Predicting Horizontal Movement for a Tunnel by Empirical and FE Methods

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ABSTRACT: There are a number of benefits İn constructing transport tunnels in highly populated areas for relieving traffic congestion and increasing the speed of travel for commuters. Instead of constructing a single tunnel, multiple tunnels constructed side by side offer more benefits. However, to avoid any adverse effects of tunnels on one another, more attention should be given to estimation of horizontal movement, which will ease not only support design, but also help to determine critical regions around a tunnel. Thus, a number of Finite Element Methods (FEM) as well as empirical analyses were conducted in this research to estimate horizontal movement profiles for a tunnel. The results of both analyses were compared with field measurements as well as each other. The comparison showed that the empirical models could be used to estimate the far-field settlement profiles, but that they could not be used for the near-field ground response to tunnelling. However, finite element analyses were in very good agreement with not only far-field but also near-field ground response tunnelling.

1 INTRODUCTION

The tunnelling process is very complex and requires a robust design to ensure its own stability as well as the stability of other structures which are in interaction. In urban areas, tunnels have a greater potential for disruption due to thetr impact on surface structures when induced settlements exceed the tolerable limits for these structures.

To evaluate ground response to tunnelling without using large-scale trial tunnels, FEM and empirical models are widely used when designing a tunnel. However, it İs a matter of selecting the best analysis method to estimate the horizontal movement more accurately around a tunnel. Thus, this work aimed to appraise and compare predictions of FEM and empirical models for horizontal displacement profiles of a tunnel.

A number of FE analyses and recently developed empirical models were applied and a comparison was made for a tunnel which was constructed in London clay in 1992. This tunnel is of particular importance due to its being the first New Austrian Tunnelling Method (NATM) in London Clay. Thus, the field measurements recorded during the construction of the Heathrow Express trial tunnel in London clay were extensively used throughout the research for comparison with the predictions.*

In order to reflect the non-linear elasto-plastic stress-strain behaviour of London Clay in the FEM analysis, the Modified Cam-clay soil model with non-linear porous elasticity was used for London Clay. The Drucker-Prager plasticity model was adopted for Thames Gravel and made ground because die model considers the effects of die intermediate principal stresses on the failure mechanism in the FEM analysis. The Hypometical Modulus of Elasticity (HME) soft lining approach was employed to consider the 3-D tunnelling problem as well as deformations prior to lining installation for finite elements.

2 FINITE ELEMENT ANALYSIS

2.1 *Heathrow express trial tunnel*

The Heathrow Express trial tunnel excavation was undertaken as part of the Heathrow Express Rail Link, which provides a 15-minute high-speed rail service between Paddmgton railway station in central London and Heathrow Airport. The main concern of the trial was to investigate the ability of NATM to control and limit settlement İn London Clay. Thus, in order to establish the feasibility of NATM in London Clay, three different types of

excavation and support sequences were investigated with this trial tunnel.

The trial tunnel contract consisted of a 10.65m internal diameter x 25m-deep access shaft and 100m of 8.66m-diameter running tunnel. Excavation of the trial tunnel started with Type 1 (TSI), a double side drift sequence, which was followed by Type 2 (TS2), a single side drift sequence, and Type 3 (TS3), crown, bench, and invert face excavation (Figure 1). The trial work on the site was started in February 1992 and completed by June 1992. To ensure the stability of the surface structures and the existing tunnels at the site, the multiple excavation sequences were devised to reduce ground movements and settlement. Thus, this work provided a work process that reduced the ground movements. Moreover, the project aimed to provide information, as well as observation of the ground movements, to tunnel designers about the behaviour of London Clay when excavated. Each trial section progressed for at least 30m in order to obtain adequate and meaningful data over each section.
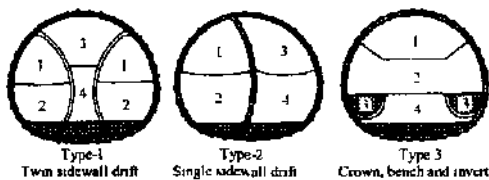

Figure 1. Heathrow Express Trial tunnels.

