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Ground and structure response to a hand driven decline in London clay

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ABSTRACT: Settlement monitoring of a basement structure and the ground beneath was carried out during construction of a shallow inclined hand excavated tunnel. The recorded volume loss was found to correlate with a prediction based on load factor. A mode of behaviour of the structure acting as a simple weightless beam interacting with the soil was observed, widening the settlement trough. Soil-structure interaction effects were observed to extend below foundation level.

1 PROJECT AND SITE DESCRIPTION

Brown & Root provided design and site supervision services to London Electricity for the construction of a cable tunnel between St Johns Wood and Back Hill in Central London. The route of the main tunnel follows Euston Road near the south-east corner of Regent's Park, and a short spur tunnel was required to provide a link to the Longford Street substation (Fig. 1).

The 77m spur includes a 40m long decline with an internal diameter of 2.44m, hand driven entirely through London Clay at a downward gradient of approximately 1:6, commencing at a shaft in the substation and passing beneath two structures. The first structure is the recently constructed No. 50 Triton Square development which has deep piled foundations between which the decline passes. The second is the location of a former office block development (316 – 336 Euston Road), where a basement car park structure remains, founded on shallow pad foundations (Fig. 2).

Precise levelling points were installed in the basements of both structures and in the ground above the tunnel. Monitoring was required in order to ensure that suitable mitigation measures for the structures could be taken if required as construction progressed. The concern in tunnelling under the basement car park was that induced settlements might lead to distortion of the base slab or differential vertical movement between slab and columns, causing cracking leading to water ingress. The monitoring also provided an opportunity to examine the movements induced by tunnelling at various depths in one location. Such a study has relevance for the design and risk assessment of other shallow inclined tunnels such as escalator barrels for metro station complexes.

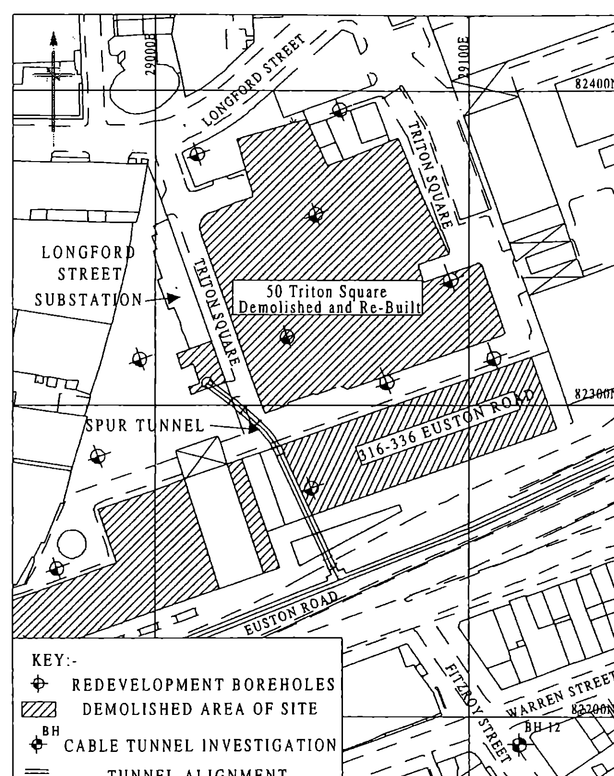

Figure 1. Site location plan.

Ground investigations for the cable tunnel alignment comprised open hole percussion boreholes with standard in situ and laboratory testing. These indicated that the London Clay is on average 25 metres thick beneath the site with the upper

