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Prediction of subsidence above shield driven tunnels in cohesive soils

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ABSTRACT: A theoretically based procedure for predicting settlement above tunnels constructed in soft ground is evaluated with reference to several case histories. These records encompass very stiff to soft clays. The comparison of observed and calculated behaviour indicated that the proposed theory does provide a reasonable estimate of the surface settlement and the proposed procedure can be used for preliminary design purposes.

1 INTRODUCTION

Empirical procedures such as that proposed by Peck (1969) have been widely used to assess potential ground movements due to tunnelling. When applied with appropriate judgement based on similar past experience, these procedures can yield quite adequate and economical designs. They are, nevertheless, subject to some important limitations; firstly, in their applicability to different tunnel geometries, ground conditions and construction techniques, and secondly, in the limited information they provide about the distribution of settlement other than at the surface.

Based on the results of elasto-plastic three-dimensional finite element analyses (Lee and Rowe; 1990a, 1990b), a simple, theoretically based procedure suitable for the analysis of settlements induced by tunnels constructed in cohesive soils had been developed by Lee et al. (1992). A parameter called the "gap" is defined and is used in an attempt to quantify practical tunnelling experience and various components of lost ground. This gap parameter can then be used in conjunction with two-dimensional finite element methods (such as that developed by Rowe et al. 1983) or by empirical correlations to predict the resulting ground deformations.

The objective of this present paper is to discuss the range of applicability of the proposed theoretical technique. The validity of the proposed design technique will be assessed by comparing the calculated and measured ground displacements in several case histories of shield driven tunnels in soft to stiff cohesive soils.

2 SIMULATION OF LOSS OF GROUND - THE GAP PARAMETERS

In simulating construction of shield tunnels in cohesive soils, consideration must be given to the loss of ground caused by overcutting due to (1) the difference between the diameter of the tunnelling machine and that of the lining; (2) three-dimensional soil movements ahead of the tunnel face; and (3) alignment problems encountered when steering the shield. The net effect of these factors may be approximately incorporated in a two-dimensional plane strain analysis in terms of a void or the so-called "gap" parameter. As the shield advances, the weight of the lining will cause it to rest on the bottom of the excavated surface, thus the gap parameter can be visualized as the vertical distance between the top of the tunnel lining and the point in the soil which will eventually become the crown of the excavated surface; in other words, the maximum crown settlement (see Figure 1). In most practical situations the loss of ground will be greater than that resulting from the difference in the diameter of the tunnelling machine and the final outside diameter of the lining. Causes of lost ground include overcutting due to three-dimensional movement into the face, overcutting due to alignment problems encountered when steering the shield. Thus the gap parameter (GAP) is defined (Lee et al., 1992) as

$$\text{GAP} = G_p + u^*_{3D} + \omega \qquad (1)$$

where:

G_p = physical gap (usually the difference between the maximum outside diameter of the tunnelling machine and the outside diameter of the lining for a circular tunnel)

