Subsidence owing to tunnelling. I. Estimating the gap parameter

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A simple, numerically based procedure for the estimation of the so-called gap parameter is described. The gap parameter represents the vertical displacement above the crown of the tunnel and is a measure of ground loss owing to tunnelling in soft ground. This parameter is a function of the three-dimensional elastoplastic deformation at the tunnel face, the tunnelling machine shield and lining geometry, and workmanship. This parameter can then be used to predict the resulting ground deformations using either two-dimensional finite element methods or empirical correlations.

Key words: tunnelling, settlement, subsidence, clays, soft ground, analysis, design.

Une procédure simple à base numérique pour estimer le paramètre nommé gap est décrite. Le paramètre gap représente le déplacement vertical au-dessus de la voûte du tunnel et est une mesure de la perte de sol due au creusage de tunnel dans le sol mou. Ce paramètre est fonction de la déformation élastoplastique tridimensionnelle à la face du tunnel, de la géométrie du bouclier du tunnellier et de la garniture, de même que de la qualité de l'exécution. Ce paramètre peut alors être utilisé pour prédire les déformations résultantes du sol au moyen soit de méthodes d'éléments finis bidimensionnelles, soit de corrélations empiriques.

Mots clés : percement de tunnel, tassement, affaissement, argiles, sol mou, analyse, conception.

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Introduction

It is well recognized (Peck 1969) that the deformations caused by tunnelling (and the consequent potential damage to adjacent and overlying services and structures) will depend on (*i*) the ground and groundwater conditions, (*ii*) the tunnel depth and diameter, and most importantly (*iii*) the construction details. Empirical procedures (Peck 1969) have been widely used to assess potential ground movements owing to tunnelling. When applied with appropriate judgement based on similar past experience, these procedures can yield adequate and economical designs. They are, nevertheless, subject to some important limitations; first, in their applicability to different tunnel geometries, ground conditions, and construction techniques, and secondly, in the limited information they provide concerning the distribution of settlement other than at the surface.

A theoretically based method for the prediction of settlements at the surface and at various depths has been suggested by Lo and Rowe (1982) and Rowe *et al.* (1983). An important aspect of this approach is the introduction of a parameter, called the gap parameter, which takes into account the ground loss as a function of strength and deformation behaviour in the elastic and plastic state, physical clearance between the excavated diameter and lining, and workmanship. It has been found that the method yields settlement profiles that are in reasonable agreement with field measurements in several well-documented case histories (Rowe and Kack 1983; Ng *et al.* 1986), provided that the gap parameter is correctly chosen. It should be noted that in the previous investigations the so-called gap parameter was estimated largely based on engineering judgement. To readily apply this approach in engineering applications, a rational design procedure for estimating the gap parameter is needed.

The purpose of this paper is to develop a simple design procedure for estimating the gap parameter for tunnels excavated in soft clayey soils under undrained conditions. This improved procedure is based on the results of three-dimensional (3D) elastoplastic finite element analysis (see Lee and Rowe 1990*a*, 1990*b*) and extends earlier work reported by Lo *et al.* (1984). The procedure only involves simple equations and conventional soil properties, hence these methods may be readily applied in design analysis of tunnels in clays. This gap parameter can then be used in conjunction with two-dimensional (2D) finite element analyses (e.g., see Rowe *et al.* 1983) to predict the resulting undrained ground deformations. A companion paper (Rowe and Lee 1992) evaluates the validity of the proposed design guideline by comparing the calculated and measured ground displacements for several case histories.

Simulation of loss of ground: the gap parameter

Excavation of the tunnel provides an opening into which the soil can deform and the constraint to soil movement is primarily a function of the machine characteristics, workmanship, lining geometry, and lining flexibility. The movement of the soil into the opening can be related to the concept of "loss of ground," which is defined as volume of material (whether through the face or radial encroachment over and around or behind the shield) that has been excavated in excess of the theoretical design volume of excavation. Field studies (Peck 1969; Cording and Hansmire 1975) have highlighted the 3D nature of the problem and the effect of 3D ground loss on the subsequent deformations. Based on field case histories, the loss of ground can be considered to occur in three stages as the tunnel advances in the soil mass: (1) ahead of the face, (2) over the shield,

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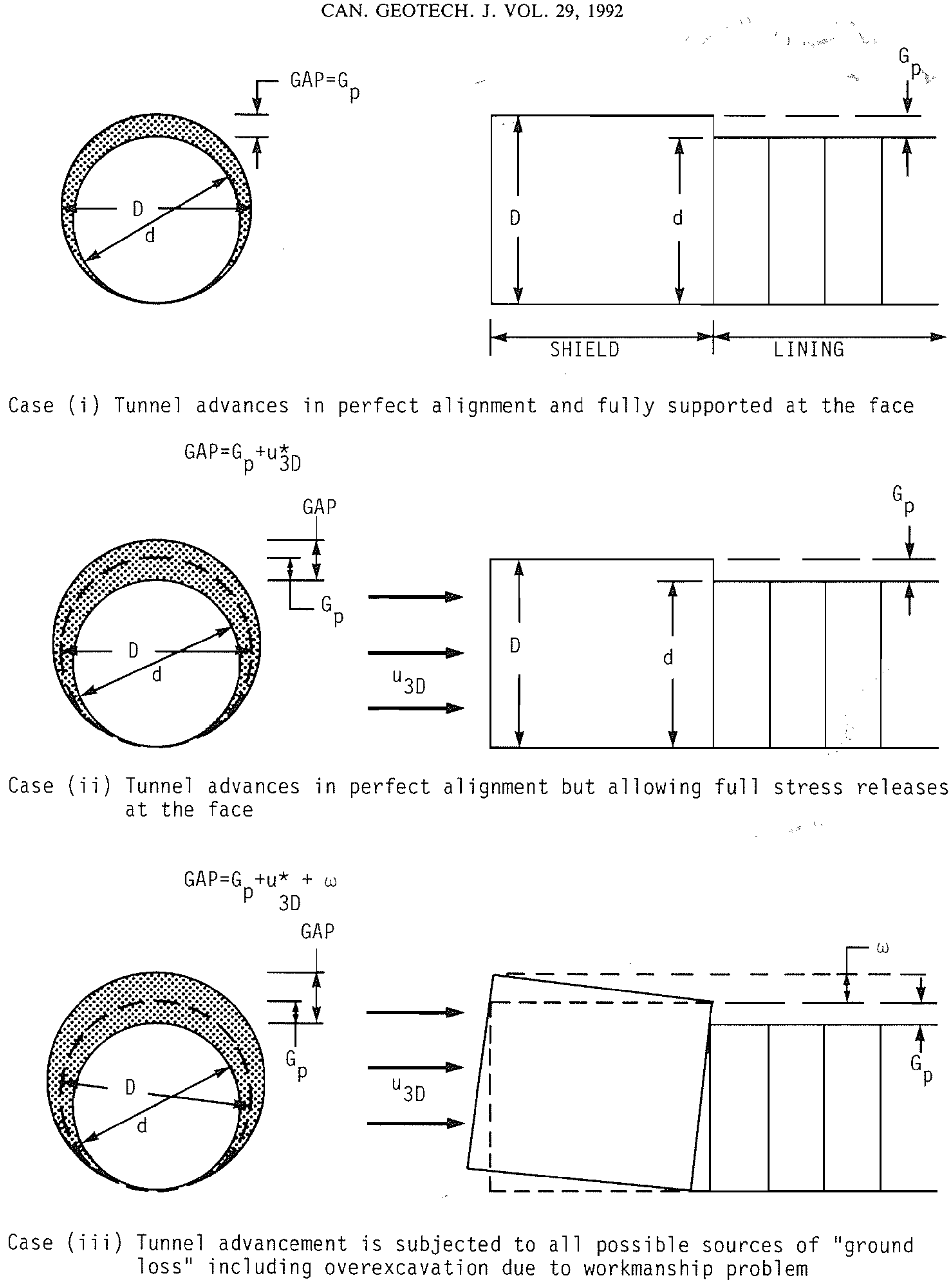

FIG. 1. Simulation of loss of ground. The total gap parameter.

and (3) upon the erection of the lining. It is further considered that additional loss of ground may result from creep, consolidation, and (or) a long-term change in hydraulic conditions.