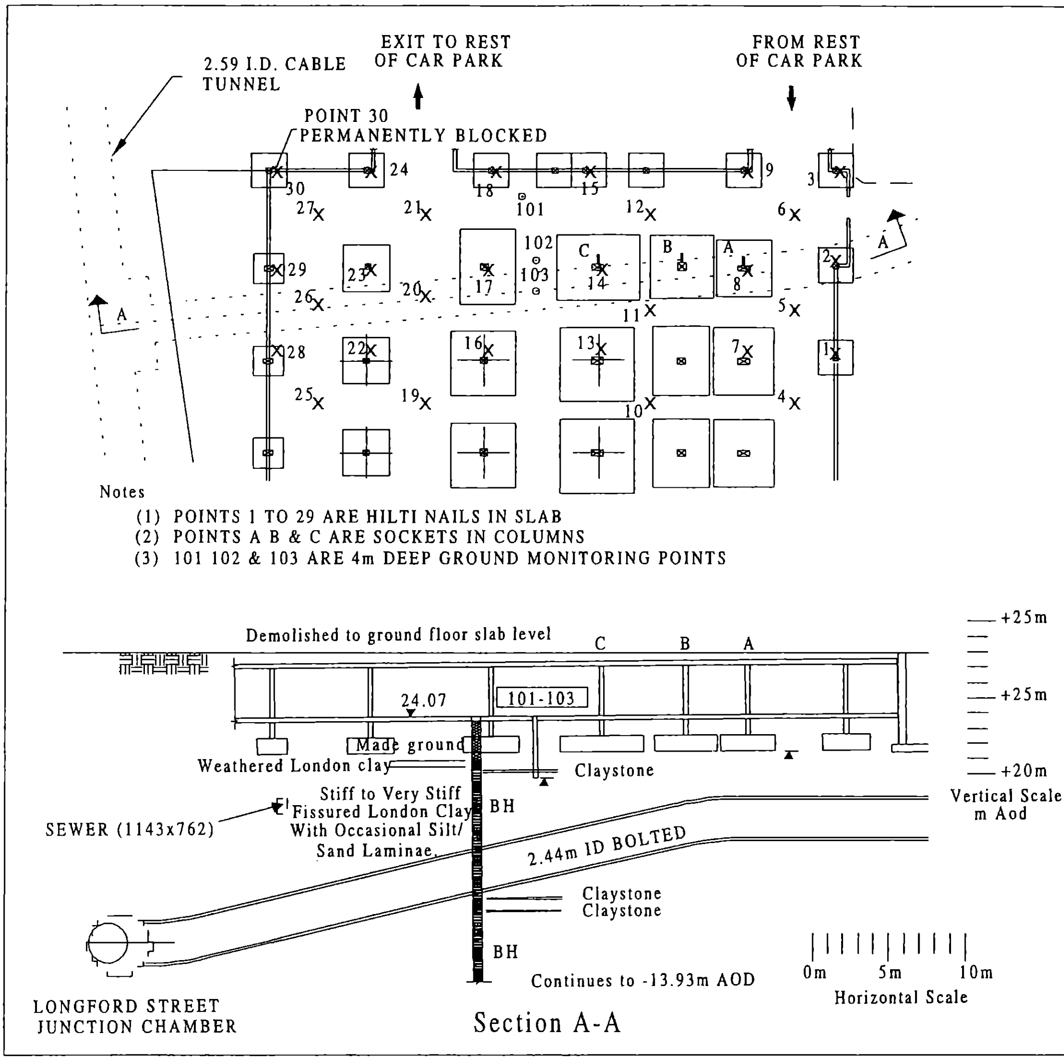

Figure 2. Plan and section through site showing monitoring layout.

boundary 3m below basement slab level. The clay was logged in the boreholes as a stiff to very stiff fissured laminated clay with sand and silt partings and bands up to 10mm thick, occasional calcareous mudstone nodules ("claystones") and lignite fragments at depth. The mottled clays of the Reading Formation immediately underlie the London Clay.

Average values of index properties for the London Clay at the tunnel depths were: moisture content – 27%; plastic limit - 25%; liquid limit - 71% and bulk density - 1.99 Mg/m^3.

In situ standard penetration tests (SPT) and unconsolidated undrained triaxial tests (UUT) on U100 tube samples suggest that at the tunnel depths, shear strengths in the clay vary from 70 – 130 kPa. According to Clayton (1995), shear strengths derived from the SPT test will approximate to the mass shear strength in fissured clays. Good correlation between UUT and SPT estimates of shear strength was observed.

The 316 – 336 Euston Road development was constructed in the mid-1960's and demolished to ground level in 1996. The original structure was partly of three storey construction (including the basement) and partly eight storey, rising to a total height of 29m above basement slab level. The demolished structure extended for 90 x 25 metres in plan, with continuity to the adjacent buildings at basement level. The axis level of the decline tunnel varies from 7m to 15m below the basement slab.

