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Tunnelling in soft ground in the UK

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SYNOPSIS: Tunnelling is one of the major construction activities in the UK and will continue to be prominent within UK Civil Engineering activities as major transit schemes are developed. Many different tunnelling techniques and lining types are used depending on the type of ground encountered, which in the UK can be quite varied. Design related to tunnelling projects is mainly empirical. Monitoring of tunnelling projects has provided valuable insights into the ground response enabling improvements to design practice to be made.

1 CONSTRUCTION METHODS

1.1 Tunnel construction

The ground conditions encountered in the UK are highly variable reflecting the geological history of the country. The soils encountered include competent soils such as London Clay ("ideal" for tunnel construction), soft alluvium, water bearing sands and gravels, and glacial tills. It is not uncommon for tunnelling projects to encounter a wide range of ground conditions. The variable ground conditions and the different purposes for which tunnels are constructed means that many excavation techniques need to be available.

Hand excavation without a shield is possible in situations where there are good ground conditions, for example grouted gravels and stiff, competent clays, where water is not a problem. It is a rather special tunnelling technique and tends to be used for short drives and where the use of a shield would be impractical. It is a method used by London Underground Ltd (LUL) for construction of short access or link tunnels which are often threaded around existing tunnels or other obstacles. A recent example is the construction of the new escalator access and platform tunnels as part of the Angel Station Redevelopment (Moriarty and Cooper, 1991). The project included tunnels ranging in size from 3.85 m to 9.5 m internal diameter. An interesting feature of this particular project was the use of pilot tunnel construction where the final tunnel diameter was significantly larger than 4m. It is a method of construction often used by LUL for this type of project. It was observed that the volume loss during excavation of the enlargement from the 4m pilot tunnel to the final diameter of 6 - 9m for the concourse or escalator tunnels was considerably lower than for the excavation of the pilot and other tunnels constructed in a single pass to final diameter (Mair, 1993). The reason for this can be seen by considering Fig. 1. Ground movements result from excavation of a heading with soil yield into the tunnel face. During the enlargement there is considerable extra support provided by the pilot tunnel which acts a stiff dowel in the face being excavated. Shear stresses can develop along the pilot tunnel which then limit the potential for ground movements to occur.

Generally, it is preferred to use a shield during excavation and this may be combined with hand excavation for small diameter sewer tunnels where a machine might be impractical or uneconomic for the particular length of tunnel involved. For this method of construction, it is likely that a segmental lining will be used.

Machine excavation without a shield is unusual except where shotcrete is used for the lining. This "NATM" type of tunnelling (more generally referred to as Shotcrete Support System, SSS), which by definition does not utilise a shield, is likely to become more common in the UK. Short lengths of trial tunnel were excavated recently in London Clay near Heathrow airport to demonstrate the viability of the system and SSS is planned for use on the Heathrow Express and Jubilee Line Extension (JLE) projects, particularly for the complex station tunnels. Also, it is likely that SSS would feature in station tunnels for the proposed CrossRail project.

Machine excavation with an open face and with a shield is used in situations where there are good ground conditions and where there is not a major water hazard. The degree of complexity of the tunnel machine will be related to the length of tunnel. The system can be used in conjunction with compressed air to control groundwater as was the case for the new rail tunnels to Stansted.

Full face slurry support machines are not very common due to the often variable ground conditions, though they may feature with some pipe jacking systems for tunnels of the order of 1 - 4m in diameter. Earth pressure balance (EPB) machines are increasingly being used. EPB machines were used for construction of drainage tunnels beneath the Royal Docks in London (Ferguson et al, 1990) and similar machines are likely to be used for sections of the Jubilee Line Extension which is about to commence in London (tunnel diameter typically 4.5m ID).

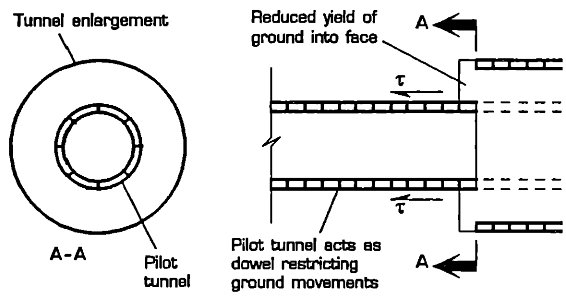


Figure 1. Effect of pilot tunnel in reducing ground movements (Mair, 1993).

