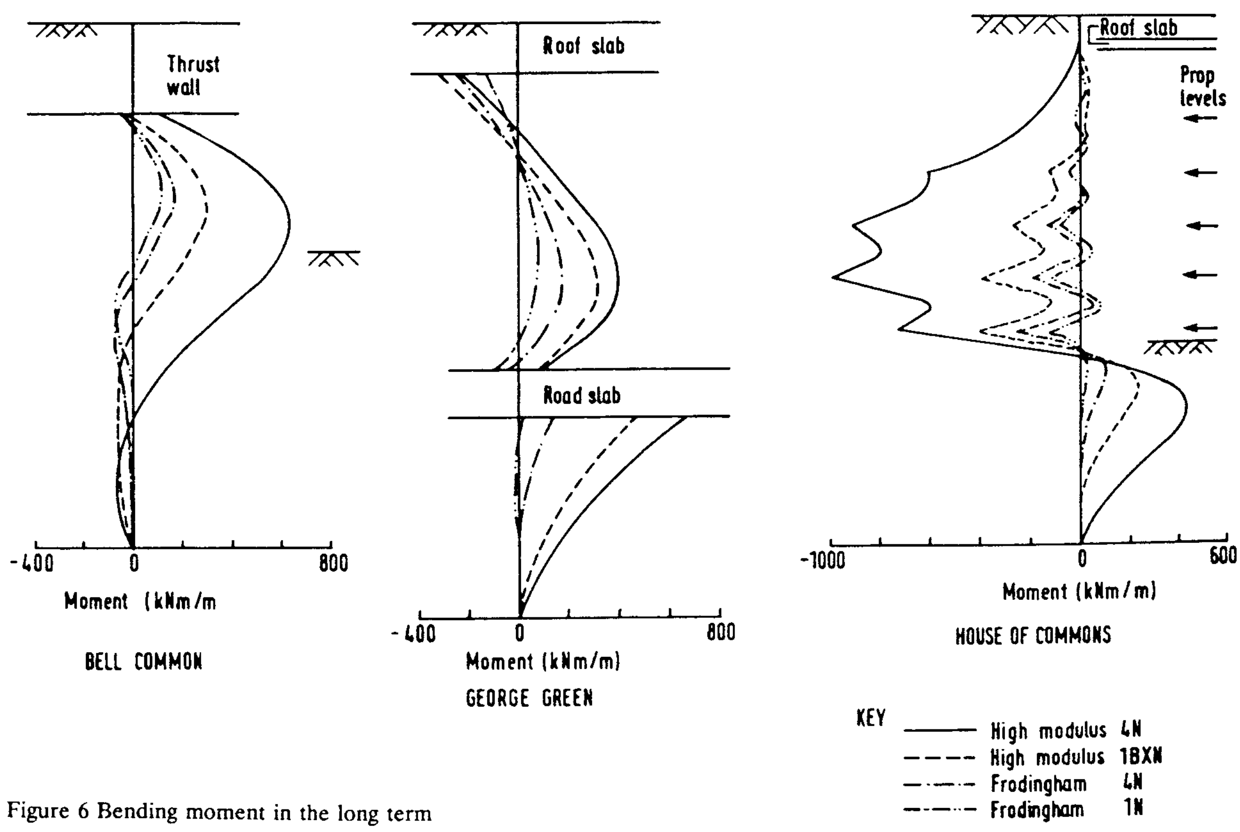


Figure 6 Bending moment in the long term

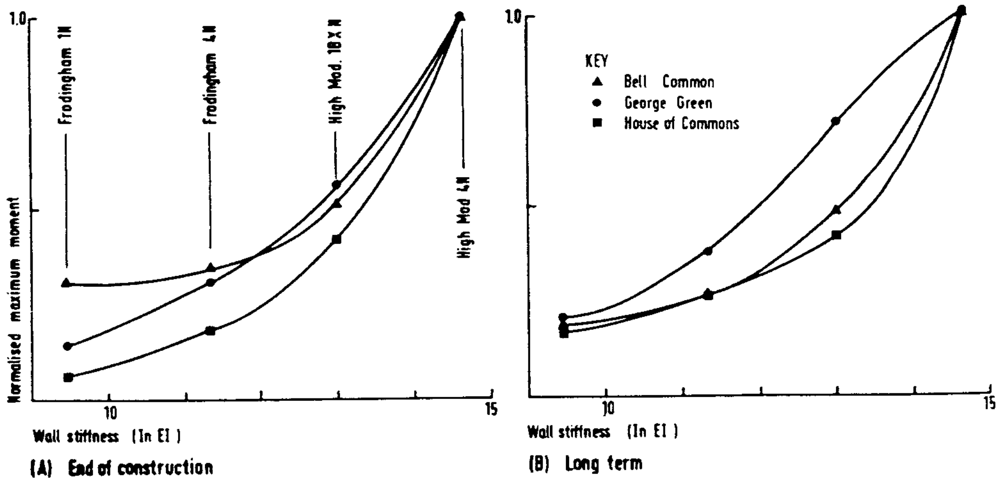


Figure 7 Normalized maximum bending moment

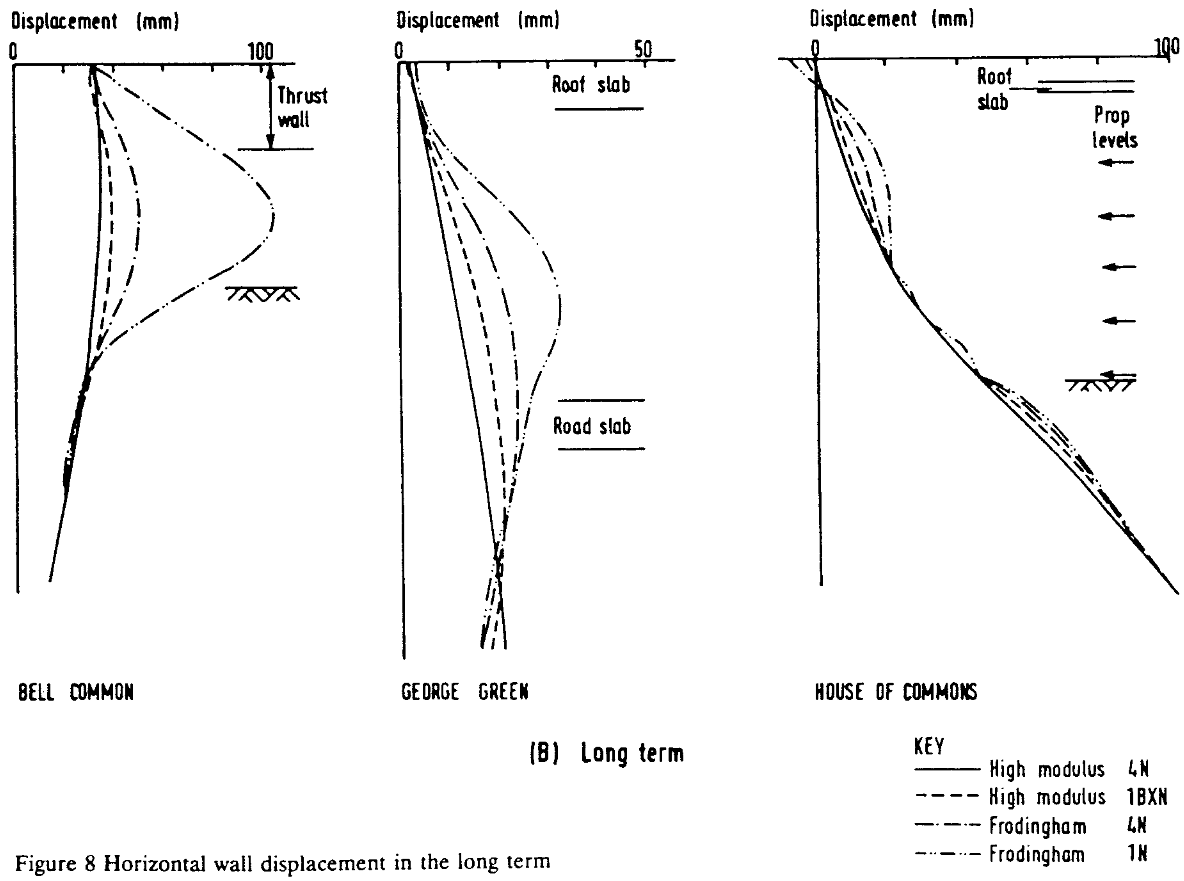


Figure 8 Horizontal wall displacement in the long term

stiffness. The normalized maximum deflection, defined as the maximum deflection divided by the maximum deflection of the High Modulus 4N wall, is plotted against wall stiffness in Figure 9. This shows the maximum deflection is not only dependent on the wall stiffness but also on other factors such as the number and type of props. For the singularly propped Bell Common Tunnel there is an increase in wall deflection with reduced wall stiffness. For the multi-propped House of Commons car park, the maximum deflection in only slightly influenced by the wall stiffness. This suggests that in this case the wall deflection is controlled more by the props rather than the wall stiffness.