After completion of the three different types of excavation sequences, the data analysis proved that the maximum settlement occurring over the runnel centre line for TSI, TS2 and TS3 were 27.9mm, 26.8mm, and 40.3mm respectively (TRL, 1992). Thus, only TS2 was included in this analysis as the final settlement was minimum.

2.2 Method of analysis

The finite element analyses were conducted by using the ABAQUS finite element program developed by Hibbit, Karlson & Sorensan, Inc. (1997). Conventional plane strain analysis was used in the analysis stage. In this case, the outer boundaries are located far away from the tunnel so that they are not influenced by the tunnel. The model geometry used in this work was 130m width and 50m height. The selected tunnel size was approximately 7.9m high x 9.2m wide. The model was fixed in the horizontal direction at each side, and the bottom part of the boundary was pinned so that neither vertical nor horizontal movements were permitted. The top surface of the model was free in both directions. Figure 2 illustrates the 2D-modeI geometry. Eight-node biquadratic reduced integration plane strain elements, CPE8R, were used for the continuum body and three-node quadratic curved beam elements, B22, were used for the lining throughout the two-dimensional analysis. The advantage of using the reduced integrated continuum element was a reduction in CPU time, leading to less cost for complex analyses, especially for three-dimensional cases.

The HME soft lining approach was used to consider the 3-D face effects and deformation occurring prior to lining installation. This technique was applied to the Heathrow Express tunnel design at Terminal 4 by Powell et al. (1997). This method was chosen due to its flexibility for multi-stage excavation simulation. The approach considers a lower elasticity modulus for the lining when the lining is in place just after the excavation stage. Thus, there will be deformation occurring due to lower elasticity modulus of the lining (HME). Then, the HME value is increased to the assumed short-term elasticity modulus of the lining. As a result, the use of this approach in a numerical analysis can control deformations and settlements due to tunnel excavation.
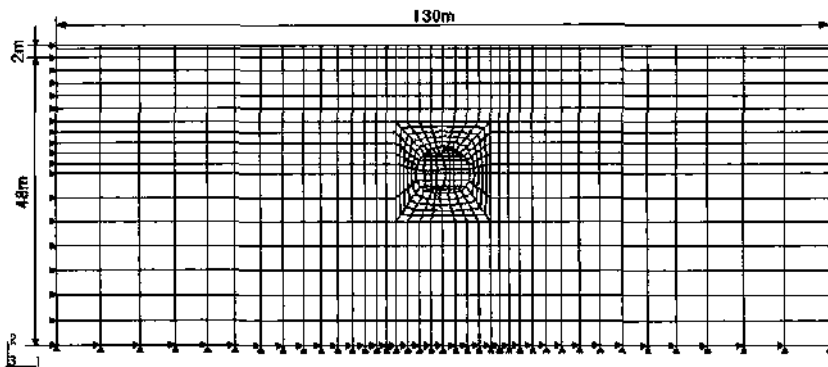

Figure 2. Finite element mesh and boundary conditions.

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Tunnel excavation was carried out in nine steps in the analysis. The initial condition for each step in ABAQUS is the history of the analysis at the end of the previous step. Therefore, complex loading conditions, such as sequential tunnel excavations, can be conveniently analysed. Primary stress conditions in the ground representing a stage prior to excavation are a function of overburden, i.e., a function of the earth pressure at rest, and any additional surcharge because of the existing car park over the tunnel. Thus, all the analyses were covered in a'preliminary run in the first step. However, deformations in this step are ignored since they are not related to tunnelling.