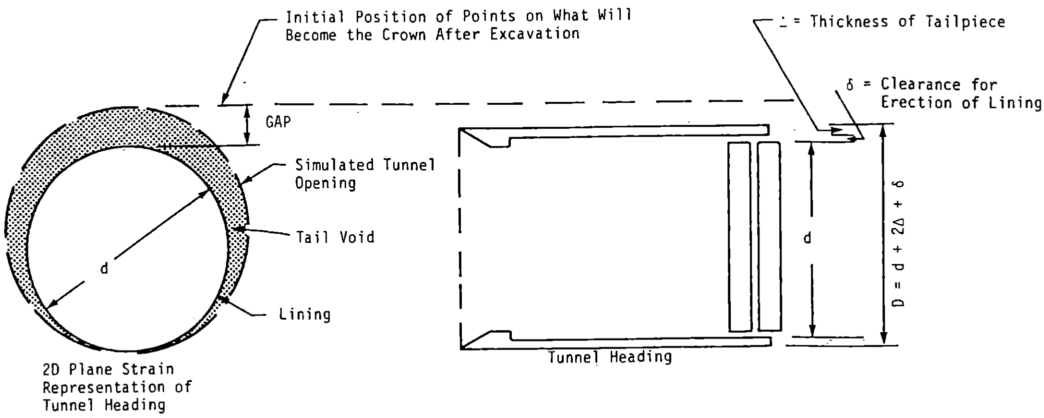

Figure 1 Definition of GAP

u^*_{3D}	=	three dimensional elastoplastic deformation	
u^*_{3D}	≤	$0.5\ \delta_x$	(2a)
δ_x	=	face intrusion = $\Omega\ a\ P_o / E$	(2b)
P_o	=	$K_o'\ P_v' + P_w - P_i$	(2c)
Ω	=	is a dimensionless displacement factor evaluated from Figure 2	
N	=	stability ratio = $(\gamma H - P_i)/c_u$	
a	=	tunnel radius (diameter D = 2a)	
E	=	Young's modulus (typically the undrained modulus in extension, $E=E_u$)	
K_o'	=	effective coefficient of earth pressure at rest	
P_v', P_w	=	vertical effective stress and pore water pressure at springline of the tunnel	
P_i	=	tunnel support pressure	
γ, c_u	=	representative unit weight and undrained shear strength of the soil	
H	=	distance from ground surface to springline of the tunnel	
ω	=	workmanship factor	
ω	≤	$0.6\ G_p$; and	(3a)
ω	≤	$u_i/3$	(3b)

$$u_i = a\left[1 - \left\{\frac{1}{1 + \frac{2(1+\nu_u)\,c_u}{E_u}\left[\exp\left(\frac{N-1}{2}\right)\right]^2}\right\}^{1/2}\right] \tag{3c}$$

ν_u = Poisson's ratio for undrained conditions (typically $\nu_u = 0.5$).
and for N ≤ 1, the soil response is elastic and is given by Lo et al. (1984).

The reader is referred to Lee et al. (1992) for a detailed discussion of the evaluation of the GAP parameter as defined above.

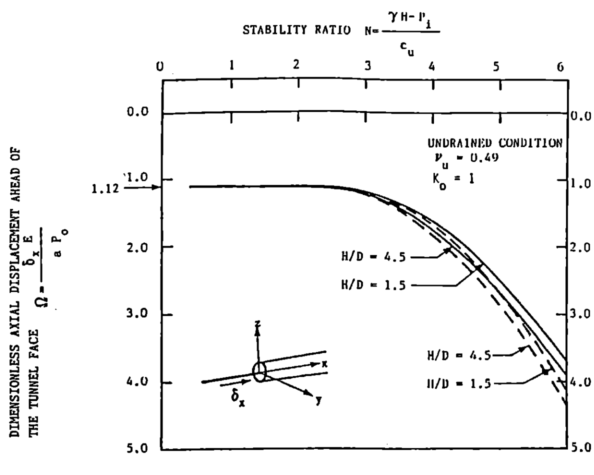

Figure 2 Dimensionless axial displacement ahead of the tunnel face (after Lee et al. 1992)

3 PREDICTION OF SURFACE SETTLEMENT

The gap parameter can be evaluated prior to tunnel construction and hence can be used to estimate the surface settlement for a proposed method of construction in the design stage. The gap parameter can be: (i) incorporated into two-dimensional (transverse section) finite element analyses; or (ii) used in conjunction with empirical relationships.

Details concerning the incorporation of the gap parameter into two-dimensional finite element analysis have been given by Rowe et al. (1984) and Ng et al. (1986) and will not be repeated herein.