1.2 Tunnel linings

The two main sources of information and guidance on UK practice are Craig and Muir Wood (1978) who describe a wide range of lining types and discuss their general use, and BS 6164 (1990) which considers safety aspects and describes temporary and initial ground support (though in many cases this will be the same as the final support). Table 1 summarises the preferred lining type for different soft ground conditions.

In the UK cast iron linings are now only used in the form of spheroidal graphite (SG) iron. This is a strong ductile iron lining and used where high stresses may exist either due to poor ground conditions or where the construction is complex, for example due to the presence of openings or some nearby construction. For long lengths of relatively straightforward tunnels, concrete linings are normal. Expanded segmental linings can be used in competent clays and high tunnelling rates can be achieved. In poorer ground, bolted linings are used, the type depending on the size of tunnel and potential loading. These linings are slower to install and require grouting. Bolted steel linings are relatively rare. Shotcrete has not been a major lining type for tunnels in soft ground but this is likely to change in the near future.

Table 1. Preferred form of lining for a variety of soft ground conditions (after Craig and Muir Wood, 1978).

	BC	SBC	EC	PJ	BCI	ECI	SC	BS
Drift above watertable	*	*		*	*			
Drift below watertable	*			*	*			
Silts and clays	*	*		*	*			
Very soft clays				*	*			*
Stiff fissured clays	*	*	*			*	*	

Key:
- BC Bolted concrete
- SBC Smooth bore concrete
- EC Expanded concrete
- PJ Pipe jacking
- BCI Bolted cast iron
- ECI Expanded cast iron
- SC Sprayed concrete
- BS Bolted steel

2 STRESSES ON TUNNEL LININGS

Tunnel construction results in complex stress changes and movements around a tunnel face. It is a truly 3 dimensional problem and very difficult to analyse due to the major influence of construction details (see Fig. 2). It is usual, therefore, to make simplifications when analysing tunnel construction with a view to determining lining loads. In most analyses, the lining is assumed to be "wished into place" i.e. it is installed in an elastic or elasto-plastic continuum without any ground movements developing. Clearly, this is a highly idealised simulation of the tunnel construction process. However, this continuum approach is the basis of the analyses presented by Muir Wood (1975) and Curtis (1976) which are often used in the UK at least for checking designs. A key element to the calculation is the initial ground stress profile, in particular the lateral stress. It is here that considerable judgement is required in deciding whether to use the in situ values of lateral stress prior to tunnel construction or some other modified value to take into account the stress relaxation at the tunnel face. In assessing the loads acting on the lining it is usual to consider a range of possible combinations of vertical and horizontal ground stresses to determine the maximum induced hoop stresses and bending moments. These calculations will almost certainly involve the possibility of the full overburden stress acting on the tunnel although it may be more critical to consider the lowest likely percentage of full overburden that may act.

For particularly complex situations, it is possible to undertake finite element analyses. However, great care must be taken to model realistically not only the tunnel construction and lining installation but also the detailed behaviour of the soil. Such calculations can also be used to investigate long term changes due to pore pressure redistribution following tunnel construction. In this case it is important to include a representative value for lining permeability (O'Reilly et al, 1991).

The parameters required for lining stress calculations can be very basic if a simple continuum type of analysis is used. A knowledge of in situ stress is required by some designers. This is determined from the bulk unit weight and an estimate of the coefficient K_o. However, in reality it is the choice of the value of the coefficient (usually between about 0.5 and 0.8) which determines the design calculation; this is not in practice related to K_o. Elasticity parameters are required for the lining and soil; for the latter, often some judgement will be required to select appropriate values for Young's modulus and Poisson's ratio.

