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Observations of deformations created in existing tunnels by adjacent and cross cutting excavations

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ABSTRACT: The excavation of NATM and conventional tunnels for Contract 104 - London Bridge Station of the Jubilee Line Extension has involved the adjacent and undercutting excavation of both existing cast iron lined and new NATM tunnels. In all these locations, monitoring of deformation, combined in many cases with stress, was undertaken. Four such locations and the results of the monitoring are described, with particular attention to the response of a recent NATM and two existing cast iron lined active Northern Line running tunnels, to undercutting by two Jubilee Line NATM running tunnels. The results of monitoring are compared with simple techniques used to estimate subsurface vertical settlement above tunnels and deformations due to adjacent excavations using continuum methods. The results show that vertical settlement of the overlying tunnels can be reasonably predicted using empirical Gaussian settlement estimation techniques, with a correction factor for depth applied to the trough width. Deformations due to adjacent excavations when predicted using closed form solutions for continuum and finite element models are shown to be reasonably accurate. Finally it is concluded that the methods of estimation described can economically be used for a range of values, effectively giving a sensitivity analysis. If this information is combined with careful construction procedures and detailed monitoring then in many cases close proximity tunnels, can be safely constructed.

1 INTRODUCTION

1.1 Background

Closely spaced tunnels are a common configuration of most underground railway systems. As the systems are enlarged and added to, there is a necessity to construct tunnels, often of a larger size, in close proximity to the existing tunnels. In many cases, these must remain operable, placing limits on allowable deformations. This problem is being encountered more frequently as existing systems are updated to deal with increasing passenger numbers, and is reflected in recent studies and reports on the effects of multiple tunnels.

Most of these investigations describe 2-dimensional finite element analyses (Perri, 1994; Wang and Chang 1992; Kanzki et al, 1992) which in some cases, provided 'reasonably accurate estimations' of deformations and, occasionally, stresses when compared to actual monitoring (Saitoh et al, 1994 and Nishimura et al 1994). In many cases it was noted, however that in order to obtain a very good approximation of the deformations, a 3- dimensional finite element model was required due to complicating factors such as:

i) driving sequences and rates,
ii) changes in primary ground stresses and stiffness, and
iii) the type of excavation and lining used.

However, the cost of using such models can generally be considered prohibitive, particularly if the stiffness parameters for the ground and structure were not of very high quality.

It is the intention of this paper to examine the accuracy of two methods of estimating displacement due to undercutting or adjacent tunnels.

1.2 Close proximity tunnels - Contract 104 - LOB

The tunnelling works on Contract 104 - London Bridge Station are shown in Figure 1, with the areas of most interest in the context of this paper being the:

i) JLE west and east bound running and station tunnels using NATM, undercutting the Northern Line existing and new tunnels
ii) A NATM trial comprising 11.3 and 5.35 m diameter tunnels, with adjacent JLE tunnels
iii) Northern Line works consisting of a new station tunnel being constructed adjacent to the existing cast iron segmental station tunnels and cross passages.