The basement structure that remains consists of reinforced concrete columns, of 530x460mm and 790x460mm sections at 6.1m centres, supporting a continuous 305mm structural slab at ground surface level. The columns are founded on discrete reinforced concrete pad footings typically 4 metres square in plan and 2.3m below basement slab level, designed to a net bearing pressure of about $20kN/m^2$. The basement floor slab was evidently cast separately, after the columns, with its own reinforcement mesh for crack control, not contiguous with the column reinforcement.

2 TUNNELLING METHOD

The spur tunnel was hand driven, as the short length and change in gradient made mechanised excavation infeasible. The face was typically vertical and benched, held open over its whole extent during excavation and then boarded up between shifts and at weekends to prevent instability. The lining was constructed immediately behind the face and the annulus grouted at most two rings behind.

Excavation took place over two shifts of ten hours each per day, five days per week, the remaining time being allowed for maintenance. The typical rate of progress was 5 rings of 0.75m length every 24 hours. Tunnelling commenced from the access shaft at the nearby substation and reached the top of the decline on 27th July. The advance of the tunnel resumed on 11th August, breaking into the main tunnel enlargement on 28th August.

3 MONITORING LAYOUT

The monitoring included precise levelling of surface settlement points on the basement floor slab and columns (Fig. 2). Levelling sockets were placed in three columns approximately over the tunnel centreline, with Hilti nails in the slab adjacent to indicate relative movement. A total of 30 No. Hilti nails were placed in the concrete slab within 7.5m either side of the tunnel centreline.

Three deep ground monitoring points were installed in the clay 4m below floor slab level, approximately 4.5m above the tunnel crown. The subsurface points consisted of a sleeved rod anchored at the base of a borehole within a concrete plug, the hole being back-filled with bentonite slurry.

J. Murphy & Sons (JMS), the tunnelling contractor, installed the surface monitoring points on 27th July. The ground investigation contractor installed the subsurface levelling rods on the 5th August.

The precise levelling was carried out by a JMS surveyor at the start of each day shift during tunnelling, and subsequently weekly for a further three months. The overall accuracy of individual readings, taking into account temperature effects and closure errors, is estimated to be ±0.5mm.

4 EMPIRICAL SETTLEMENT PREDICTIONS

The traditional design procedure for the assessment of the risk of damage due to tunnelling induced settlements involves calculation of "greenfield" settlement troughs and applying them to the structure, which is assumed to be "flexible". Such calculations would assume a normal distribution to describe the settlement profile.

Critical to such calculations is a prediction of the volume loss parameter (V_L), the ratio of the volume of the surface settlement trough (V_S) per metre run to the excavated face area. O'Reilly & New (1982) list a number of case histories that may be used as a basis for assigning a suitable value of V_L at design stage. As an alternative they state that, for tunnelling with or without a shield in stiff fissured London Clay, V_L ranges between 0.5 – 3%, usually in the range 1 – 2%.

Alternatively, Mair et al. (1981) demonstrated a relationship between V_L and load factor (LF) where LF is defined as the ratio of the working stability number (N) (after Broms & Bennermark 1967) to the stability number at collapse (N_{tc}). O'Reilly (1988) combined this work with that of Kimura & Mair (1979) in order to estimate V_L for a number of case histories in London Clay. Volume loss at Longford Street predicted by this approach is of the order of 0.5%. However, the conditions at this site fall at the lower limit of the range of test and field data upon which the relationship between LF and V_L was derived, leading to uncertainty in the choice of V_L.

In addition, a "trough width parameter" (i) is required, defined as the transverse distance from the point of maximum settlement to the point of inflexion of the settlement trough. Correlations between i and the depth to the tunnel have been proposed by Mair et al. (1993) among others.

Estimates of settlement (ω) could thus be made according to the following expressions relating ω, i and the transverse distance normal to the tunnel axis, y:

$$\omega_{max} = V_s/(2.5\, i) \qquad (1)$$

$$\omega(y) = \omega_{max} \exp(-(y^2/2i^2)) \qquad (2)$$

It is known that the presence of a structure may significantly modify the development of the surface settlement trough. Depending on the structure type and stiffness, the bending moments and strains in the structure may be predicted either by semi-empirical methods (e.g. Potts & Addenbrooke 1997) or by numerical analysis.