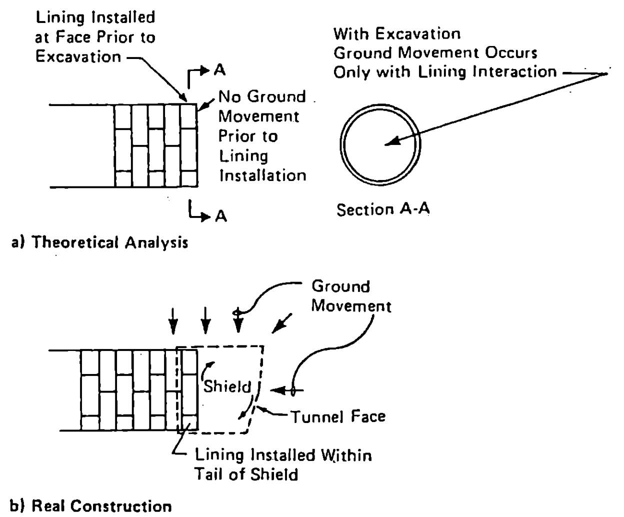


Figure 2. Comparison between theoretical analysis and real construction (after Hansmire, 1984).

3 GROUND MOVEMENTS

There has been considerable emphasis in recent years in assessing deformations due to tunnel construction. The major impetus came from the Transport Research Laboratory who both sponsored research projects and initiated an extensive programme of monitoring to improve the general understanding of the development of movements during tunnelling. The various studies proved to be an excellent sequel to the pioneering monitoring of tunnels by the Building Research Establishment.

The main focus of the research was to assess the magnitude of ground loss due to tunnelling under different conditions and to develop empirical rules for predicting the surface settlement trough. It followed from a need to have an improved understanding of the effects of tunnelling and the impact on shallow buried services. Volume (or ground) loss, V_L, is the ratio of the volume of surface settlement trough, V_s, to the volume of tunnel excavation. Thus, for a tunnel of diameter D,

$$V_L = \frac{4V_s}{\pi D^2} \tag{1}$$

Data from monitoring of tunnelling projects in the UK were correlated by O'Reilly and New (1982) and a review of this work and recent advances is presented by New and O'Reilly (1991). They concluded that the form of the settlement trough (when no longer affected by construction at the tunnel face) was reasonably well described by the Gaussian distribution curve, proposed by Peck (1969):

$$S_v = S_{max} \exp\left[\frac{-y^2}{2i^2}\right] \tag{2}$$

where S_v = vertical settlement
S_{max} = maximum vertical settlement (above tunnel centre line)
y = horizontal offset from tunnel centre line
i = horizontal offset to the point of inflexion of the Gaussian curve.

Integration of eqn. 2 gives

$$V_s = S_{max} \sqrt{2\pi}\ i \tag{3}$$

O'Reilly and New (1982) proposed the correlation that:

$$i = Kz_o \tag{4}$$

where K is the trough width parameter and z_o is the depth to tunnel axis level. It was found that the governing soil parameter was simply whether during construction the soil was undrained (constant volume deformations) or drained (volumetric strains possible). For fine grained soils (clay/silt), O'Reilly and New (1982) found that $K \approx 0.5$. This was confirmed by Rankin (1988) who also analysed data from tunnels other than in the UK and found that K was generally in the range 0.4 to 0.6 and that $K = 0.5$ was therefore reasonable. Data of surface settlements above tunnels in coarse grained soils were found to be much more erratic. Though deformations are usually concentrated in the column of soil directly above the tunnel it was concluded that the surface settlement profile could be approximated to a Gaussian distribution curve and the synthesis of data by O'Reilly and New indicated that $K \approx 0.25$ was reasonable. However, it should be noted that there was noticeable scatter in the data. Equations 1 - 4 and an estimate of likely volume loss (from case histories) can be used to determine the surface settlement profile.

The majority of tunnelling where there is concern about ground movements is in urban environments and the assessment of ground movements and the associated potential building damage has become a major issue. It is usually assumed, at least in the first instance, that the building foundations follow the deformation profile due to the tunnel excavation. This neglects any stiffness of the building in modifying the profile and therefore gives a conservative assessment. For shallow foundations it is often assumed that vertical settlement is given by the Gaussian distribution curve as described above but with depth measured from foundation level to the tunnel axis. At increased depths below the ground surface (or depths greater than for shallow foundations) field and centrifuge test observations indicate that the settlement profile appears to get narrower, but only gradually. Mair et al (1993) correlated published data of subsurface movements and showed that while a Gaussian distribution curve still provides a reasonable description of subsurface settlement troughs, the trough width parameter K tends to increase with depth. The available data were reasonably consistent and an empirical expression for K for tunnels in clay was found to be

$$K = 0.325 + \frac{0.175}{(1 - z/z_o)} \tag{5}$$

where z_o and z are the distances from the ground surface to respectively the tunnel axis and the subsurface level of interest at which K is to be determined (see Fig. 3). The expression is largely based on data from measurements during tunnel construction in London clay and from a limited number of centrifuge tests (see Fig. 4). Although derived from a limited range of data, it is likely to give a reasonable variation of K for a number of different situations.