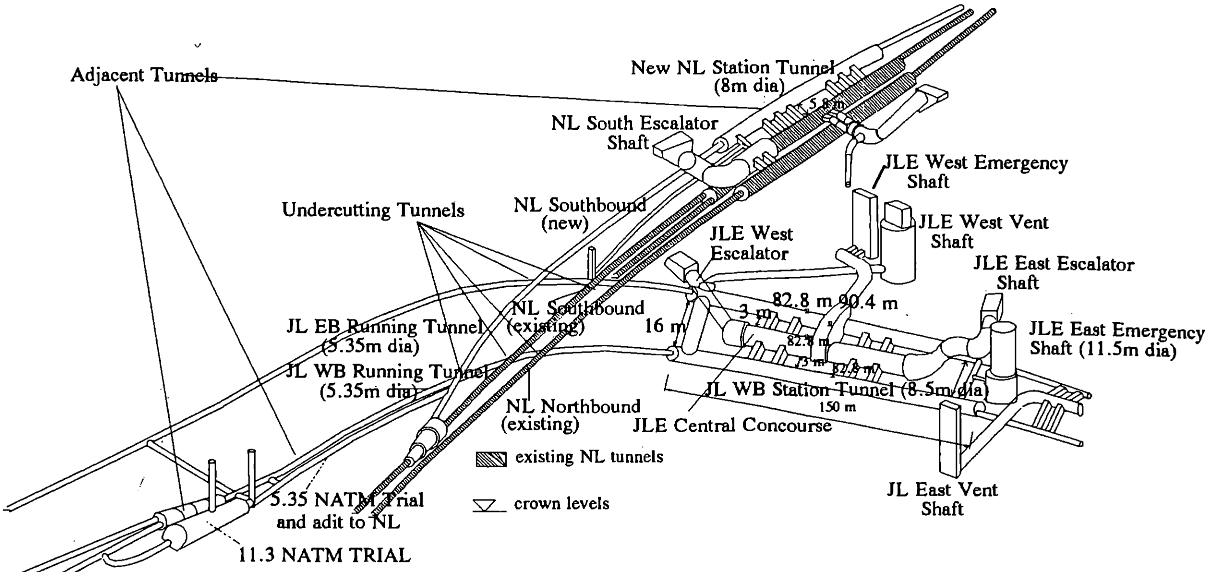

Figure 1. A 3 dimensional plan of tunnel construction for C104 - London Bridge Station

1.3 Geology and ground profile

A simplified geology for the area and a range of possible values are given in Figure 2. Variations of strata levels at the various locations may vary by 0.5 - 1.5 m.

2 DEFORMATION ESTIMATION

2.1 Settlement due to undercutting

Settlement above tunnels has been calculated using curve fitting to a Gaussian distribution as described by Rankin (1988). In this case, the maximum vertical settlements on a plane above a tunnel of radius r, due to a given volume of face loss (V_l) is given by:

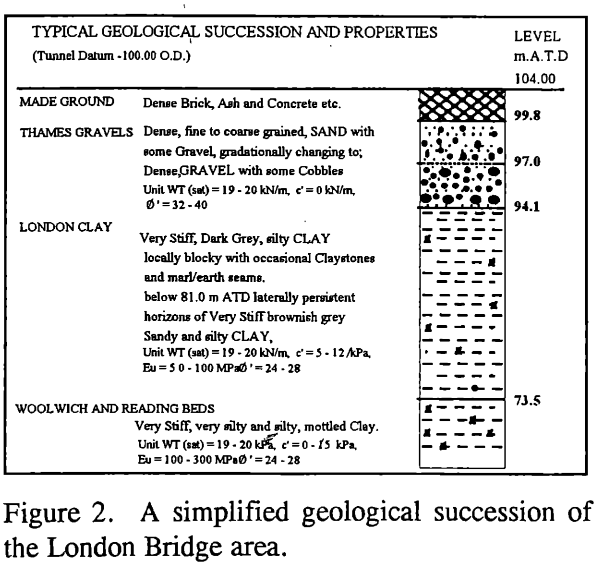

Figure 2. A simplified geological succession of the London Bridge area.

$$W_{max} = 0.0125\ V_l\, r^2 / i_y \tag{1}$$

$$\text{and, } i_y = K\, H_0 \tag{2}$$

where i_y describes the trough width as a function of the depth to axis of the tunnel (H_0) and K, a material constant typically taken as 0.5 for London Clay. This technique has been shown to give reasonable estimates of surface settlements, but has a tendency to over predict the magnitude of subsurface settlements and under predict their lateral extent. Mair et al (1993) proposed that the parameter K increases with depth, providing a wider zone of settlement (trough width) closer to the source of the volume loss. Values of K for clays against depth being of the proposed form as shown in Figure 3.

2.2 Deformations due to adjacent tunnelling

Deformation and stresses developed in single and closely adjacent tunnels may be estimated using the simplified approaches based on the numerical results of continuum model and Finite Element

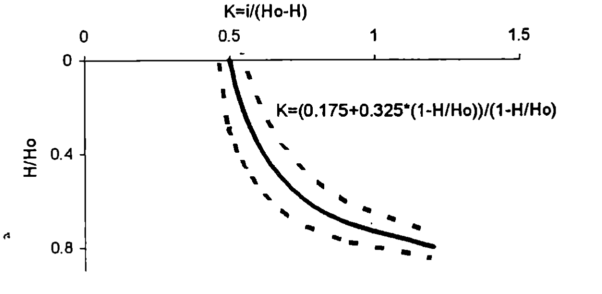

Figure 3. Values of K with depth for subsurface settlement prediction in clay (after Mair et al, 1993).

solutions as described by Duddeck and Erdman (1982) and (1988), and Soliman et al (1993).