Three-dimensional finite element analysis such as that described by Lee and Rowe (1990*a*, 1990*b*, 1991) can be used to predict the spatial 3D ground displacements within the soil mass. However, because of the high cost and processing time associated with this analysis, simplified 2D procedures are usually adopted. For the purpose of performing a two-dimensional plane strain analysis, the components of loss of ground discussed above may be represented quantitatively in terms of the so-called gap parameter. Thus the gap parameter is a measure of various components of lost ground into the tunnel as illustrated in Fig. 1. Under idealized construction conditions in which the tunnelling machine is kept hard against the face and (or) compressed air is used to minimize stress changes and deformations into the face, and the tunnel is advanced under perfect alignment, the gap parameter is equivalent to the physical gap (G_p), which is defined as the vertical distance between the crown of the tunnel lining and the crown of the excavated surface prior to removal of the tunnel tractions (schematically represented as the term G_p in case *i* of Fig. 1). If less conservative construction procedures are adopted, 3D movements ahead of the tunnel heading may be significant (represented as u^*_{3D} in case *ii* of Fig. 1). Similarly, construction difficulties such as steering and alignment problems with the tunnelling shield can cause over-excavation and (or) remoulding of the soil adjacent to the tunnel (represented as workmanship parameter ω in case *iii* of Fig. 1). These movements can be approximately incorporated in the 2D plane strain model by assuming a slightly larger excavated tunnel diameter, with an additional volume corresponding

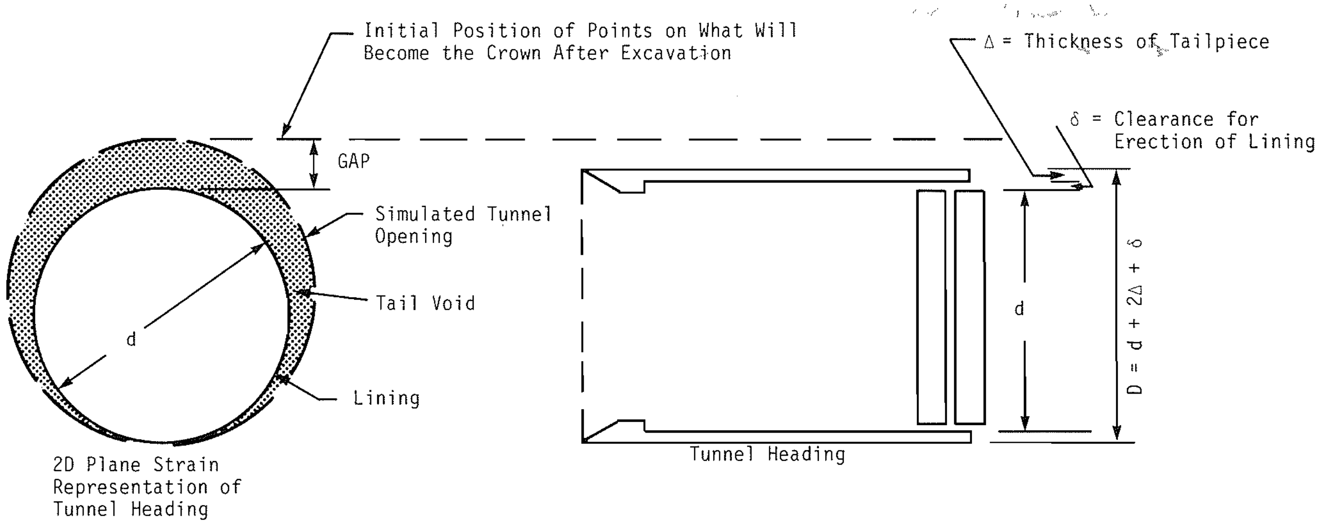

FIG. 2. Definition of GAP (modified from Lo *et al.* 1984). The gap parameter is given in [1].

to the volume of ground lost through the heading and over the shield (as shown in Fig. 2). Thus, the gap parameter (GAP) can be considered as the maximum settlement at the tunnel crown, and it may be expressed as

[1] $\mathrm{GAP} = G_p + u^*_{3D} + \omega$

where $G_p = 2\Delta + \delta$. The physical gap ($G_p = 2\Delta + \delta$) represents the geometric clearance between the outer skin of the shield and the lining, and it is composed of the thickness of the tailpiece (Δ) and the clearance required for erection of the lining (δ). The term u^*_{3D} represents the equivalent 3D elastoplastic deformation at the tunnel face. The term ω takes into account the quality of workmanship.

Method of analysis and principal assumptions

In the present study, 3D elastoplastic finite element analysis is used as a tool to develop a simple design procedure for estimating the gap parameter for tunnels excavated in cohesive soils. The finite element mesh adopted in the present study consisted of about 4800–6000 elements depending on the geometry of the problem. Eleven-noded, nonconforming brick elements were used to discretize the soil. This nonconforming element is very similar to the 8-noded linear Lagrangian element, except it has three auxiliary internal incompatible displacement nodes to improve the field variable representation within an element (see Lee and Rowe 1990*a*, for details).

It should be noted that the proposed design procedure is only applicable for predicting settlement owing to various sources of ground loss in an undrained state for saturated cohesive soils; it does not consider consolidation resulting from the stress changes around the tunnel or reconsolidation of any remolded zone that results from the tunnelling process. These effects have been discussed briefly by Ng *et al.* (1986).

The soil is assumed to have an elastic – perfectly plastic constitutive relationship which (for undrained conditions) was defined by a Tresca failure criterion and a flow rule of the form proposed by Davis (1968). Details regarding this form of the soil model, the choice of finite element, the construction simulation, and the solution technique have been described by Lee and Rowe (1990*a*) and will not be repeated herein.

In this study, the effect of plasticity, for a given geometry, is examined by considering a number of values of undrained shear strength c_u. The elastic Young's modulus profile with depth (E_u) adopted in the analysis is similar to that adopted by Lee and Rowe (1990*b*) and involves a E_u/c_u ratio in the range of 200–800. The vertical initial stress is assumed to be due to the weight of the overburden with a unit weight of soil of 20 kN/m^3, and the *in situ* initial stress ratio is assumed to be unity. Since only the undrained behaviour of the soil mass is considered, there is no plastic volume change.

Analyses were performed to study the 3D behaviour of the soil mass due to excavation of a tunnel of diameter D at tunnel cover depths, H (distance from ground surface to tunnel springline), of 1.5 to 4.5 D. In this paper, only summarized results will be presented; detailed results of analyses for various geometric and ground conditions were given elsewhere (Lee 1989).

Excavation simulation

The method of excavation and support that is usually employed in soft ground consists of mechanical excavation followed by immediate erection of a tunnel lining within the tailpiece of a protective shield. In finite element analysis, this construction process can be simulated by deducing the tractions that would be acting around the surface of the tunnel (prior to excavation) and then removing those tractions. This approach is valid for all values of the coefficient of earth pressure at rest (K_o) (e.g., see Kulhawy 1974) and is considered to provide a reasonable approximation of the change in stress that occurs with the advance of the tunnelling machine.