The effect of wall stiffness on the maximum long term deflection of the doubly propped George Green Tunnel wall lies between the other two cases. The different shape of the curve for the short term deflection of the George Tunnel wall is likely to be due to the full moment connection to the roof slab at the top of the wall.

For the worst case (corresponding to the George Green tunnel immediately after construction) reducing the stiffness of the wall from the High Modulus 4N pile to the Frodingham 4N pile results in only a 60% increase in maximum displacement. For the other cases the effect of wall stiffness is much smaller.

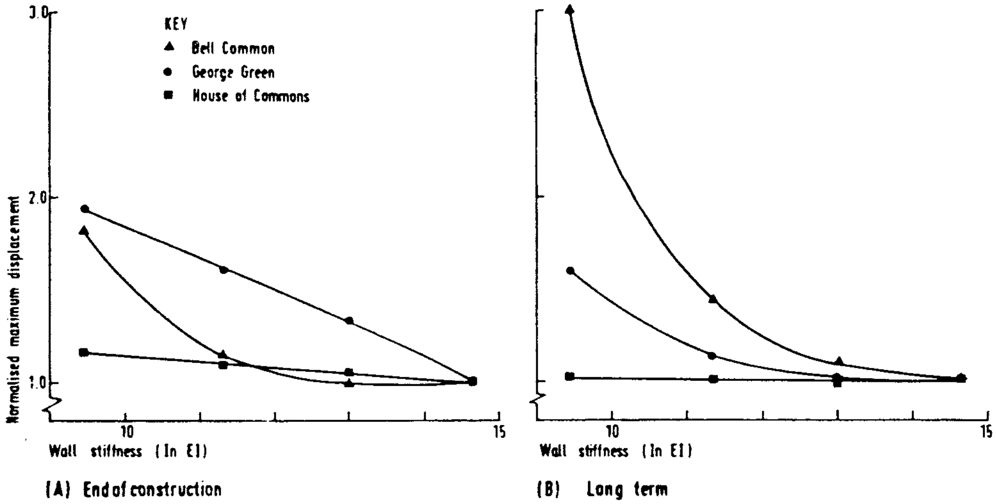


Figure 9 Normalized maximum displacement

Table 2 Maximum bending moment in sheet pile walls. (Grade 50 steel)

	Bell Common		George Green		House of Commons	
Axial Force (kN/m)	650		410		500	
Bend. Moment (kNm/m)	Anal.	Allow.	Anal.	Allow.	Anal.	Allow.
Frodingham 1N	120	40	170	129	170	93
Frodingham 4N	170	347	410	490	270	372
High Mod. 1BXN	305	655	740	878	410	676
High Mod. 4N	635	2710	1250	3500	1000	2748

Axial force can be significant in reducing the maximum bending moment capacity of the sheet pile, particularly with more flexible walls. The maximum axial force in the Bell Common and House of Commons car park walls shows very little dependence on the wall stiffness (Figure 10). The maximum difference in axial force occurs in the George Green Tunnel. In this case there is approximately a 15% change in maximum axial force over the range of wall stiffness analysed.