Alternatively, the gap parameter can be considered to be the vertical displacement above the crown of the tunnel (δ_c) and hence maximum surface settlement (δ_{max}) can then be estimated from the settlement ratio (δ_{max}/δ_c) established through empirical correlation. One such relationship was recently developed by Ng (1991) based on eighteen case histories. The relationship between settlement ratio δ_{max}/δ_c and dimensionless tunnel parameter H/2aN (where H, a

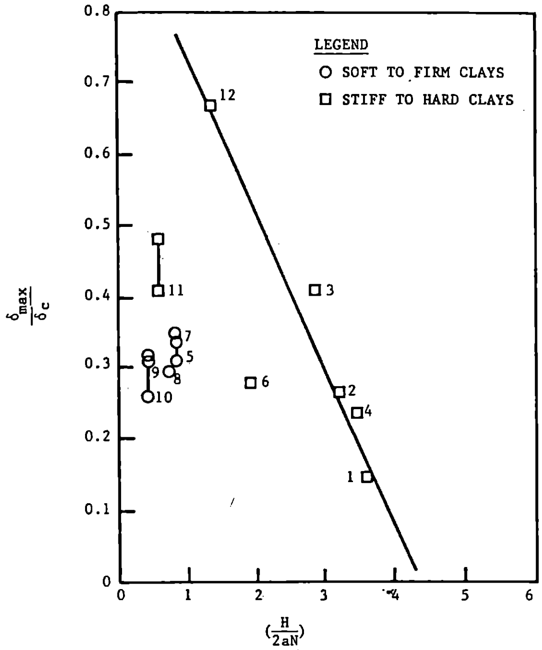

Figure 3 Relationship between (δ_{max}/δ_c) and (H/2aN). (After Rowe and Lee, 1992)

and N are as previously defined) tunnels in clays is reproduced in Figure 3. It is noted that, for soft clays, the average δ_{max}/δ_c ratio is about 1/3 regardless for the dimensionless parameter H/2aN. This relationship of $\delta_{max} = \delta_c/3$ = GAP/3 for soft clays suggests the likely improvement in maximum surface settlement corresponding to an unit improvement of crown settlement. For stiff to hard clays, the ratio of δ_{max}/δ_c is approximately linearly related to the dimensionless tunnel parameter H/2aN (see Figure 3). The recorded ratios varied from 0.1 to 0.7. In general, the absolute magnitude of settlement associated with hard clay is usually small.

The proposed methodology only predicts the maximum surface settlement developed at the ground surface δ_{max}. The shape of the settlement trough can be reasonably described by the Gaussian distribution curve as proposed by Peck (1969):

$$\delta_v = \delta_{max} \exp(\frac{-y^2}{2i^2}) \tag{4}$$

where

δ_v = vertical settlement

δ_{max} = maximum vertical settlement (above tunnel centre line)

y = horizontal distance from tunnel centre line

i = horizontal distance to the point of inflexion of the Gaussian curve

The total volume of settlement trough V_s (or the total volume of ground loss for the undrained case) can be obtained by integration of Eqn. (4), which gives

$$V_s = \delta_{max} \sqrt{2\pi} i \tag{5}$$

To completely define the shape of settlement trough, an additional parameter, i, must then be defined. O'Reilly and New (1982) proposed the correlation that:

$$i = Kz_o \tag{6}$$

where K is the trough width parameter and z_o is the depth to tunnel axis level (ie. z_o = H). 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:

$$K = 0.325 + \frac{0.175}{(1-z/z_o)} \tag{7}$$

where z_o and z are the vertical distances from the ground surface to the tunnel axis and the subsurface level of interest at which K is to be determined, respectively. The expression in eqn. (7) is largely based on data from measurements during tunnel construction case histories and from a limited number of centrifuge tests (Mair et al, 1993). Although derived from a limited range of data, it is likely to give a reasonable variation of K for a number of different situations.

Empirical relationships as shown in Equations (4) to (7) can be used in conjunction with the proposed technique to completely define the surface and subsurface settlement troughs of grounds induced by circular shield driven tunnels.