Once the soil clears the tailpiece of the tunnel shield, soil would invade the tail void behind the shield and the weight of the lining will cause it to rest on the excavated surface (as shown in Fig. 2), while the lining will limit the possible deformations of the soil. The tail void left behind the tail skin can be represented by a diameter of the circular opening in the finite element mesh slightly larger than the diameter of the lining. The tail void itself is nonuniform in shape (Fig. 2), with a zero thickness at the tunnel invert. By definition, the difference between these two diameters is represented by the so-called gap parameter as described by [1].

In the present analysis, tractions to simulate the excavation process are determined by the procedure proposed by Brown and Booker (1985). This approach ensures that total equilibrium is always satisfied and, depending on the magnitude of *in situ* initial stresses, nonuniform tractions around the tunnel periphery will generally result. These deduced tractions are then removed from the tunnel opening periphery in increments. The displacements at all nodes on the periphery are monitored; if the displacement of a node indicates that the gap has closed at that point, then the soil–lining interaction proposed by Rowe *et al.* (1983) is activated. It should be noted that once the soil comes into contact with the lining, its movements are constrained by the lining and lining pressures will develop. Construction simulation is completed when all boundary tractions are removed. The gap parameter may be considered as the maximum settlement at the tunnel crown when interpreting field measurements such as the vertical displacement near the crown level as determined from extensometer or deep settlement probe. Furthermore, the surface settlement at the centreline of the tunnel can be empirically correlated with the crown settlement by the relationship such as that established by Lo *et al.* (1984) and Ng (1991).

The following sections will describe the approximate technique for estimating various components of the gap parameter.

Ground losses ahead of the tunnel faces

Removal of the *in situ* stresses ahead of the tunnel face resulting from the excavation of the tunnel will cause the soil to intrude into the tunnel face. Therefore, a volume of lost ground is developed equal to the amount of overexcavated or displaced material at the face. Based on the results of 3D finite element analyses (Lee 1989), it is found that the progressive development of 3D "face loss" owing to the continuous advance of a tunnelling shield can be approximately simulated in a plane strain finite element analysis (at a transverse section) by increasing the maximum allowable radial displacement at the tunnel crown. This 3D effect can then be incorporated in the gap parameter in terms of u^*_{3D}. By representing a 3D configuration into an equivalent 2D approximation (see Appendix for detail), it is found that u^*_{3D} can be given approximately by

$$[2] \quad u^*_{3D} = \frac{k_1}{2}\delta_x$$

where k_1 is a factor taking account of the doming effect across the tunnel face and can be expressed as:

$$[3] \quad k_1 = \frac{\text{volume of nonuniform axial intrusion across the tunnel face determined by 3D analysis}}{\text{volume assuming uniform axial intrusion}}$$

(i.e., $0 < k_1 < 1$; $k_1 = 1$ represents an uniform intrusion at the tunnel face); and δ_x is the magnitude of maximum axial intrusion at the tunnel face. Thus, the 3D soil movements ahead of the tunnel face can be represented by u^*_{3D} provided that the values of k_1 and δ_x are determined.

The parameter k_1 has been examined by Lee (1989) using 3D elastoplastic finite element analyses on various combinations of soil parameters and tunnel geometries. Results from these analyses indicate that the k_1 parameter is generally in the range of 0.7–0.9. It should be noted that analyses performed by Lee (1989) did not consider the shoving forces generated by the advancing shield. Peck (1969) indicated that the frictional forces between the skin of the shield and the surrounding soil caused by the shoving action of the shield can develop longitudinal tensile stresses in the clay that may tend to cause failure and plastic flow of the clay into the tunnel face and the annular void between the tail skin. These frictional forces were not simulated in the analysis; however, if this effect was simulated, a more uniform soil intrusion across the tunnel face would be expected. In design situations, a value of $k_1 = 1$ would appear to be reasonable. Therefore, [2] can be simplified as

$$[4] \quad u^*_{3D} = \frac{\delta_x}{2}$$

The displacement δ_x can be represented in a nondimensional form, viz,

$$[5] \quad \Omega = \frac{\delta_x E}{a P_o}$$

where

$$[6] \quad P_o = (K_o' P_v' + P_w) - P_i$$

where E is the elastic (Young's) modulus at the tunnel springline (typically the undrained Young's modulus in extension, i.e., $E = E_u$); a is the tunnel radius, P_o is the total stress removal at the tunnel face, K_o' is the effective coefficient of earth pressure at rest, P_v' is the vertical effective stress at the tunnel springline, P_w is the pore pressure at the tunnel springline (prior to construction of the tunnel), and P_i is the tunnel supporting pressure (if the tunnel is fully excavated, $P_i = 0$; the presence of compressed air at the tunnel face would cause $P_i > 0$).

Another nondimensional parameter frequently used in soft-ground tunnels is the stability ratio N, which can be considered as a measure of the available shear strength to the average boundary stress removed, and is defined as

$$[7] \quad N = \frac{\gamma H - P_i}{c_u}$$

where c_u is the undrained shear strength.

Based on the result of 3D elastoplastic finite element analysis, the development of axial displacement ahead of the tunnel face (in terms of the dimensionless parameter Ω) with stability ratio N is shown in Fig. 3 for H/D of 1.5 and 4.5. Two soil profiles were considered: (1) the undrained modulus and shear strength (E_u, c_u) are assumed to be constant with depth; (2) the undrained strength is assumed to increase linearly with depth. The elastic modulus is assumed to be proportional to the undrained strength (i.e., with a constant ratio E_u/c_u). In both cases, the vertical initial stress increases linearly with depth because of the weight of the overburden. As shown in Fig. 3 (for the case of $K_o' = 1$), the behaviour of axial displacement δ_x, if calculated in terms of the dimensionless displacement Ω, is quite insensitive to the tunnel depth (H/D) or the distribution of soil properties. For stability ratio N less than about 2.5, a constant dimensionless displacement Ω of 1.12 is obtained for all the variables considered, and the soil mass ahead of the face remains largely elastic. It may be seen that deforma-

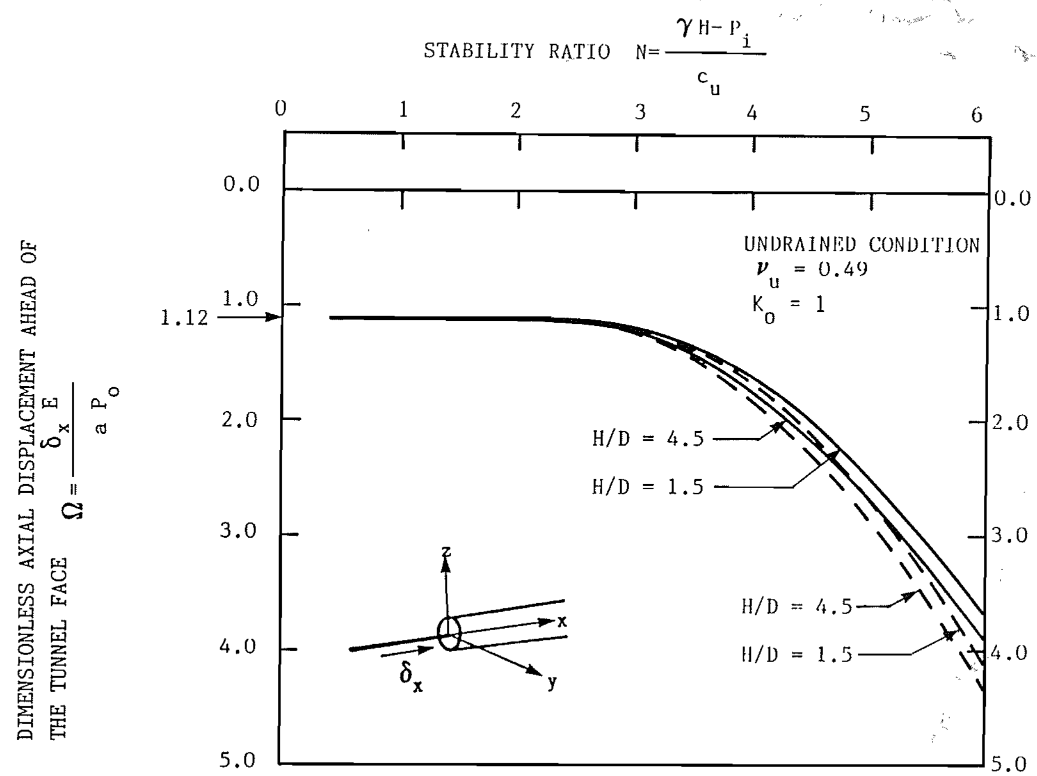

FIG. 3. Dimensionless axial displacement ahead of the tunnel face. —, constant soil parameters; ---, soil parameters increase linearly with depth.