The shear force in the wall is directly related to the bending moment distribution. Hence the maximum shear force in the wall is also dependent on the wall stiffness (Figure 11).

4.2 Prop forces

The prop forces, both in the short and long term, calculated in the Bell Common and House of Commons car park analyses are nearly constant over the range of wall stiffness. The largest change in prop force is in prop level 6 in the House of Commons car park, where there is a 15% increase in prop force as wall stiffness increases.

For the George Green Tunnel the prop load exerted by the roof slab immediately after construction increases by a factor of two as the wall stiffness increases. This may be due to the rotational restraint provided by the roof slab at the top of the wall or the large initial horizontal stress at the top of the London Clay. In the long term however, the prop forces in the roof and road slabs are less sensitive to changes in wall stiffness.

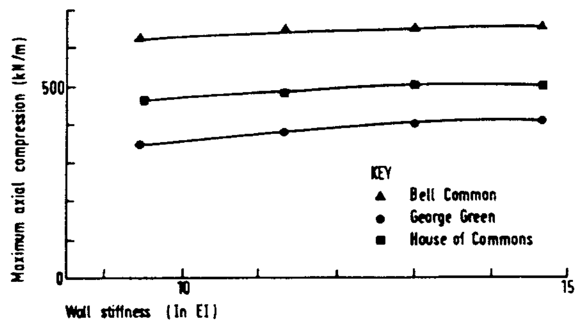


Figure 10 Maximum axial compressive force

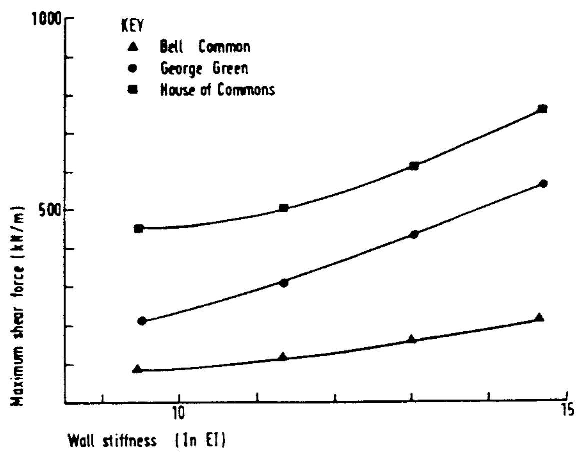


Figure 11 Maximum shear force

4.3 Ground movements behind wall

Profiles of the vertical and horizontal movement of the retained soil surface in the long term are shown in Figures 12 and 13 for the Bell Common Tunnel and the House of Commons car park. The results for the George Green tunnel show trends between these two and are not presented.

It is not necessarily the magnitude of the ground displacement that is important. A uniform movement in the vertical or horizontal direction will not cause damage to