Firstly, this approach requires that values (deformation or stresses) for a single tunnel are calculated (using an approach such as those of Muir-Wood, 1978 or Duddeck and Erdmann, 1985). Alternatively, it is proposed here that monitored values may also be used, if available (they should approximate to those calculated).

The next stage of the analysis is to determine the changes in deformation and stress in the new and existing tunnels from prepared numerical solutions based on linear FE analyses by Soliman et al (1993). An example of this calculation process to determine deformation is given below for a typical shotcrete running tunnel assumed to be sufficiently circular.

Stage 1 - single tunnel deformation

shotcrete lining thickness	$t=$	150 mm
radius of tunnel	$R=$	2.6 m
depth to tunnel	$H=$	28 m
shotcrete modulus	$E_c=$	10 - 25 GPa
London clay	$E_s=$	0.1 - .05 GPa

Duddeck and Erdmann (1985) then require that the factor α (the stiffness ratio of the ground / lining) is calculated as given by

$$\alpha = E_s R^3 / E_c I \qquad (3)$$

where $I = t^3/12$

This value can then be used to obtain a value of radial displacement, u, based on the relation

$$\max \hat{u} = \sigma_v \, 0.5 \, (R^4/E_c I) / (12+1.03\alpha) \qquad (4)$$

This relation for a range of possible stiffness is shown in Figure 4 (after Duddeck and Erdman, 1985).

Assuming properties for the shotcrete of $E_c = 25 \; 10^3$ MN/m^2 and a range of $E_s = 50$ to 100 MN/m^2 for the London Clay this would give radial displacements in the range 6 to 13 mm for a typical running tunnel.

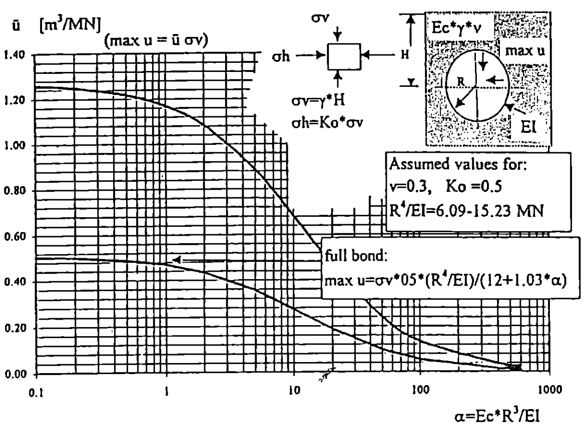

Figure 4. Radial displacements, U, of a single tunnel against stiffness ratio α.

Stage 2 - deformation due to adjacent tunnels

With the calculated or measured value of original deformation, the increase in deformation or stress can be estimated based on the solutions proposed by Soliman et al (1993) for tunnels of separation R (one radius) and 0.5 R. Extrapolating this beyond R enables a reasonable estimate of the increase or decrease in deformation to be made using a simple ratio:

$$\Psi = W_{a,\,i} \,/\, W^{o}_{i,\,a} \qquad (5)$$

Where $W^{o}_{i,\,a}$ is the value of deformation for a single tunnel. It should be noted that this original value if estimated using the approach above, considers the tunnel as two sections symmetrically about the vertical, an inner side near the new tunnel (i) and the outer (a).

The relations for change in original deformation horizontally (the inner (W_i) and further (W_a) away sides) and vertically (W_c) of the existing tunnel due to adjacent excavation are shown in Figures 5 and 6.