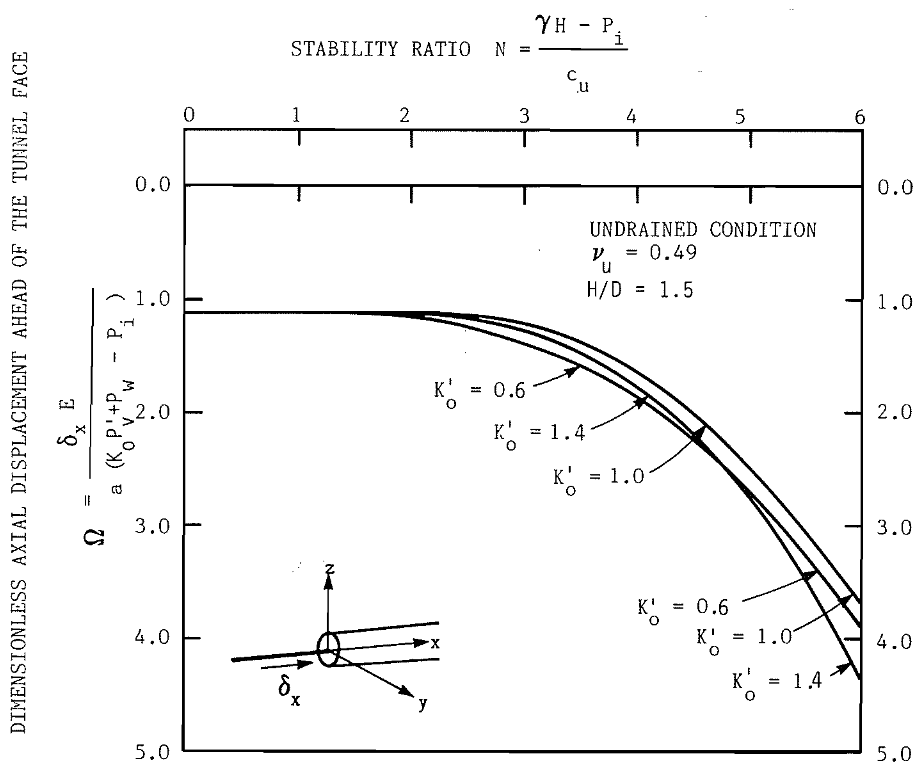

FIG. 4. Dimensionless axial displacement ahead of the tunnel face with various K_o conditions.

tion increases rapidly as N exceeds a value of about 3. This increase in displacement is due to an increase in the extent of the plastic zone. It is of interest to note that the development Ω versus N is quite insensitive to the tunnel depth to diameter ratio (H/D).

The effect of K_o' not equal to unity is shown in Fig. 4 for the case of K_o' of 0.6, 1, and 1.4. The results shown in Fig. 4 are plotted in terms of Ω as defined by [5] (P_o as defined by [6] with corresponding K_o' values). It is found that the dimensionless displacement (Ω) is quite insensitive

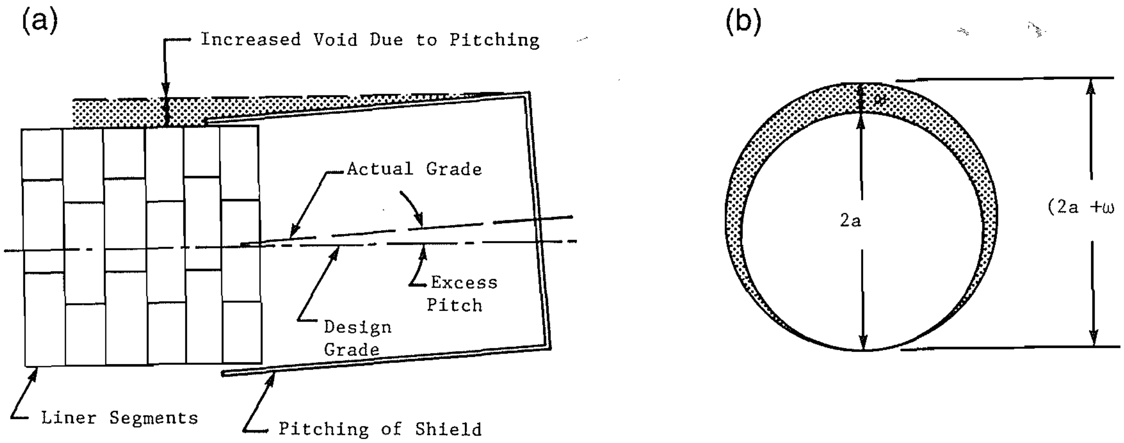

FIG. 5. (*a*) Ground loss owing to pitching of tunnel shield, given as $V_{shield} = 2\pi aL/2$. (*b*) Equivalent transverse section, where $V_{shield} = \pi[a + (\omega/2)]^2 - a^2$.

to various K_o' values, especially when N is less than about 2.5. This indicates that results obtained from the condition of $K_o' = 1$ and given in Fig. 3 can be approximately used for other initial *in situ* stresses conditions provided that P_o is as defined in [6] and K_o' does not vary substantially from the range considered ($0.6 \le K_o' \le 1.4$).

Application of equation [4]

Equation [4] can be used to estimate the equivalent soil movement at the crown (u^*_{3D}) due to 3D movements ahead of the face. The parameter δ_x can be determined by estimating the nondimensional displacement Ω from Fig. 3 for a given stability ratio N and back-calculating δ_x from [5]. If compressed air is used at the tunnel face, the term P_i in [6] and [7] corresponds to the compressed air pressure.

Closed-face tunnelling machines may be equipped with rotary excavators that support the face during excavation. The rate of intrusion of spoil through the machine head can be controlled by the size of the openings in the face. If the size of these openings is controlled in such a way that the rate of advance of the machine is consistent with the volume of soil excavated, then the amount of 3D face loss would be smaller than that calculated using [4].

Face-supported tunnelling machines (e.g., slurry shield, earth and water pressure balance machines) were found to be capable of providing positive outward pressure ahead of the face (Kitamura *et al.* 1981; Fujita 1985; Finno 1983). Thus, ground loss owing to 3D inward movement of soil into the tunnel face can be virtually eliminated by employing these types of machines. However, the exact ground-supporting mechanism and reconsolidation of the remoulded soil resulting from the use of these types of machines requires further investigation.

In [4], u^*_{3D} is determined by assuming the machine is advanced in perfect alignment. This implies that the 3D lost ground is primarily due to the deformation of soil ahead of the tunnel face. If the machine is pitched upward or downward, additional material will be excavated. This quantity is related to workmanship and is denoted by ω in [1]. An approximate method to estimate the maximum value of ω will be discussed in the following section.