No interface was introduced between the lining and the ground because shotcrete is believed to provide perfect interlock between the ground and itself. In other words, slippage cases were ignored. However, this could be of crucial importance when concrete support is introduced, as in the case of shield tunnelling, or if a shotcrete-concrete interaction case Is subjected to analysis. The detailed analysis procedures employed during this sequential excavation model (SEM) are as follows:

Step 1: The initial stress state was introduced to reach equilibrium before tunnel excavations begin. The beam elements representing the lining were deactivated, as there was no lining at the beginning of the analysis.

Step 2: The left sidewall top heading was excavated. Meanwhile, the lining elements for the top heading with lower elasticity modulus were activated. A 0.40GPa HME value was used for the lining, which was found from back analysis to obtain the required volume loss

Step 3: The stiffness of the beam element for the left top heading, i.e., the HME value, was increased

to 5GPa, which is the assumed short-term elasticity modulus of the lining.

Step 4: The continuum elements in the left sidewall bench and invert were removed and the beam elements with a 0.40GPa HME value for the bench and invert were activated.

Step 5: The HME value of tie lining on the left bench and invert periphery was increased to 5GPa.

For the right sidewall excavation, the same procedure was applied as in steps 2,3,4 and finally in step 9 the value of HME was increased to 5GPa for beam elements on the right bench and invert periphery. At die same time, beam elements representing the central inner lining were removed and the simulation was completed.

2.3 Material properties at the site and constitutive law adopted for the model

The properties of the soils used in this analysis were obtained from Atzl & Mayr (1994), and Powell et al. (1997); they are given in Table 1. The materials encountered at the site consisted of London Clay at a depth of 4.2m overlaid by coarse gravel, with 03m of cement stabilised material and above this a bituminous car park, brick earth (Ryley & Carder, 1995). Beneath the London Clay are the clays and sands of the Woolwich and Reading Beds and the sands of Thanet Beds, which were reported by Bishop et al. (1965) with the sinking of the Ashford Common shaft four kilometres to the south of the trial site. London Clay is clearly dominant at the site and the construction of the trial tunnel was for the most part carried out approximately 16.8 m below the surface.

Table 1 Material properties at the site (Atzl & Mayr, 1994, Powell et al, 1997).

Parameter, Unit and Symbols	Value		
	London Clay	Made Ground	Terrace Gravel
Total unit weight (kN/m³) (γ)	20	19	20
Log plastic bulk modulus (λ)	0 085	-	-
Log elastic bulk modulus (κ)	0.0085	-	-
Cohesion, (kN/m²) (c)	0	0	0
Drucker-Prager outer circle (d) $d = 6c\sin\phi /(3-\sin\phi)$	0	0	0
Effective friction angle (degrees) (φ)	-	35°	35°
Drucker-Prager outer circle $\tan\beta = 6\sin\phi /(3-\sin\phi)$ (degrees) (β)	-	54.8°	54 8°
Young's modulus (GPa) (E)	-	0.010	0 050
Poisson's ratio (ν)	0 15	0.3	0.3
Earth pressure at rest (K₀)	1.15	0 43	0 43
Moisture content, % (w)	30	-	-
Stress ratio at critical state $M = 6.\sin\phi /(3-\sin\phi)$	0 9	-	-
Specific gravity of grains (Gₛ)	2 75	-	-
Parameter defining the size of the yield surface on the wet side of critical state (β)	1.0	-	-
Ratio of flow stress in triaxial tension to flow stress in triaxial compression (K)	1.0	-	-

Pore water pressure was not incorporated within the analyses. However, the undrained material properties of London Clay were adopted in order to conduct total stress analysis. The made ground, 0.5m thick, and Thames gravel, 1.5m thick, were modelled using the drained material properties with the linear elastic perfectly plastic Drucker-Prager failure criteria (Table 1).

As a constitutive model for London Clay, a non-linear porous elasticity model was adopted in which the pressure stress varies as an exponential function of volumetric strain with the Modified Cam-clay plasticity model (Roscoe & Burland, 1968). Anisotropy was disregarded in the analyses conducted.