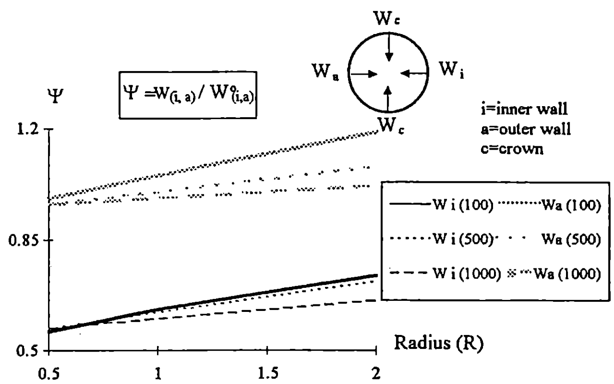

Figure 5. Radial sidewall displacements W_i and W_a as a function of α (for $\alpha = 100, 500, 1000$).

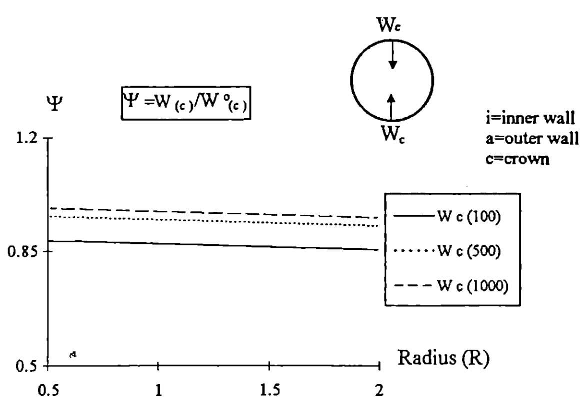

Figure 6. Crown displacements W_c as a function of α (for $\alpha = 100, 500, 1000$).

The approach, described above as stage 1 , using continuum model solutions presented by Duddeck and Erdmann (1985) has been shown to provide a reasonably robust method of rapidly checking lining design stresses, obtained from more complex analyses, (Kimmance et al, 1996). It is considered reasonable, therefore, to expect the extension of this method described in stage 2 to give reasonable predictions for stress and deformation.

3 CASE STUDIES

3.1 Adjacent NATM tunnels

The first tunnels to be excavated on contract 104 were the NATM Trial 5.35 and 11.3 m diameter tunnels, which were extensively instrumented for deformation and stress (Kimmance and Allen, 1996). The lining thicknesses of these tunnels were 150 mm and 300 mm for the 5.35m and 11.3 m diameter tunnels respectively. Depth to the axis level of both tunnels was approximately 28 m. At a later date these tunnels were influenced by adjacent excavations The geometry of these respective cases is shown in Figure 7 (a) and (b)

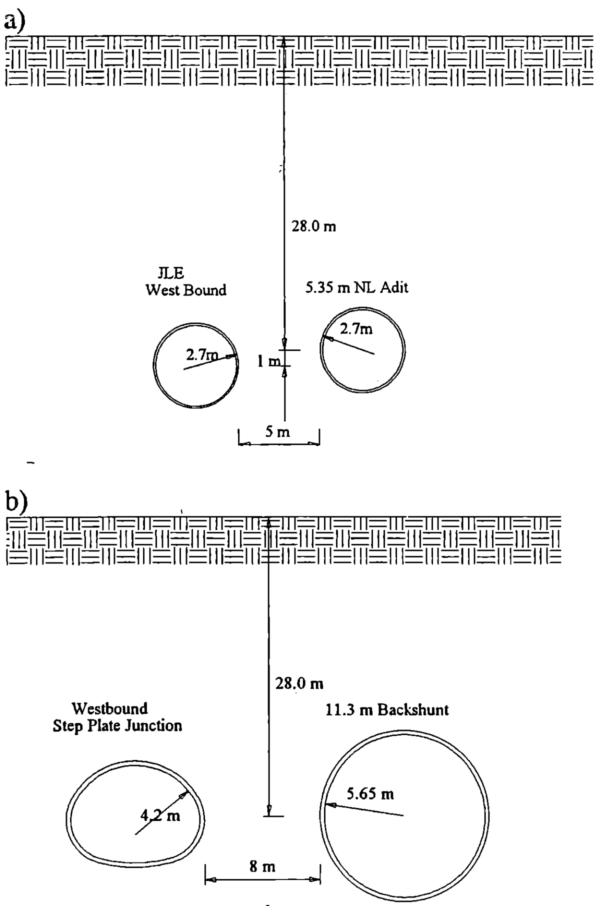

Figure 7. Section of NATM tunnels excavated a) adjacent to the 5.35m running tunnel and b) the 11.3 m tunnel.