Ground losses over the shield

The loss of ground that occurs over the shield corresponds to the volume of soil that is displaced or mined in excess of the diameter of the cutting shield. Causes of such loss in ground are primarily due to alignment problems encountered when steering the shield. One of the common alignment problems is due to the fact that it is a common practice for the tunnel operators to advance the shield at a slightly upward pitch relative to the actual design grade to avoid the diving tendency of the shield. This excessive pitch will cause overcutting of the ground near the crown of the tunnel. As suggested by Cording and Hansmire (1975), this type of ground loss can be estimated by assuming that, over the length of the shield, a point above the crown will settle an amount equal to the length of shield times the pitch of the shield in excess of the actual grade as shown in Fig. 5. The volume of lost ground (V_{shield}) over the shield owing to deviations from design grade can be estimated as

$$[8] \quad V_{shield} = \frac{2\pi aL}{2} \times (\text{excess pitch})$$

where a is the tunnel radius and L is the length of the shield. The unit dimension of V_{shield} is equal to volume of overexcavation per unit length of tunnel advance. An equivalent amount of loss of ground at the transverse section in terms of the workmanship parameter ω can be obtained by equating V_{shield} with the volume as defined in Fig. 5, viz,

$$[9] \quad V_{shield} = \frac{2\pi aL}{2} \times (\text{excess pitch}) = \pi\left[\left(a + \frac{\omega}{2}\right)^2 - a^2\right]$$

Ignoring the secondary term, [9] becomes

$$[10] \quad \omega = L \times (\text{excess pitch})$$

This equation can be used as a guide to estimate the possible value of workmanship parameter ω, if the maximum allowable "excess pitch" is specified by tunnel engineers before the construction, or this equation can be used to back-figure the volume of lost ground (and hence the workmanship parameter) if the record of excess pitch was made available after construction.

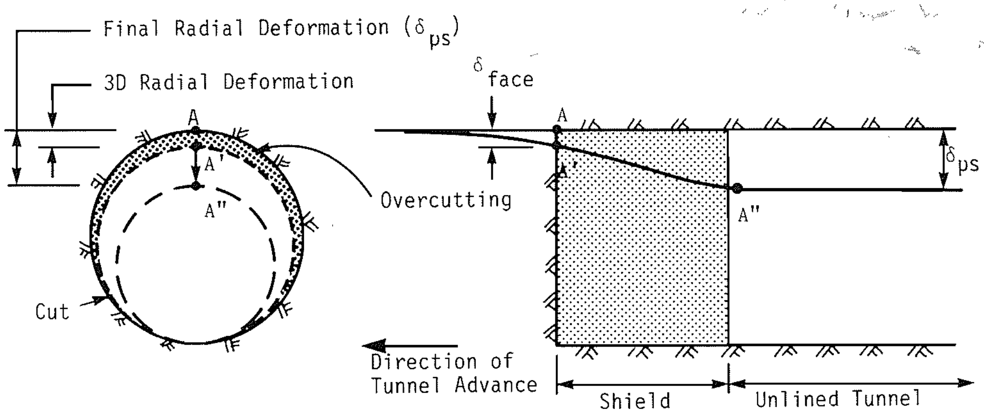

FIG. 6. Three-dimensional radial movement at the tunnel crown; unrestrained deformation.

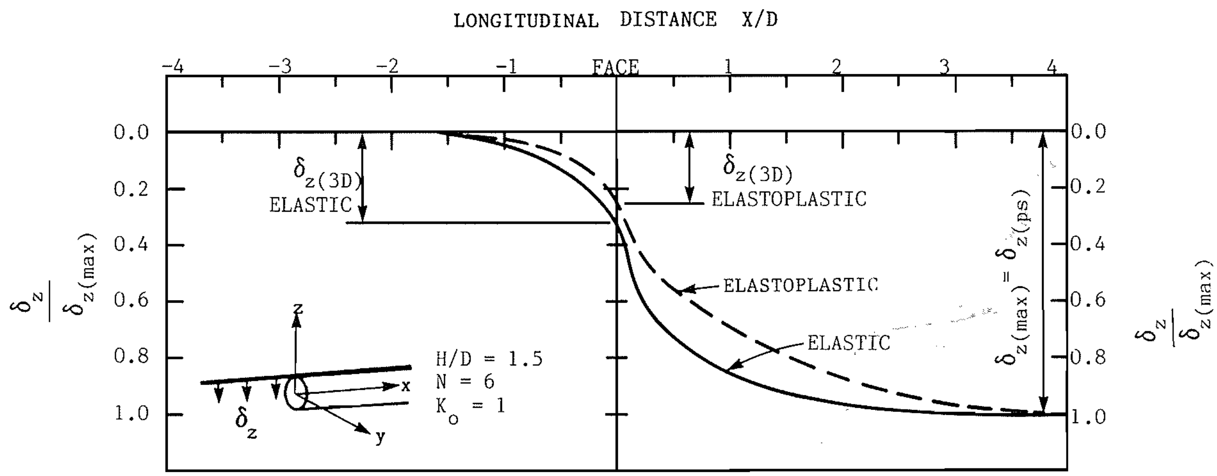

FIG. 7. Normalized displacement profiles along the longitudinal crown axis. Mode *i* deformation: unrestrained displacement.

Additional lost ground can arise due to the irregular upward or downward pitching motion of the shield at an inclination other than the tunnel design grade. Loss of ground in a similar fashion occurs owing to yawing, when the shield is allowed to move irregularly from side to side. The occurence of this irregular motion is generally intermittent and highly dependent on the experience of the workmen controlling the advance of the shield. This irregular motion will cause the shield to over-excavate additional material. This overcutting problem is primarily related to workmanship and cannot be precisely determined prior to construction.

With good tunnelling technique, the radial deformation of the ground is largely restrainted by the stiff structural support of the tunnelling machine and therefore the radial ground loss over the shield is usually quite small. However, intermittent alignment and steering problems will temporarily create an over-excavated zone around the tunnelling machine. This will allow more radial ground movement into the annular void created by the over-excavation. To estimate the radial ground loss induced by these construction problems, the following two idealized ground-deformation modes were examined.

(1) The 3D ground deformations and boundary conditions around a tunnel heading are illustrated in Fig. 6. Consider the movements of a point A located along the crown level of the tunnel opening. As the tunnel face advances forward, point A will move to A′. The total 3D radial deformation of the soil just ahead of the tunnel face would be δ_{face}. Consider the situation where the over-excavation owing to intermittent alignment problem of the shield creates an annular void of the size such that the soil is allowed to deform freely into the tunnel opening without the influence of the tunnel shield and lining, then an unlined tunnel situation will be created temporarily. As the face advances farther, point A′ will move to point A″, as in the case of an unlined tunnel. However, the unlined condition is only created temporarily because of intermittent alignment difficulty of the shield. Construction control in soft-ground tunnelling (i.e., the presence of tunnelling shield and lining) seldom allows the full unlined tunnelling condition to develop. The maximum possible radial deformation of the ground at the tunnel face would be the magnitude of δ_{face}. In practice, this amount of radial intrusion of soil δ_{face} will be over-excavated at the tunnel face by the tunnelling machine, and this will represent the amount of radial ground loss over the shield owing to alignment difficulty. An approximate technique to estimate the magnitude δ_{face} is to apply a factor to the plane strain radial displacement at the crown (i.e., δ_{ps}). This factor can be determined from the results of 3D analyses of unlined tunnels (details have been given by Lee and Rowe 1990*a*). Typical results of the radial deformations of the soil along the crown axis of an unlined tunnel are shown in Fig. 7. For the elastic case, the ratio