4 ASSESSMENT OF THE PROPOSED PROCEDURE USING FIELD CASE HISTORIES

To assess the applicability of the proposed analysis, six case histories encompassing soft to very stiff clay and various construction techniques have been chosen for analysis (see Table 1). Detailed descriptions of ground conditions, method of construction, estimation of the gap parameter and finite element analysis were given by Rowe and Lee (1992).

The estimation of the gap parameters, predicted and observed surface centre-line settlement for various case histories are given Table 1. Comparison between the observed and predicted maximum surface settlements for the six case histories are

Table 1 Estimation of gap parameter for various case histories

Tunnel	Ground Condition	C/D	N	U*3D (mm)	G_p (mm)	ω (mm)	Gap Parameter (mm)	Surface Settlement Predicted (mm)	Observed (mm)
1. Thunder Bay Array 1	Soft silty clay	4.3	4.8	30	90	0-54	120-174	40-58[2]	49-76
2. Thunder Bay Array 2	Soft silty clay	4.3	4.8	0-30	90	0	90-120	30-40[2]	37-50
3. Manuel Gonzalez Tunnel - Mexicao City	Soft highly compressible clay	4.0	5.0	80	130	78	288	96[2]	105
4. Lower Market St. - Bart Subway	Recent soft marine silty clay	3.4	4.6-6	0	160	-90 - 0	70-160	23-53[2]	25-50
5. Green Park Underground	Stiff O/C clay	7.1	2.2	20.5	6.5 (size of bead)	0	27	4[1]	5-7
6. Mississauga Sewer Tunnel	Dense sand-clay till	3.1	0.9		($u_i < G_p$) $u_i = 11$ $G_p = 100$		11	2.5[1]	2-3

(1) predicted by empirical relationship as shown in Figure 3
(2) predicted by empirical relationship $\delta_{max}=\delta_c/3$

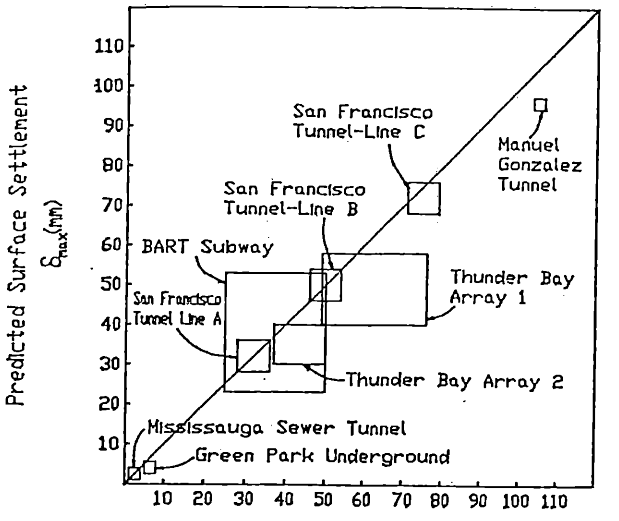

Figure 4 Comparison of the predicted and observed maximum surface settlement for the case histories

given in Figure 6. It can be seen that the proposed theory does provide a reasonable estimation of the range of the gap and consequently the surface settlement for all six case histories.