At Longford Street it was not clear exactly what modification would occur due to the overlying structures, in particular the contribution of the basement floor slab. It was not known whether the slab would act in combination with the columns and their footings, or whether it would interact directly with the underlying soil, independently of the remainder of the structure.

Table 1. Summary of settlement observations.

Location	Elevation	Max. settlement (ω_{max})	Trough width parameter (i)
	m AOD	mm	m
Slab	24.07	2.0	7.0 – 8.0
Column base	21.75	2.2	7.4 – 9.5
Sub-surface	20.00	3.3	5.0 – 6.0

5 RESULTS AND DISCUSSION

The recorded settlement observations are summarised in Table 1.

5.1 *Volume loss*

The results from the deep settlement points indicate a maximum short-term settlement above the tunnel centreline of approximately 3.3mm. A value of i of 5.0 – 6.0m at the level of the subsurface points was estimated from a plot of $\log_e(\omega)$ versus y^2 assuming a normal distribution. V_L was thus estimated to be 0.8% ±0.2%.

In order to compare the volume loss at Longford Street with the predictions of Mair et al. (1981) and O'Reilly (1988), a plot of V_L versus LF was constructed, in semi-log space in order to improve clarity at load factors below 0.5 (Fig. 3). Data points from field data held within Brown & Root's database and other published case histories (Macklin, 1999) are also shown.

The point representing the measured volume loss of 0.8% at Longford Street is shown falling close to a regression line drawn through the data points, and would thus appear to be a reasonable result.

5.2 *Subsurface Trough Width*

Figure 4 shows the settlements measured by the deep settlement points and on the slab at the same chainage, plotted with transverse distance from the tunnel centreline. The observed settlement of column C is also shown, together with the predicted greenfield settlement trough at the level of the deep settlement points, assuming ground level at column foundation level. The prediction assumes V_L = 0.8% and $i = k(z_0* - z*)$, where k is a constant, and z_0* and $z*$ are the depth from column foundation level to the tunnel centreline and to the observation point respectively. The prediction uses the relationships derived by Mair et al. (1993) for the calculation of subsurface settlement trough widths.

The measured subsurface settlements are less than those predicted. Assuming that the settlement profile was symmetrical about the tunnel centreline, a trough width of 5.0 – 6.0m was observed. This is

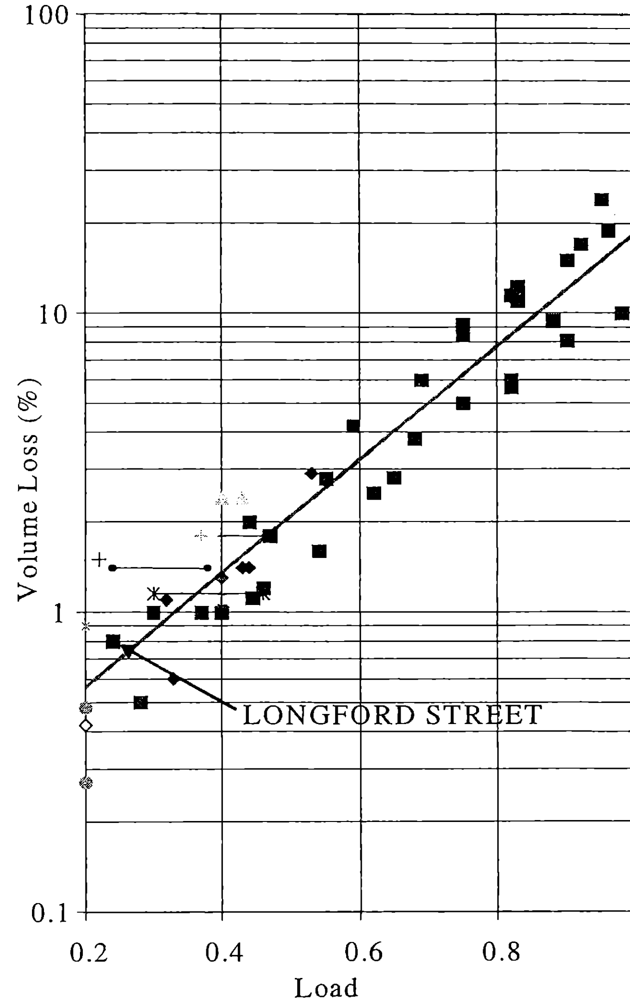

Figure 3. Volume loss – load factor relationship.

considerably wider than the predicted greenfield value of 2.6m. This effect is attributed to the interaction between the soil and the structure, the influence of which appears to extend at least 2m below foundation level.