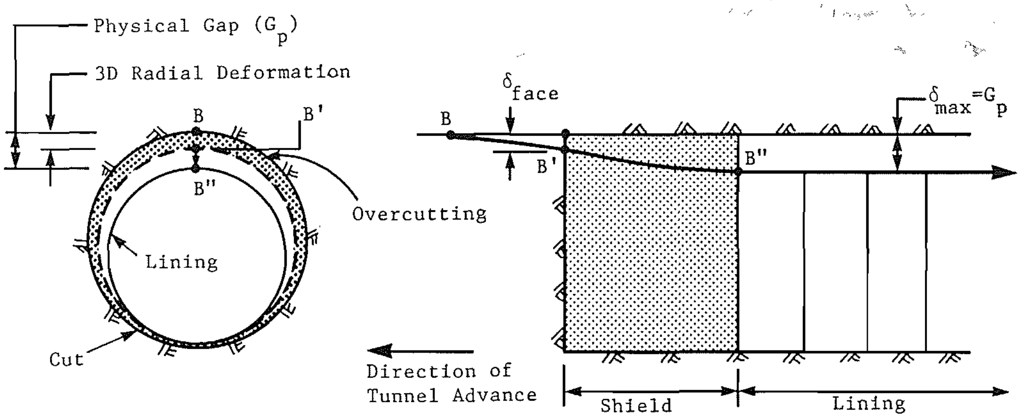

FIG. 8. Three-dimensional radial movement at the tunnel crown; constrained deformation.

of the radial deformation at the face (δ_{face}) to the maximum plane strain displacement (δ_{ps}) is approximately 1/3. A similar ratio was suggested by Lo *et al.* (1984) for elastic deep tunnels. If plasticity is allowed to develop around the tunnel heading, this ratio is reduced to about 0.24–0.26 (depending on the amount of plasticity and H/D ratio, see Lee and Rowe 1990*b* for details). Here, a conservative estimate for the correction factor of 1/3 (regardless of the amount of plasticity) is suggested. Thus the maximum possible effective ground loss over the shield, under this deformation mode and boundary conditions, can be written as

$$[11] \quad \omega \doteqdot \frac{1}{3} u_i$$

where u_i is the elastoplastic plane strain displacement at the crown (equivalent to δ_{ps} as defined in Fig. 6). As suggested by Lo *et al.* (1984), for elastoplastic conditions ($N > 1$), the radial crown displacement u_i is approximately given by

$$[12] \quad \frac{u_i}{a} = 1 - \left(\frac{1}{1 + \frac{2(1+\nu_u)\, c_u}{E_u} \left[\exp\left(\frac{N-1}{2} \right) \right]^2} \right)^{1/2}$$

where E_u and ν_u are the undrained modulus and Poisson's ratio, respectively; a is the radius of the excavated opening; and N is the stability number as defined by [7].

(2) The second scenario of radial deformation around the tunnel shield is illustrated in Fig. 8. Consider the movement of a point B located along the crown level of the opening. As the face advances forward, point B will move to B′. In this scenario, the tunnel is assumed to be lined, and the amount of ground displacement will eventually be limited by the presence of the lining. However, just before the actual contact between the soil and the lining, the ground is allowed to deform freely into the tail void, which is the distance between the crown of the tunnel and the top of the lining (or the so-called physical gap G_p). Consider the situation that an intermittent alignment problem creates an annular void around the shield, so the soil is temporarily allowed to deform into the physical gap. Assuming point B′ lies on the surface of the temporarily created annular void, it will deform into the opening until it comes in contact with the lining at B″. The presence of the lining will arrest the deformation of the soil and transfer all the load into the structure components of the lining. In that case, there will be no further ground movement, and the final deformation BB″ is denoted by δ_{max}. If G_p is greater than the final deformation at B, then the full unlined tunnel condition will develop. This case will then be identical to case 1. If G_p is small, then the radial displacement behind the tunnel shield will be restricted by the lining, and the maximum possible radial displacement (δ_{max}) is equal to the physical gap G_p. With this deformation mode and boundary conditions, the maximum possible radial deformation of the ground at the tunnel face would be the magnitude of δ_{face}. This amount of radial intrusion will be excavated at the tunnel face by the tunnelling machine. The ratio of δ_{face} to the maximum radial deformation behind the tunnel face (δ_{max}, which is equal to the physical gap G_p) can be determined by 3D finite element analysis of a lined tunnel. Figure 9 shows the typical displacement profiles along the crown axis as predicted by finite element analyses for an elastic and an elastoplastic case. The ratio δ_{face}/G_p (or $\delta_{face}/\delta_{max}$) is found to be 0.42 and 0.56 for the elastic and elastoplastic cases, respectively. Results from a number of analyses on different tunnel geometry (H/D ratios) indicate that the ratios δ_{face}/G_p are generally in the range of 0.5–0.6 (see Lee 1989 for details). For simplicity of calculation, a correction factor of 0.6 is suggested. Therefore, under this deformation mode, for typical sizes of physical gap (usually in the range of about 60–200 mm), the maximum possible ground loss over the tunnel shield can be expressed as

$$[13] \quad \omega = 0.6 G_p$$

where G_p is the physical gap of the tunnel system and is equivalent to $(2\Delta + \delta)$ as defined by [1].

Since the extent of overcutting owing to alignment and steering problems is primarily related to quality of workmanship and cannot be precisely determined prior to construction, it is suggested that the two proposed equations (i.e., [11] and [13]) could be used to establish a possible range of workmanship parameter ω. For most of the soft-ground tunnelling situations, the radial displacement of the soil along the tunnel shield will usually be restricted by the presence of the lining. Under most circumstances, [13] (i.e., $\omega = 0.6G_p$) will govern the amount of lost ground over the shield. However, if the physical gap size is greater than the possible unrestrainted deformation at the crown, then full plane strain conditions may be allowed to develop. Thus, the maximum possible ground loss over the tunnel shield may be estimated as

$$[14] \quad \omega = 0.6 G_p \qquad \text{provided that } \omega \le \frac{1}{3} u_i$$

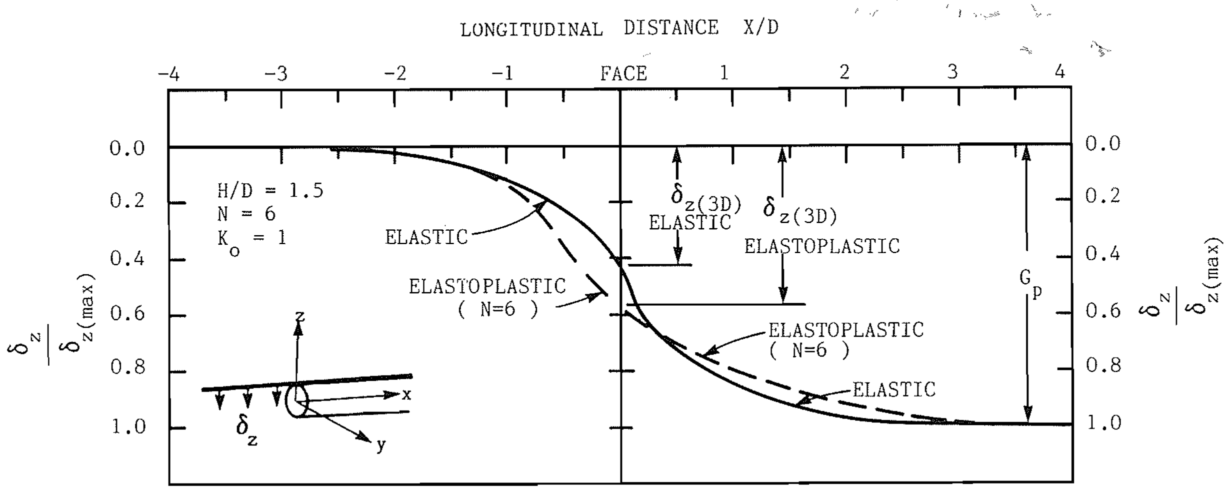

FIG. 9. Normalized displacement profiles along the longitudinal crown axis. Mode *ii* deformation: constrained displacement.