5 APPLICATION FOR THE "ADVANCE" CLOSED-FACE SHIELD

When assessing the gap parameter to be used in predicting likely settlements it is important to recognise that, irrespective of the methods of ground control that are used, some three-dimensional movement at and in front of the shield usually takes place during face advance. Depending on the pressures applied at the face, these movements may be outward (e.g. for high face pressures ahead of closed-face shields) or inward (e.g. for conventional open-face shields or if face pressure is lost due to a work stoppage when using an "advance"closed-face shield). To assess the GAP parameter to be used in predicting settlement it is necessary to consider the likely face condition but also it may be useful to consider the implications of loss of face pressure. When considering the use of full-face tunnel boring machines (TBMs) the case of a face pressure restricts inward movement can be modelled by assuming that there is no three-dimensional component to the GAP parameter and focusing on the "workmanship" and "physical" components of the gap. On the other hand, if there is a loss of face pressure (e.g. due to a work stoppage), then there will be inward movement at the face, and an upper bound to the settlement can be obtained using the proposed approach for predicting the three-dimensional component of the gap. If excess face pressure is used, this may result in remoulding of soils around the tunnel and, although it results in smaller short-term settlement, the additional consolidation that will occur in these zones must be considered in assessing the long-term settlement. This concept can be illustrate by a case history as described by Clough et al. (1983) for a tunnel constructed in soft San Francisco Clay mud.

This project involved a 3.7 m diameter EPB shield driven through a 915 m long tunnel beneath downtown San Francisco. The soil profile consists of 6.1 m of rubble fill which overlies a 9.1 m layer of soft, saturated cohesive soils known as Recent Bay Mud. A colluvium layer underlies the Recent Bay Mud. The tunnel section is located within the Recent Bay Mud and has a cover of approximately 9.1 m; water heads above the crown are typically 4.5 m. The outside diameter of the erected ring was 3.55 m. With allowances for the tail seals, construction tolerances, and the skin thickness of the tail of the shield, the tail gap (physical gap G_p) left between the soil and the outside of the liner was 150 mm.

As reported by Clough et al. (1983), the shield was advanced generally on a slightly upward pitch of typically less than 0.5% above grade along most of the line, and there was no major alignment or steering problem recorded. therefore this would result in essentially negligible ground loss due to overcutting or "workmanship" problem. The workmanship parameter ω for this case history can be considered as zero. Since the EPB advanced tunnelling shields have the capability to balance soil and water pressures at the tunnel face or even operate with excess face pressure, the u^*_{3d} can be zero or even negative. As a first prediction, the gap parameter (GAP) for this case can be assumed as GAP = G_p, and it is equal to 150 mm. Using the empirical correlation for soft clay of $\delta_{max} = \delta_c/3$ = Gap/3 as define in the previous section, the predicted maximum surface settlement (δ_{max}) would be 50 mm. This predicted value is plotted as line B in Fig. 5 and compared with the observed surface-settlement profile along the centre-line of the tunnel alignment. Except between stations 5+00 and 7+00, and between 28+00 and 30+00, this predicted settlement B of 50 mm generally overestimates the observed surface settlement along the majority of tunnel alignment, in particular, between stations 7+00 and 28+00 where the settlement averaged about 30 mm. The predicted surface settlement of 50 mm is also plotted as point B on the histogram of surface settlement measurements as shown in Fig. 6. Again it indicates that this predicted value of 50 mm is falling on the high side (upper 25%) of the observed surface settlement, and hence this method of calculation represents an upper bound to the likely settlement if the face pressure is maintained.

Examination of Fig. 5 indicates that maximum settlements of about 76 mm were recorded at stations 1+60 and 24+50. Clough et al. (1983) suggested

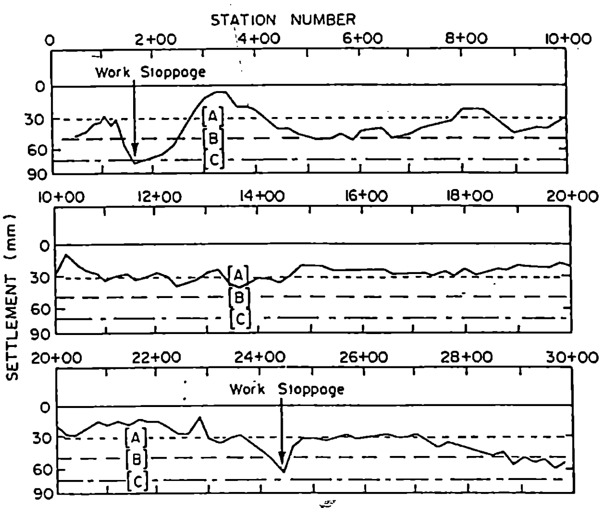

Figure 5 Surface settlement profile along centreline of tunnel alignment (modified from Clough et al. 1983)