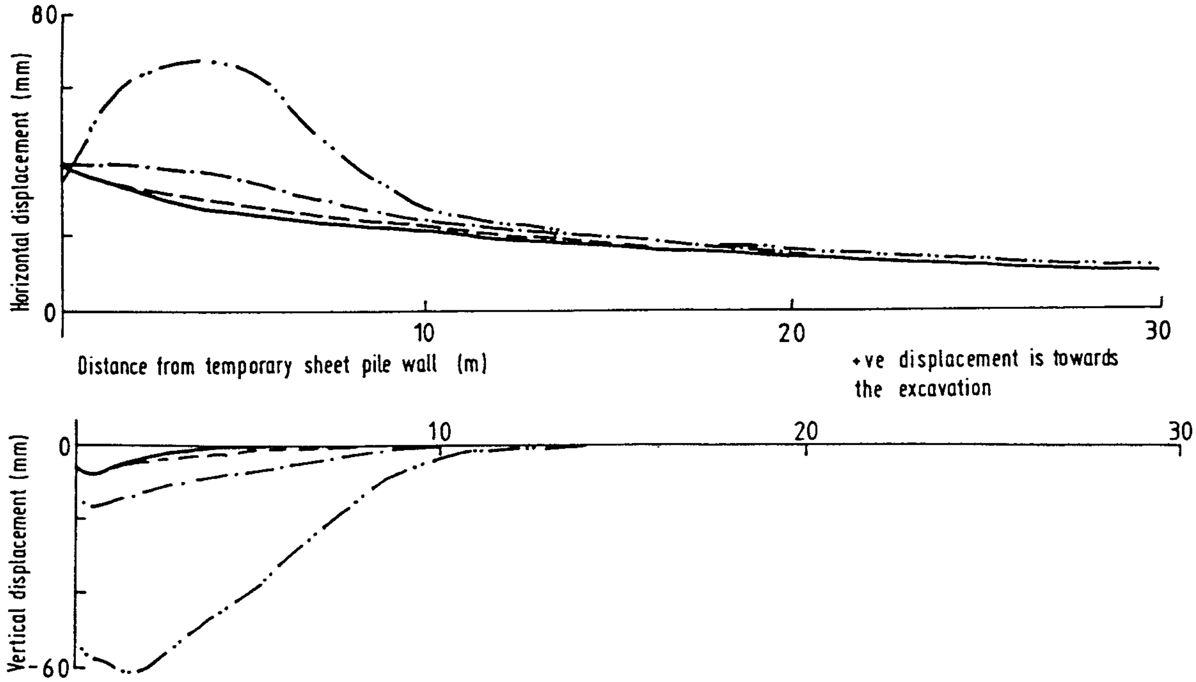


Figure 12 Ground surface movements at Bell common tunnel in the long term

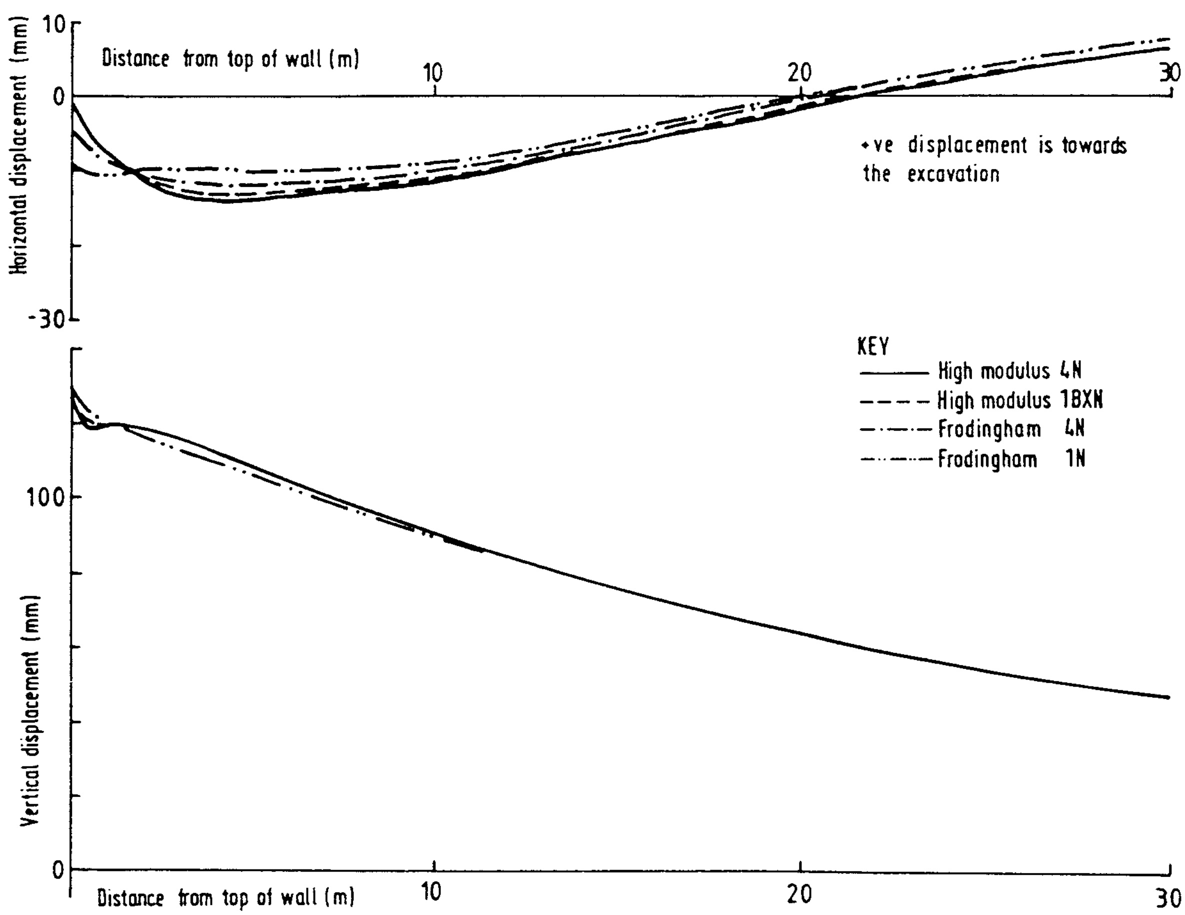


Figure 13 Ground surface movements at the House of Commons car park in the long term

structures. Similarly, a uniform rate of change of vertical displacement with respect to distance amounts to a simple rotation of the ground surface which again may not cause significant damage (but may result in instability or un-serviceability).

Whether or not the differences in displacement are significant depends on the proximity and sensitivity of other structures near the wall. A few general comments can be made.

For the Bell Common tunnel the Frodingham 1N pile is structurally inadequate (see Table 2) and impractical (for driving) and so can be ignored. The ground deflections due to the other three pile sections are similar and the differences likely to be insignificant. The small differences in both the horizontal and vertical displacement in the long term extend for less than 10m from the wall. Beyond this point there is a very little change due to wall stiffness.

The very small changes that occur in the vertical ground movement behind the House of Commons car park wall due to changes in wall stiffness extend for only about 8m from the wall. Large differences of the horizontal ground movements, however, occur near the wall due to changes in wall stiffness. Beyond about 4m from the wall the changes in horizontal movement are small and the rate of change of movement with respect to distance appears almost independent of wall stiffness.

The short term vertical and horizontal surface displacement profiles show trends similar to the long term movements and the above comments are equally valid.

5 COST OF WALLS

The relative cost of construction (Table 3) of concrete walls (both diaphragm and secant pile) and sheet pile walls at each of the three sites has been calculated by Dearle and Henderson Chartered Quantity Surveyors (1989). The basis of the cost calculations and a description of the alternatives is given in the appendix. The cost of each alternative has been expressed as a fraction of the cost of an "as formed" diaphragm wall at the relevant site.

Construction costs have been calculated for Frodingham 4N and Larssen 6 (138.7 kg/m) (selected as its stiffness, ln EI = 12.4, lies between that of the Frodingham 4N and the High Modulus 1BXN) sheet pile walls. Because the depth of penetration of the House of Commons car park walls is greater than 25m, Frodingham 4N piles were considered impractical and were not costed at this site. It should be remembered that 1m thick diaphragm walls and 1.2m secant pile walls have a bending stiffness similar to the High Modulus 4N section used in the analysis.

The cost comparisons indicate that the use of steel sheet piling in lieu of diaphragm walling can give savings in construction cost in the region of 25% to 40% for the equivalent finish. Substantial savings can also be made with sheet piling in lieu of secant pile walls.