Otherwise, $\omega = \frac{1}{3}u_i$. It should be noted that [11]–[14] are used to estimate the workmanship factor and are quite distinct from the previous estimates of the component of GAP owing to face displacement (eq. [4]).

Radial loss due to overcutting bead

The construction details of the shield tend to control the total volume of ground lost. A particular source of loss arises from the bead, or similar device, welded on the leading edge of the hood and shield to overcut the ground, reduce friction during shield advance, and allow the shield to be steered more easily.

There are three usual cases: no bead, the bead spans the upper 180° of hood, and the bead covers the full circumference (360°) of hood and shield. In each case, the total thickness of the bead should be included in the parameter ω, i.e.,

[15] ω = (value of ω from [14]) + n × (thickness of the bead)

where $n = 1$ when the bead spans only the upper 180° of the hood and $n = 2$ for the full 360° bead.

Ground losses upon the erection of the lining

The loss of ground, which occurs upon erection of the tunnel lining, results because the tunnel lining usually insufficiently replaces the cross-sectional area of the tail of the shield. Tunnel liners are frequently required to be erected under the protection of a shield tail with certain clearance. As the shield moves forward, the change in stress state induced in the clay by the excavation causes the ground to squeeze into the void left by the combination of the thickness of the tail skin (Δ) and the clearance (δ). In most tunnelling situations, the size of this tail void remains a significant factor contributing to the overall settlement. This source of lost ground is represented by the physical gap G_p, which is composed of

[16] $G_p = 2\Delta + \delta$

This component of loss of ground may reduce if an expanded lining is used. Some reduction in ground subsidence has been obtained by backfilling the void with grout upon advance of the shield, thereby decreasing possible ground loss. However, the effectiveness of grouting depends on the rate of soil movement into the void.

Once the soil comes into contact with the tunnel lining, the lining will deflect under the applied loading. It has been observed that the crown of the tunnel usually will compress under this loading, whereas the springline will expand (Ward 1969; Peck *et al.* 1972). This may be considered directly by using the lining flexibility in the finite element analysis or alternatively, if a rigid lining is used in the analysis, the deflected shape (see Peck *et al.* 1972) may be represented as an additional component of lost ground; however, for most cases, the magnitude of this component will not be as significant as the other sources described.

Procedure for estimating the gap parameter

The proposed numerically based approach described in the preceding paragraphs is developed under idealized conditions. In applying the proposed solution technique to practical situations, each case must be carefully considered to evaluate whether the fundamental assumptions are satisfied. In summary, the gap parameter is given by

[1] $\mathrm{GAP} = G_p + u^*_{3D} + \omega$

where $G_p = 2\Delta + \delta$ (see Fig. 2), Δ is thickness of the tailpiece, δ is clearance required for erection of the tailpiece, u^*_{3D} is 3D elastoplastic deformation ($\leq 0.5\delta_x$, from [4]), δ_x is given in Figs. 3 and 4 (see also [5]–[7]), and ω is workmanship factor and is the minimum of $0.6G_p$ and $\frac{1}{3}u_i$, i.e., $\leq 0.6G_p$ (as defined above) and $\leq \frac{1}{3}u_i$ (where u_i is given by [12]).

The following comments may be helpful in using the proposed method in tunnelling projects in clays.

(1) The physical gap G_p, which is equal to the difference between the diameter of the cut surface (based on the dimensions of the tunnelling machine) and the external diameter of the lining, can be evaluated based on geometry of the tunnelling machine (TBM) and lining system.

(2) If the physical gap is small and support follows closely behind the tail of the shield (e.g., the Extruded Concrete Lining Method and the "Injectoshield" technique recently developed in Europe and Japan), or an expanded lining is used, in these cases, the radial crown displacement cannot develop and the main sources of ground loss will be largely restricted to the 3D movement at the tunnel face and workmanship problems. For this case, $0 \leq G_p \leq 2\Delta + \delta$ and will approach 0 if well implemented.

(3) The 3D movement ahead of the tunnel face u^*_{3D} can be estimated by [4], where the value of δ_x can be estimated from the design charts given by Figs. 3 and 4. It should be noted that these results are based on the assumption that the horizontal *in situ* stress ahead of the tunnel face is fully released. For the condition where compressed air is used at the tunnel face, the term P_i in [6] and [7] should correspond to the value of the compressed air pressure. Three-dimensional movement ahead of the face can be virtually eliminated by the use of appropriate construction technique (e.g., appropriate use of an earth pressure balance machine, slurry mole). In these cases, $0 \leq u^*_{3D} < 0.5\delta_x$ and will approach 0 if there is no release of pressure at the face.

(4) With good tunnelling technique, the radial ground loss over the shield is usually quite small. However, if the quality of workmanship is in doubt, or during the early phase of the construction process where the tunnel crew is not sufficiently familiar with the construction technique, the maximum value of the workmanship parameter (ω) can be estimated by the following conditions: $\omega = 0.6G_p$ provided that $\omega \leq \frac{1}{3}u_i$; otherwise, $\omega = \frac{1}{3}u_i$, where u_i is the elastoplastic plane strain displacement at the crown and can be determined by the closed-form solution given by [12], and G_p is the physical gap of the tunnel system (i.e., $G_p = 2\Delta + \delta$).

(5) If an overcutting bead or any similar device is attached to the leading edge of the machine, the total thickness of the bead should be included in the workmanship parameter ω.

(6) The effectiveness of grouting the tailpiece voids depends on the rate of soil movement into the void. In most of the tunnelling cases in soft clays, the grout is injected after soil movements have occurred and hence is ineffective. However, if special care is taken to ensure that the grout is injected before movements occur (e.g., by maintaining compressed air pressure until grouting of the void is achieved, then the gap will be reduced). This reduction in gap may be incorporated into the workmanship term ω as a negative value.

(7) For stiff clays, since the rate of ground settlement near the crown is usually much slower than the average rate of advance of the tunnel shield (Attewell and Farmer 1974; Lo *et al.* 1984), grouting (or pea gravel) can usually be injected into the tailpiece voids without difficulty, and the grouting should be effective. Thus, $G_p = (2\Delta + \delta) = 0$, provided that injection is conducted close behind the tunnel shield so that the unsupported length of tunnel between the rear of the tail and the last grouted portion is small. On the other hand, if a grouting procedure is not adopted, the ground is allowed to deform freely into the tailpiece void. As suggested by Peck (1969), the stability ratio N for stiff clay is usually less than 2 and the soil mass around the opening remains largely in elastic condition. An initial estimate of the crown displacement u_i can be obtained from the plane strain solution, [12]. If the calculated crown displacement u_i is less than the physical gap G_p, plane strain conditions will be satisfied. For this special case, the gap parameter is given by

$$\text{GAP} = u_i \quad \text{if } u_i \leq G_p$$
$$\text{GAP} = G_p \quad \text{if } u_i > G_p$$

(8) Ideally, the elastic and strength parameters should be determined in triaxial tests following the stress path of the soil surrounding the tunnel during excavation. However, for the approximate analysis described herein, it is often adequate to employ vane strength for soft clays and anisotropically consolidated undrained triaxial extension tests, on good quality samples, for the determination of modulus. If conventional tube samples are used, a correction for sample disturbance must be applied. A more detailed discussion about the selection of soil parameters relevant to tunnelling analysis has been given by Ng and Lo (1985) and Rowe and Lee (1989).