that the peak at station 1+60 represented the effect of a 15 day work stoppage required for connecting the EPB power train to the shield, and the settlement peak at station 24+50 was caused by a work stoppage when the spoils retaining area was emptied of soil to clear the slots in the cutterhead of wooden debris. This work stoppage would result in a gradual loss of earth pressure support at the bulkhead near the tunnel face as a result of soil consolidation or earth and porewater pressure redistribution. It is also a common practise that the soil slurry accumulated at the bulkhead may have to be cleared before further advance of the shield. This would result in significant reduction of outward supporting pressure at the tunnel face after the work stoppage. The u^*_{3d} under these conditions is not equal to zero and is temporarily similar to the condition imposed by a "conventional" open-face tunnelling shield. This value can therefore be estimated by Eqn. (2a-c). Based on the subsurface conditions given by Clough et al. (1983), the undrained shear strength of the Recent Bay Mud is about 30 kPa around the tunnel axis. The ratio of elastic modulus to undrained shear strength (E_u/c_u) is taken as 300, as suggested by Finno and Clough (1985). The calculated u^*_{3d} is therefore about 66 mm. The gap parameter (GAP) is the sum of G_p of 150 mm and u^*_{3d} of 66 mm, or a total of 216 mm. The predicted maximum surface settlement δ_{max} can be taken as GAP/3, or 72 mm. This calculated value is then plotted as line C in Fig. 5 and 6. It can be seen that the observed maximum surface settlement (associated with work stoppage with the maximum value of 76 mm at stations 1+60 and 24+50) is closely approximated by the predicted value δ_{max} of 72 mm.

The existing form of the proposed predictive technique does not allow one to estimate, at priori, a negative value of u^*_{3d} to account for outward movement and heaving of soil in front of the tunnel face. The field observations reported by Clough et al. (1983) indicated that the magnitude of outward soil movement for this tunnel was of the order of 52

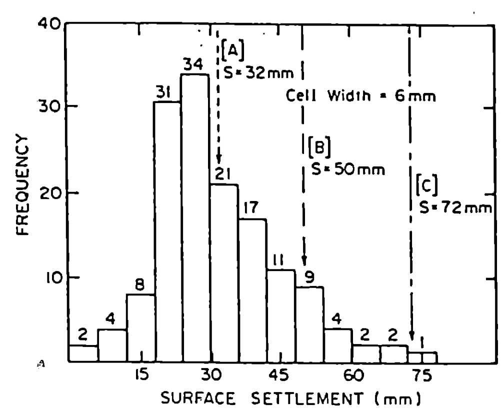

Figure 6 Histogram of surface settlement data (modified from Clough et al. 1983)

mm just ahead of the tunnel face. In this case we can make an improved estimate of the likely settlement by taking the u^*_{3d} of -52 mm, giving a GAP = $G_p + u^*_{3d} = 150 + (-52) = 98$ mm. The predicted maximum surface settlement is therefore equal to GAP/3 of 32 mm. This value is very close to the most commonly recorded value of 30 mm as indicated by Line A in Figs. 5 and 6.

CONCLUSION

The results presented in this paper provide some indication as to when a reasonable prediction of settlement could be made (prior to construction) from a limited knowledge of the soil profile and the construction procedure. Analyses of seven case histories involving tunnelling in clays of widely different strength and geometries have been performed by the proposed technique. It is found that the calculated surface displacements agree reasonably well with measured displacements. It is suggested that the proposed simplified procedure can be used for preliminary design purposes of circular sheild driven tunnels constructed in soft to stiff clays.

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