6 CONCLUSION

Soil-structure interaction plays a dominant role in the behaviour of retaining walls supporting deep excavations. The maximum bending moment in the retaining walls analysed in this paper was greatly dependent on the stiffness of the wall. Approximately a five fold reduction in maximum bending moment occurred when the wall stiffness was reduced from that of a 1m concrete section to that of a Frodingham 1N sheet pile. This occurred for all cases considered and appeared independent of the number of prop levels. Increased soil and wall movements accompany this reduction in bending moment; however the magnitude of this increase depends on the degree of support provided to the wall. For multi-propped walls the increased movements of the wall and ground surface can be small and may often be insignificant when compared to the accuracy to which such movements can be predicted.

Significant cost savings can be made if flexible sheet pile walls are used to support deep excavations in stiff clay instead of stiffer diaphragm or secant pile walls. If the increased movements associated with more flexible walls can be tolerated or reduced by extra propping, such walls may provide a viable alternative. It may however be necessary to carry out a soil-structure interaction analysis to assess the structural forces applied to the wall and to obtain an indication of the effect on soil movements. In many situations such an analysis is often required in any case. Nevertheless the cost of such an analysis is much less than the savings gained by the results which can lead to a more economic solution.

7 ACKNOWLEDGEMENTS

This work described in this paper has resulted from a research project partly sponsored by British Steel General Steels. Mr Day is grateful to the Commonwealth Scholarship Commission for providing financial support.

8 APPENDIX

The cost calculations are based on:

- Prices at the first Quarter of 1989.
- The method of procurement would be by bills of quantities under the ICE Form of Contract.
- An assumed saving in contract period and consequent reduction in time related/preliminary items.
- Diaphragm wall is 1m thick with 1m x 1m guide walls and 2% reinforcement.
- Secant pile wall consists of 1180mm diameter bored piles at 1080mm centres.
- Grade 50A steel sheet piles.
- Installation of sheet piles is by two stage driving using high frequency vibrator and Serf Pilemaster techniques to maintain noise and vibration comparable to diaphragm walling operations.

The cost comparisons exclude the following:

- Obstructions located within the ground.
- Dewatering.
- Professional fees.
- Value Added Tax.
- Finance charges. (Reduction of interest payments would occur from savings in time with the use of the steel pile alternatives)

The following alternatives have been considered for each site:-

Diaphragm wall

- finished "as formed".
- with blockwork facing.
- with 150mm thick reinforced insitu concrete facing.

Secant pile wall

- Reinforcement at 100 kg/m^3
- UB 914x305x289 in lieu of reinforcement.

Steel sheet piles

- finished "as formed".
- with 140mm solid blockwork facing built fair.
- with corrosion protection coating (PC 2) - Shot blast cleaning to steel piles and one coat high-build

Table 3 Relative construction costs

	Type of wall construction				
	Diaphragm	Secant Piling		Sheet Piling	
		100 kg/m^3	UB914x305x289	Larssen 6	Frodingham 4N
BELL COMMON TUNNEL					
As formed	1.00	0.81	1.20	0.77	0.59
Blockwork facing	1.03	0.85	1.24	0.79	0.62
Concrete facing	1.05	0.87	1.25	N/A	N/A
Corrosion protection	N/A	N/A	N/A	0.83	0.64
Fire protection	N/A	N/A	N/A	0.85	0.67
GEORGE GREEN TUNNEL					
As formed	1.00	0.76	1.22	0.73	0.52
Blockwork facing	1.05	0.80	1.26	0.73	0.57
Concrete facing	1.07	0.83	1.29	N/A	N/A
Corrosion protection	N/A	N/A	N/A	0.80	0.57
Fire protection	N/A	N/A	N/A	0.83	0.61
HOUSE OF COMMONS CAR PARK					
As formed	1.00	0.81	1.24	0.77	—
Blockwork facing	1.05	0.85	1.29	0.82	—
Concrete facing	1.08	0.88	1.32	N/A	—
Corrosion protection	N/A	N/A	N/A	0.84	—
Fire protection	N/A	N/A	N/A	0.87	—

isocyanite cured coal tar epoxy pitch.

- with one hour fire protective coating - shot blast cleaning to steel piles and epoxy metallic zinc rich primer and intumescent spray paint.

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