5.3 *Structural response*

The structure was observed to respond to the passage of the tunnel by developing vertical displacements of the columns (Fig. 4) and longitudinal and transverse settlement troughs in the slab. A contour plot of the response of the slab immediately after the tunnel had completed its passage under the basement on 26th August is shown in Figure 5.

The maximum settlement of the columns was only about 50% of the settlement that would have been predicted based on the volume loss determined from the deep monitoring points (0.8%). Using Equation 1 and maximum column settlements a notional trough width at column foundation level was calculated, giving a range of i of 7.4m – 9.5m.

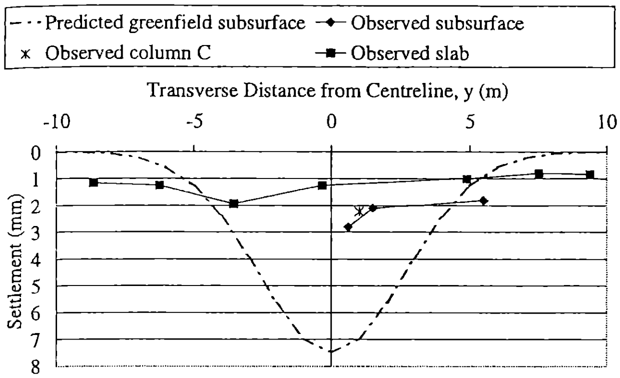

Figure 4. Predicted (V_L = 0.8%) and measured transverse settlements.

A trough width parameter of 7 – 8m is estimated from the settlement contours for the slab, compared with a predicted value of 5.1m based on ground level at slab level. The slab settlements show a general decrease with increasing tunnel chainage and depth. In addition the slab settlements are locally higher close to the columns, suggesting that a structural connection between the slab and columns was maintained. The line of maximum slab settlement appears to be offset by 2 – 3m to the left of the tunnel centreline. This may be due in part to the effect of the unusually large settlement at the column to the left of column A.

Thus the structure is seen to exhibit wider settlement troughs and smaller maximum settlements than greenfield predictions suggested. The reason for this is believed to be an interaction between the structure and ground. The mechanism for this interaction is postulated to be due to the development of shear stresses beneath the column foundations near to the clay-gravel interface, reducing or restraining the horizontal component of displacement, causing movements to be predominantly vertical and hence more widespread.

Such interactions were modelled numerically by Potts & Addenbrooke (1997), representing a building as a rough weightless simple beam above a plane strain tunnel heading in a non-linear elastic-plastic soil. They found that the settlement trough at foundation level tended to become wider and flatter with increasing flexural and axial relative stiffness. They also found that the greater the relative axial stiffness of the structure the more restricted the horizontal movements, while relative flexural stiffness had little effect on horizontal displacements.

At Longford Street the structure consists of continuous slabs at basement and ground levels, both of which enclose the columns and, because tensile strains are likely to be small, may be considered to be acting in both tension and compression. The settlement observations suggest a structural connection between slab and columns. Thus it is