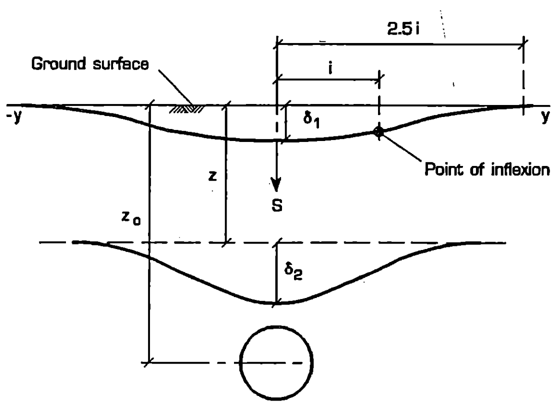
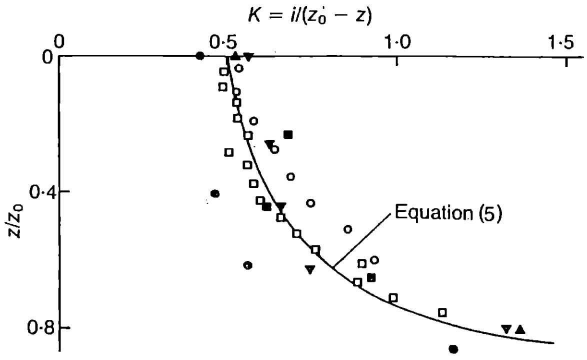


Figure 3. Form of surface and subsurface settlement profiles.

It is often necessary to determine horizontal strains potentially induced in a building and the horizontal component of movement, S_h, due to tunnelling is therefore required. The radial flow analysis given by O'Reilly and New (1982) predicts that the displacement vectors should be directed towards the tunnel axis i.e.

$$S_h = \frac{y}{z_o} S_v \tag{6}$$



	Location	Soil type	D: m	z_0: m	Reference
●	Green Park	London Clay	4·1	29	Attewell & Farmer (1974)
▲	Regent's Park (northbound)	London Clay	4·1	20	Barratt & Tyler (1976)
▼	Regent's Park (southbound)	London Clay	4·1	34	Barratt & Tyler (1976)
■	Willington Quay	Soft clay	4·3	13.5	Glossop (1978)
○	Centrifuge* model 2DP	Soft clay	0·06	0·13	Mair (1979)
□	Centrifuge* model 2DV	Soft clay	0·06	0·22	Mair (1979)

*Models tested at 75g: equivalent full-scale D = 4·5 m, z_0 = 9·8 m (2DP), 16·5 m (2DV)

Figure 4. Variation of K with depth for subsurface settlement profiles above tunnels in clays; (Mair et al, 1993).

This assumption has the convenience that vertical and horizontal strains, determined by differentiating expressions for vertical and horizontal movement, are equal and opposite i.e. a constant volume (undrained) condition is predicted as required for short term deformations of fine grained soils. It should be noted that little is known about horizontal movements in coarse grained (granular) soils.

Introducing $K = f(z)$ affects the calculated vertical and horizontal strains. In order to achieve a constant volume condition, i.e. equal and opposite vertical and horizontal strains, it turns out that the displacement vectors should be directed at a point on the centre-line $0.175z_o/0.325$ below tunnel axis level if eqn. 5 is used for K. This has the effect of reducing horizontal strains from those that would be predicted using the assumption of displacement vectors directed at the tunnel axis.

4 MONITORING OF PERFORMANCE

The degree of monitoring undertaken varies depending on the nature of the project. For most urban tunnels it is quite common to measure settlement. This will usually be restricted to the centre line of the tunnel though the settlement of nearby buildings may be monitored and a transverse array might be incorporated to establish the extent of the zone of influence. It is comparatively rare to make subsurface measurements and these are usually restricted to research programmes or special cases where subsurface structures may be affected. Even less common are measurements of horizontal movements.