Adopting the method described in Section 2.2, the predicted deformations of the 5.35 and 11.3 m diameter tunnels to adjacent construction have been estimated. Assumed values of E_s =0.1 - 0.05 GPa for the London Clay and for the shotcrete (more confident value due to frequent testing), E_c = 25 GPa with thicknesses of shotcrete t = 150 and 300 mm respectively. Stage 1 of the calculation was supplemented with observed values of displacement. These values and those predicted and observed due to the adjacent excavation are given in Table 1.

Table 1. Adjacent NATM tunnels predictions and observations of deformations.

	Single tunnel values		Adjacent excavation	
Units (mm)	Predicted	Observed	Predicted	Observed
5.35 m NATM				
Horizontal	6 to 12.9	3 to 5	5	4
vertical	- 6 to 12.9	- 4 to 6	-5.3	-5
11.3 m NATM				
Horizontal	14.8 to 28	8 to 13	12.2	5 to 8
vertical	-14.8 to 28	- 7 to 9	-14	- 9 to 12

The calculated values of deformation for a single tunnel were used in the estimation of the deformation due to the second tunnel. Only the upper strength value of London Clay was used in the deformation due to adjacent tunnelling as, these produced a closer estimation to the observations at the single tunnel stage. This is reasonable considering the relatively undisturbed and very stiff nature of the London Clay prior to excavation.

3.2. Adjacent cast iron segmental tunnels

The JLE works at London Bridge also include works for the Northern Line safety measures programme. This involved the creation of a new station and central concourse tunnel arrangement, requiring the excavation of a new 8 m diameter station tunnel parallel to the existing southbound station tunnel (7 m dia., in cast iron) as shown in Figure 8a. Assumed properties for the old cast iron lining were a stiffness of 170 GN/m^2 and an effective thickness of 0.11 m. The same London Clay properties were adopted as used above. No data was available regarding deformations of the first tunnel, and it was therefore necessary to adopt stages 1 and 2 of the analysis described above. These predicted values are compared against crown levels and horizontal deformation measurements using a tape extensometer in Table 2. Crown levels were used due to concrete and track beds in the invert and haunches of the tunnel.

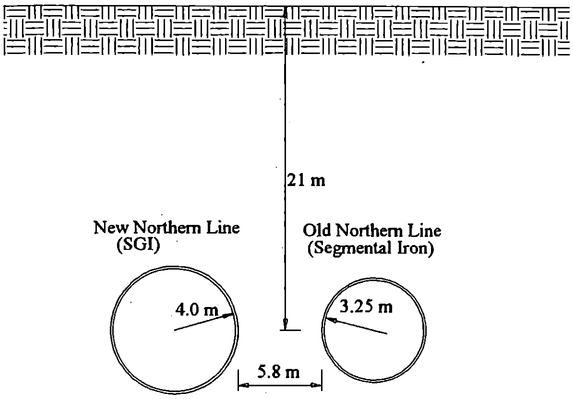

Figure 8a. Adjacent Northern Line station tunnels.

Table 2. Deformations of the existing NL south bound station tunnel.

Units (mm)	Single Tunnel	Adjacent Excavation	
	Predicted	Predicted	Measured
Northern Line			
Horizontal	6 to 12	5 to 11	4 to 6
Vertical	-6 to -12	-5.5 to -10	-5 to -7

As might be expected for an area where so many excavations have historically take place the best estimations were those obtained using lower bound values for the London Clay.

3.3 Crosscutting tunnels

The JLE West and Eastbound running tunnels passed under the three existing tunnels of the New Northern Line NATM tunnel and the existing NL southbound and northbound running tunnels. The angle of intersection was approximately 40 degrees (see Figure 8b) from the perpendicular, with the JLE Westbound running tunnel being the first tunnel (and with the most clearance) to proceed.