For the shotcrete used as lining, a typical elasticity model was used in the FEM. The properties of the shotcrete adopted for the model are given in Table 2.

Table 2 Shotcrete properties used in the analysis (TRL, 1992)

Parameters for lining	Outer Lining	Inner wall lining
Thickness, mm	250	150
Poisson's ratio	0.17	0.17
Unit Weight, kN/m^3	25	25
Elasticity Modulus, GPa	5	5

3 EMPIRICAL MODELS

As an alternative method, empirical models based on stochastic distribution analysis are widely used for surface and subsurface settlement analysis. These models assume that a constant volume of ground deformation occurs due to the loss of ground (Bowers, 1997). Attewell (1978), New & O'Reilly (1978), O'Reilly & New (1982) and O'Reilly (1988) developed empirical models to predict the far-field (one tunnel diameter beyond the tunnel periphery) settlement profiles induced by tunnelling. These models provide settlement patterns not only in the transverse direction to the tunnel axis but also İn the longitudinal direction. It has been reported that the normal Gaussian distribution curve provides a good approximation to the shape of the ground settlement above a tunnel (Peck, 1969). Therefore, the settlement, S, at a point of transverse distance, y, from the tunnel centreline is given by the following expression:

$$S = S_{max} \exp(-y^2/2i^2) \qquad (1)$$

where S^ is the maximum settlement at the tunnel centreline, and i is the standard deviation of the

curve. The value i defines the trough width and corresponds to the value of y at the point of inflexion of the settlement curve as shown in Figure 3.
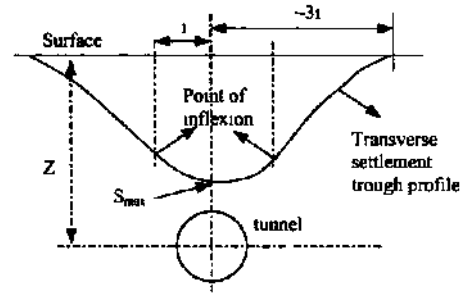

Figure 3. Tunnel settlement trough

The total half width of the settlement trough is given by approximately 2.5i. O'Reilly & New (1982) proposed a linear relationship for the trough width parameter, i, as follows:

$$\text{(For cohesive soils)} \quad i = 0.43Z + 1.1 \qquad (2)$$

$$\text{(For granular soils)} \quad i = 0.28Z - 0.12 \qquad (3)$$

where Z is the depth. For most practical purposes, the value of i is simplified to the following form:

$$i = KZ \qquad (4)$$

where K = 0.5 for cohesive soils and 0.25 for granular soils. For stiff clays and soft-silty clays, this value varies from 0.4 to 0.7 respectively.

The settlement induced by tunnelling is often considered by the term "ground loss or volume loss" and is expressed as a percentage of the notional excavated volume of the tunnel (Mair et al., 1993). When equation 4 is integrated, the volume of the settlement trough per meter of tunnel length, V_s, is calculated as follows:

$$V_s = \sqrt{2\pi} \, i S_{max} \cong 2.5 i S_{max} \qquad (5)$$

Then the generalised settlement can be given as:

$$S = \frac{V_s}{\sqrt{2\pi} \, KZ} \exp(-y^2/2(KZ)^2) \qquad (6)$$

The displacement in the horizontal direction, $H_{(v)}$ can also be derived by using the relationship given below.

$$H_{(y,z)} = \frac{y}{z} S_{(y,z)} \qquad (7)$$

Substituting Equation 6 into Equation 7, the following relationship is derived to calculate horizontal displacements around the tunnel:

$$H_{(y,z)} = \frac{V_s y}{\sqrt{2\pi} KZ^2} \exp(-y^2 / 2(KZ)^2) \qquad (8)$$

The method above, which was developed by O'Reilly & New (1982), is known as *the point sink radial-flow* model. The method assumes that the displacement flow is directed towards a "sink", which is located at a point below the axis level of the tunnel.