In major tunnelling projects, it may be both desirable and necessary to measure the detailed ground response due to tunnelling. This is particularly important where tunnels pass beneath major structures. A recent development in instrumentation is the exploitation of electrolevels. These devices measure rotation and have a resolution of about 3×10^{-4}deg. They are often fastened to beams approximately 1 m long which are then attached to a structure to measure its rotation. Also, the beams can be connected to form long strings of electrolevels. Provided the movement at one end of the string is known, the distribution of movement can be determined in a similar way as inclinometers are used to determine horizontal movements.

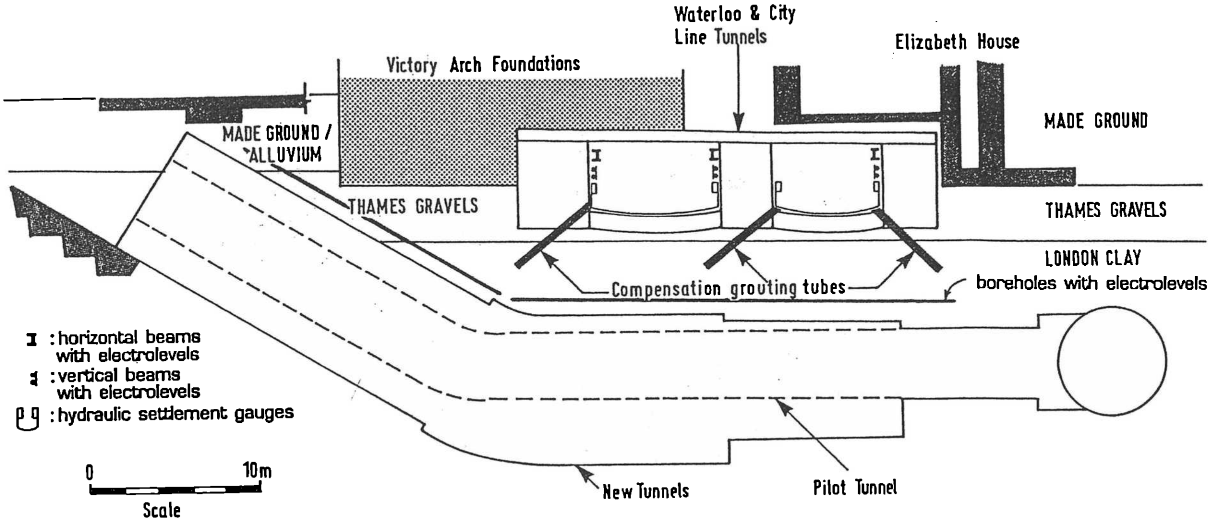


Figure 5. Section through new tunnels at Waterloo Station.

A recent project involving considerable instrumentation was the construction of a new passenger access from the Waterloo Station in London to the LUL Northern line platform (Mair et al, 1994). This involved an inclined escalator tunnel (7.5 m internal diameter) passing within 2 m of the foundations of the Victory Arch and horizontal passageway tunnels (3.85 - 6.5 m ID) passing 5 m beneath the existing Waterloo and City Line tunnels. In order to protect the overlying structures, compensation grouting techniques were used. A key element of this type of ground movement control is continuous monitoring of movements. The structures and the ground were heavily instrumented as shown in Fig. 5. During tunnel construction, the settlement and rotation were recorded and as indications of movement were noted, so grouting operations were undertaken to arrest the settlement. In this way it was possible to maintain settlements to within about 12 mm, considerably lower than the predicted movements (in the absence of grouting) of 50 - 100 mm.

Data of tunnelling induced movements are very valuable and much has been learned from back analysis of the data obtained at Regent's Park (Barratt and Tyler, 1976) and Green Park (Attewell and Farmer, 1974). A major instrumentation project has recently been undertaken by the Transport Research Laboratory for the Heathrow Express trial and the results will be published at the forthcoming Tunnelling '94 conference.

5 CODES OF PRACTICE

The most authoritative documents on tunnel construction in the UK are the report by Craig and Muir Wood (1978) and the Code of Practice on Safety in Tunnel Construction (BS 6164, 1990). State-of-the-Art papers dealing with the development and prediction of ground movements are O'Reilly and New (1982) and Rankin (1988). In determining building damage due to tunnel construction, the most important references are Burland and Wroth (1975), Burland et al (1977) and Boscardin and Cording (1989).

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