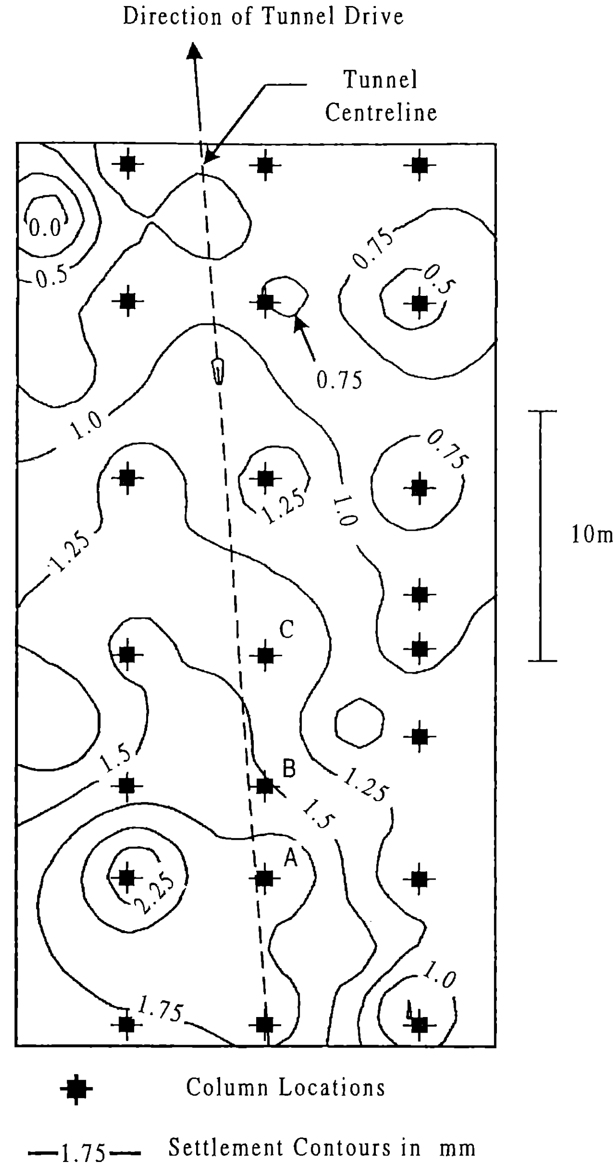

Figure 5. Contour plot of slab settlements, 26th August 1997.

feasible for the structure to behave as a simple beam, with the bending section consisting just of the two slabs separated by the storey height (3.9m).

Making this assumption and applying it to the approach of Potts & Addenbrooke, estimates of relative bending stiffness ρ^* = 0.47 and relative axial stiffness α^* = 12.7 are obtained. Based on their results for a wide building (their Fig 6), the i/z_0 ratio is predicted to be around 1.0. This correlates well with the estimated value of i from the measured column displacements (i=7.4m), based on an 'effective' ground level at column foundation level (i/z_0*=1.05). This appears to confirm the work of Potts & Addenbrooke given that their assumption of a weightless building is appropriate as the overlying structure has recently been demolished.

The locally greater slab settlements close to the columns indicates structural connection between the slab and columns in the vertical direction. The consequently lower slab settlements away from the columns leads to the postulation of another aspect of the slab settlement behaviour, where the slab is tending to follow the settlements of the ground immediately beneath it. At this level, the settlement trough would be expected to be wider than at column foundation level, due to the widening of the trough further from the tunnel.

Thus the slab behaviour may be summarised as consisting firstly of constraint to follow the "global" structure response to the tunnelling. In this case this is manifested as a settlement imposed on the slab at the column locations. The second element of the slab behaviour is a "local" behaviour between columns, where the settlements are lower. This may be explained in terms of compatibility of displacements, with the slab interacting directly with the ground beneath it.

5.4 *Time dependent effects*

In the medium term, 3 – 4 months after construction, the settlements of the slab and columns close to the tunnel centreline increased to maximum values of 2.9mm and 4.0mm respectively. The deep settlement points did not show significant movement.

6 CONCLUSIONS

The estimate of the volume loss observed in this case history correlates well with other field data and laboratory test data when considered in terms of load factor.

The structure was observed to interact with the tunnelling-induced ground movements causing widening of the settlement trough at slab and column foundation level. However at the level of the deep settlement points the trough width was not greenfield as expected but was also considerably wider. This suggests that interaction effects extend significantly below foundation level.

Based on the trough width values estimated for the slab and assuming similar behaviour for the columns on account of the high degree of structural connection, and taking effective ground level at column foundation level, the global structure behaviour compared favourably with the numerical study by Potts & Addenbrooke (1997). This is considered to be a reasonable result given that the assumption of a weightless beam is appropriate in these circumstances.

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