The response of the ground to tunnelling is evaluated in two zones. The first is the far-field response to tunnelling, which is at least one tunnel diameter beyond the tunnel periphery. The second is the near-field response to tunnelling, which is within one tunnel diameter. The tunnel diameter in some cases is so large that the near-field environmental impacts cannot be ignored. The point-sink radial-flow model can model the far-field tunnelling response efficiently. However, New & Bowers (1994) concluded that the point-sink radial-flow model fails to predict the pattern of movement in the near field. Thus, they proposed the *"ribbon-sin^* model, whose predictions are much better than the point-sink model for near-field movements. However, Bowers (1997) concluded that as a relatively simple method to calculate volume loss in the near field, this model has a major disadvantage, in that there is no clear relationship between the sink shape and the .tunnel geometry.

As a recent alternative approach to the prediction of near-field settlement, Mair et al. (3993) proposed the "variable-K point-sink model". This model is a modification of the point-sink model and considers the value of the trough width parameter, *K,* which varies with depth. They proposed the following relationship for $K\setminus$

$$K = \frac{0.175 + 0.325(1 - z/z_0)}{1 - z/z_0} \qquad (9)$$

Where z is the depth of the point considered and zg is the depth of the tunnel axis below the ground surface. Incorporating Equation 9 into Eqs. 6 and 8, displacement in both the vertical and the horizontal directions are calculated. The relationship given above, however, is dependent on the ground conditions and it can vary within a wide range. This is the major drawback of this model.

New & Bowers (1994) proposed circle-sink and disk-sink models to analyse the near-field tunnelling response as accurately as possible and provide recognisable parameters to understand the near-field ground response. They, however, concluded that both the ribbon-sink and the variable-K models provide very good fits to field measurements, circle-sink and disk-sink models produce greater settlement curvature in the near field, which results in greater horizontal strains and angular distortion than with the other models. Thus, only the point-sink radial-flow and variable-K models were used to predict horizontal movement using Eq. 8 in this research.

4 EVALUATION OF EMPIRICAL AND FE ANALYSIS RESULTS

As stated earlier, the measurements recorded during the Heathrow Express trial tunnel, type-2, single-sidewall excavation were used to evaluate both the empirical and FE analysis predictions. The instrumentation methodology which was used for the trial work is given in Figure 4. In-tunnel and surface settlement evaluations were omitted as they are beyond the scope of this paper.
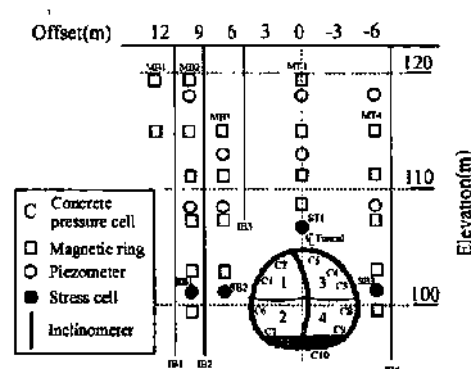

Figure 4. Subsurface and in-tunnel instrumentation around tunnel type 2.

Figures 5-8 show the comparative horizontal movements in the transverse direction to the tunnel predicted by the stochastic models and the sequential excavation finite element model. The range in the predictions by the point-sink model and the variable-re model are not in very good agreement with the field measurements. Even the trend of the movement did not show a close relationship to the field measurements. Both of the empirical models failed to predict the horizontal displacements accurately. However, the predictions of these models for the

surface settlements are very close to the measurements, as discussed in detail elsewhere by Karakuş (2000). Therefore, these models need to be subjected to further investigation. The relationship between surface settlement and horizontal movement given in Eqs. 6 and 8 could be related to different parameters such as tunnel size, excavation pattern and support elements.