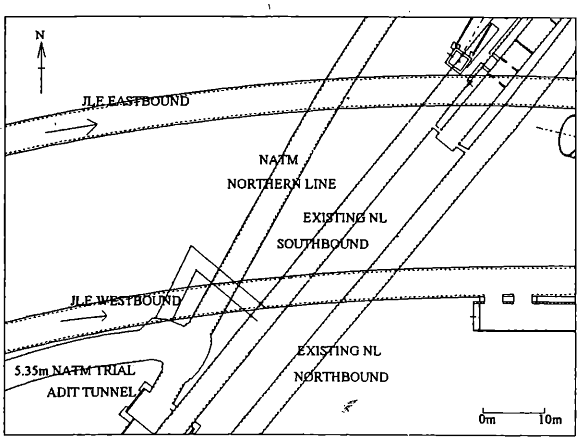

Figure 8b. Plan of undercutting tunnels

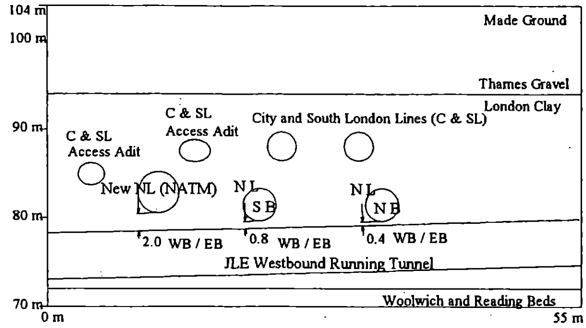

Figure 9. Cross section of undercutting tunnels

Instrumentation comprising electrolevels, tape extensometers, precise levelling and stress cells in shotcrete linings were placed in the undercut tunnels. The primary aim being to detect trends of deformation which would, if unchecked, affect the safe running of trains through them.

Prior to construction, the following precautions were taken in the older and closer cast iron lined Northern Line tunnels:

1) The lining bolts were tightened, and where rings were capable of independent movement invert concrete was reinforced to provide a single longitudinal unit.

2) Additional support equipment was prepared as a contingency.

3) The undercutting Westbound JLE tunnel used a minimum thickness of shotcrete of 150 mm with 1m between lattice girders. For the JLE Eastbound, at a level approximately 1 m higher at axis level, the shotcrete thickness was increased to 200 mm with lattice girders at 0.5 m centres.

4) Excavation was on a 24 hour basis with no breaks whilst in the zone influencing the overlying tunnels.

Results of the monitoring and observations made were that:

1) No visible distress of the lining was observed

2) That settlement of the lining occured, with some skew across the lining due to the angle of incidence of the undercutting linings.

3) Movement of the linings were first detected when the excavated face was laterally 4 - 8 m away.

Predicted and observed maximum settlements and deformations of the existing tunnels are given in Table 3.

It should be noted in these analyses that the values adopted for K needed to be in the range of 0.7 to 0.85, with an volume loss value of 1.5 % assumed from previous monitoring of the advancing tunnels. These values of K lie at the extreme of the relation developed by Mair et al (1993), i.e. the tunnels were almost touching, and as a result the predicted values could be prone to greater inaccuracy than normal. However the results were reasonable.

Table 3. Predictions and observations for undercut tunnels.

Units (mm)	JLE Westbound Predicted	JLE Westbound Actual	JLE Eastbound Predicted	JLE Eastbound Actual
NL (NATM)				
deformation:				
horizontal	N/A	5		7
vertical	N/A	7		
Settlement	-14	-9	-14	-10.5
NL Southbound				
deformation:				
horizontal	N/A	4		5
vertical	N/A	8		7
Settlement	-17	-9 **	-17	-20
NL Northbound				
deformation:				
horizontal	N/A	4		5
vertical	N/A	2		8
Settlement	-18	-15	18	-14
** Only recorded for 1 month after event				

4 CONCLUSIONS

It has been shown in the cases studied that the results using the described simple estimation techniques for settlement and deformation have been satisfactorily close to those observed in practice. The ease with which the calculations upon which these estimates are made may be carried out ideally suits the techniques to sensitivity analyses.

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