Concluding remarks

The components of loss of ground induced by tunnelling in cohesive soils are represented quantitatively by the so-called gap parameter. The gap is the sum of the 3D elastoplastic deformation at the tunnel face, over-excavation of soil around the periphery of the tunnel shield due to poor workmanship, and the physical gap that is related to the machine, shield, and lining geometry. Methods for calculating the 3D deformation and approximate methods for estimating the maximum value of the workmanship parameter are described. The physical gap is determinable once the machine-support system is chosen. This gap parameter can then be used in conjunction with plane strain finite element analysis (e.g., see Rowe *et al.* 1983) to predict the resulting ground deformations.

Following the steps outlined in the present paper, a reasonable estimate of the gap and consequent expected settlements may be made prior to construction. Where significant variations in soil and (or) construction conditions are expected, a range of values of gap may also be determined to calculate the upper and lower bounds of surface settlements.

The application of these numerically based techniques will be discussed in a companion paper (Rowe and Lee 1992) where the range of applicability will be assessed by reference to case histories that encompass a wide range of soil conditions and construction techniques.

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List of symbols

a	tunnel radius
c_u	undrained shear strength of soil
D	tunnel diameter
E	elastic (Young's) modulus of soil
E_u	undrained elastic modulus of soil
GAP(G)	gap parameter
G_p	physical gap ($G_p = 2\Delta + \delta$)
G_{vh}	independent shear modulus
H	height of soil cover from the ground surface to the centre axis of the tunnel
K_o	total coefficient of earth pressure at rest
K_o'	effective coefficient of earth pressure at rest
k_1	parameter to take into account the "doming" effect across the tunnel face, $0 < k_1 < 1$
L	length of the tunnel shield
N	stability ratio
P_o	total stress release at the tunnel face
P_v'	vertical effective stress at the tunnel springline (prior to tunnel construction)
P_w	pore pressure at the tunnel springline (prior to tunnel construction)
P_i	tunnel supporting pressure (if the tunnel is fully excavated, $P_i = 0$; the presence of compressed air at the tunnel face would cause $P_i > 0$)
u_i	radial crown displacement
u^*_{3D}	equivalent three-dimensional movement ahead of the face
V_{shield}	ground loss over the tunnel shield
V_f	ground loss at the tunnel face
ν_u	undrained Poisson's ratio ($\nu_u = 0.5$)
δ	loss of ground due to the clearance required for erection of the lining
δ_{face}	vertical displacement along the crown axis at the tunnel face
δ_{max}	maximum vertical displacement along the crown axis
δ_{ps}	vertical displacement along the crown axis under plane strain conditions
δ_x	maximum axial intrusion of soil at the tunnel face
Δ	thickness of the tail piece
γ	unit weight of soil
ω	workmanship factor
Ω	dimensionless displacement parameter

Appendix. Equation to estimate ground losses ahead of the tunnel face

The volume of soil that intrudes into the tunnel face owing to pressure release at the face will eventually be excavated. Thus, there is a volume of lost ground equal to the amount of over-excavated material at the face called the face loss, V_f. The face loss can be represented by the product of the area of the face (πa^2) and the soil intrusion distance per unit length of tunnel advance ($\delta_x/\Delta x$), if soil intrusion is assumed to take place uniformly over the whole face area. In practice, because of the geometric constraint provided around the heading and drag at the cutting end of the shield, a clay soil will tend to bulge at the centre (Attewell and Farmer 1974). A factor k_1, where $0 < k_1 < 1$, could be

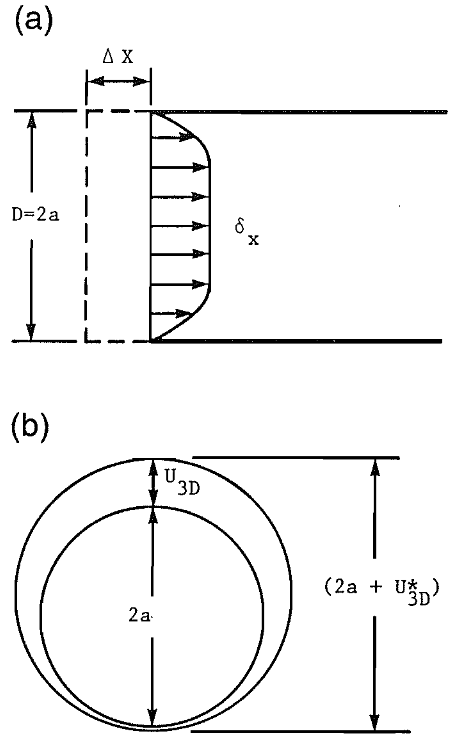

FIG. A1. Approximate method to incorporate three-dimensional face loss into equivalent radial ground loss. (a) Tunnel heading; face loss $V_f = \pi a^2 k_1(\delta_x/\Delta x)$. ($b$) Equivalent transverse section; equivalent face loss $V_f = \pi\{[a + (u^*_{3D}/2)]^2 - a^2\}$.

applied to take this doming effect into account, so V_f can be expressed as

$$[A1] \quad V_f = (\pi a^2)k_1\frac{\delta_x}{\Delta x}$$

where δ_x is the maximum axial intrusion of soil at the tunnel face arising from a tunnel advance, Δx (i.e., Δx is the distance of step size adopted in the step-by-step incremental finite element analysis). The factor k_1 is a measure of the uniformity of the axial displacement that occurred along the tunnel face. Theoretically, k_1 can be determined by field measurement. However, as shown by the finite element analysis, intrusion of soil near the centre of the tunnel face is quite uniform; an assumption of $k_1 = 1$ should give a conservative but reasonable approximation.

The unit dimension of V_f is equal to displaced volume per unit length of tunnel advance. An equivalent amount of loss of ground at the transverse section can be obtained by equating V_f with the volume as defined in Fig. A1. Therefore,

$$[A2] \quad V_f = (\pi a^2)k_1\frac{\delta_x}{\Delta x} = \left[\pi\left(a + \frac{u^*_{3D}}{2}\right)^2 - \pi a^2\right]$$

Ignoring the secondary term, [A2] becomes

$$[A3] \quad u^*_{3D} = ak_1\frac{\delta_x}{\Delta x}$$

Based on [A3], the 3D movements ahead of the face can be approximately represented by u^*_{3D} if the value of $\delta_x/\Delta x$ is determined.

Numerically, the ratio of $\delta_x/\Delta x$ can be determined by a step-by-step incremental excavation technique. However, the selection of an appropriate step size (Δx) is a complex task, since the real tunnelling situation is continuous and it is highly dependent on the actual construction procedure and the rate of tunnel advance (which in turn depends on the condition of grounds, size of the tunnel, and the experience of the workman controlling the tunnelling machine). A conservative but realistic approach is to adopt the maximum possible ratio of $\delta_x/\Delta x$ that would develop in an advancing tunnel [A3]. Based on the results of 3D elastoplastic finite element analysis, Lee (1989) observed that the critical step size (Δx) is approximately equal to one tunnel diameter ahead of the face. Based on this result, it can be argued that for the case of single-step instantaneous tunnelling simulation (since the initial stress ahead of the face is completely removed in a single step), maximum 3D ground effect would be approximately predicted if Δx in [A3] is assumed to be $1.0D$ (one tunnel diameter) ($D = 2a$). The ratio of $\delta_x/\Delta x$ in [A3] can, therefore, be approximately obtained by a single-step excavation simulation to determine the maximum soil intrusion at the tunnel face (δ_x) and, assuming Δx to be D, [A3] can be simplified as

$$[A4] \quad u^*_{3D} = \frac{k_1}{2}\delta_x$$