On the other hand, the finite element predictions for the horizontal movements are in close agreement with the field measurements. The pattern of the movements also matches the measurements well. Thus, the empirical models examined in this research were found to be inappropriate for horizontal movement analysis of a tunnel.
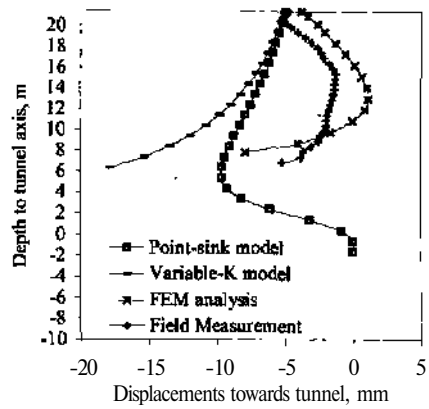
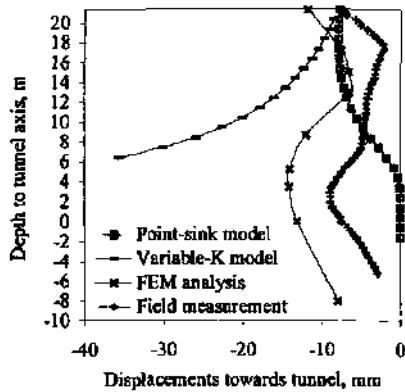Figure 7. Horizontal movements 4.35m from tunnel centreline.

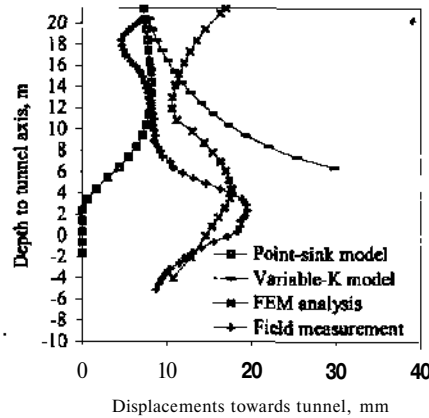Figure 5. Horizontal movements 9.63m from tunnel centreline.

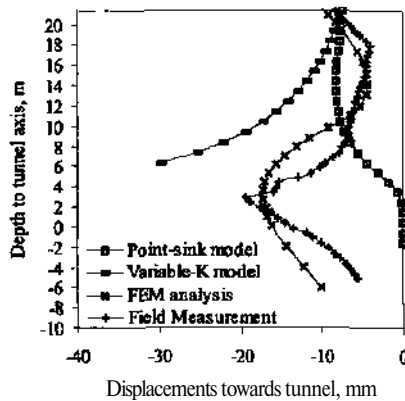Figure 8. Horizontal movements -7.69m from tunnel centreline.

5 CONCLUSIONS

From this research, the following major conclusions can be made:

1. The finite element analyses, when compared with die stochastic models, produced much better results for horizontal movement transverse to the tunnel axis. Moreover, patterns for both horizontal and vertical movements predicted by FEM analysis are in good agreement with the field measurements. Therefore, FEM analysis incorporated with an appropriate constitutive law which will reflect material behaviour as closely as possible can be used for horizontal movement analysis.

2. Use of the Modified Cam-clay model incorporated with the Hypothetical Modulus of

Figure 6. Horizontal movements 7.28m from tunnel centreline.

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Elasticity (HME) approach produced very good predictions in the FEM analysis. Thus, this methodology can be used for any ground like London Clay to investigate ground response to tunnelling.

3. The analysis of the empirical models showed that these models could be used to estimate the far-field settlement profiles, but they could not be used for the near-field ground response to tunnelling. Thus, it is considered that these models cannot be used for assessment of building damage in the near-field zone.

4. Considering that the empirical models are mainly based on past experience, these models are conservative for ground that has not undergone a tunnelling process.

ACKNOWLEDGEMENT

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