Subsidence owing to tunnelling. II. Evaluation of a prediction technique

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Received April 3, 1991

Accepted June 1, 1991

A numerical and a simplified empirical design procedure (based on the concept of the gap parameter) for predicting settlement above tunnels constructed in soft ground are evaluated with reference to 14 case histories. These records encompass very stiff to soft clays. A comparison of observed and calculated behaviour indicates that the proposed numerical and empirical approaches are capable of providing a reasonable estimate of the gap and the surface settlement provided that the soil parameters are reliably determined. It is suggested that the simplified procedure may be used for preliminary design. The gap parameter can also be used in conjunction with more sophisticated numerical methods to predict the variation in settlement with position and depth at critical sections of the tunnelling project.

Key words: tunnelling, settlement, subsidence, clays, soft-ground analysis, design, case history, finite element.

En partant de 14 histoires de cas, l'on évalue une procédure de conception numérique, et une procédure empirique simplifiée (basée sur le concept du paramètre gap) pour prédire le tassement au-dessus de tunnels construits dans le sol mou. Ces cas comprennent des argiles très raides. Une comparaison du comportement calculé et observé indique que les approches numérique et empirique proposées peuvent fournir une estimation raisonnable du gap et du tassement à la surface pourvu que les paramètres du sol soient déterminés de façon fiable. L'on suggère d'utiliser la procédure simplifiée pour le calcul préliminaire. Le paramètre gap peut aussi être utilisé avec des méthodes numériques plus élaborées pour prédire la variation du tassement en fonction de la position et de la profondeur aux sections critiques du projet de tunnel.

Mots clés : percement de tunnel, tassement, affaissement, argiles, analyse de terrain mou, conception, histoire de cas, éléments finis.

[Traduit par la rédaction]

Can. Geotech. J. **29**, 941–954 (1992)

Introduction

Experience has shown that ground subsidence invariably occurs above tunnels constructed in soft ground. At present, estimates of settlement are often based on an empirical relationship (Peck 1969), which assumes that the transverse surface settlement profile follows a normal probability curve. For the estimation of the maximum surface settlement, it has been suggested (Peck *et al.* 1972) that the volume of the surface settlement trough may be taken as 1–2% of the volume of soil excavated. With appropriate engineering judgement, this assumption could lead to adequate results. Nonetheless, settlement volumes as high as 40% of the excavated tunnel volume have been reported (Cording and Hansmire 1975).

The fact that there is such a variation in empirical observations exemplifies the need for a more logical approach to tunnel settlement prediction. Finite element techniques are, in principle, well suited to the determination of both lateral and vertical distributions of deformations caused by tunnelling. Useful work concerning the modelling of shield tunnel behaviour has been conducted by a number of workers, including Ghaboussi *et al.* (1978), Rowe *et al.* (1983), and Clough *et al.* (1985). Although various numerical modelling techniques have been proposed, a number of difficulties have also been encountered in the application of the techniques, since the real situation is truly three-dimensional (3D), involves nonlinear soil behaviour, and is, to a large extent, highly indeterminant, since it depends on the experience of the workmen controlling the tunnelling machine. In particular, the size of the disturbed zone around the tunnel and the extent of overcutting which result from the use of a closed-face tunnelling machine are primarily related to workmanship and cannot be precisely determined prior to construction.

Based on the work of Rowe *et al.* (1983) and Lo *et al.* (1984), a parameter called the gap may be defined, where the gap represents the vertical displacement above the crown of the tunnel and is a measure of ground loss owing to tunnelling. Lee *et al.* (1992) extended this approach using the results of elastoplastic 3D finite element analyses to develop a simple design procedure suitable for evaluating the gap parameter for tunnels constructed in cohesive soils under undrained conditions. This gap parameter can then be used in conjunction with transverse plane strain finite element methods (such as that developed by Rowe *et al.* 1983) or empirical relationship (such as that developed by Lo *et al.* 1984; Ng 1991) to predict the resulting ground deformations.

The objective of the present paper is to evaluate the procedure proposed by Lee *et al.* (1992) for estimating the gap and predicting the undrained settlement owing to shield tunnelling. A number of case histories are examined that encompass very stiff to soft clays and corresponding ground response ranging from elastic to predominantly plastic behaviour.

The theoretical development of the gap parameter has been discussed in detail in the companion paper (Lee *et al.* 1992). For the sake of clarity in this paper, the basic equations used for obtaining the gap parameter are summarized below.

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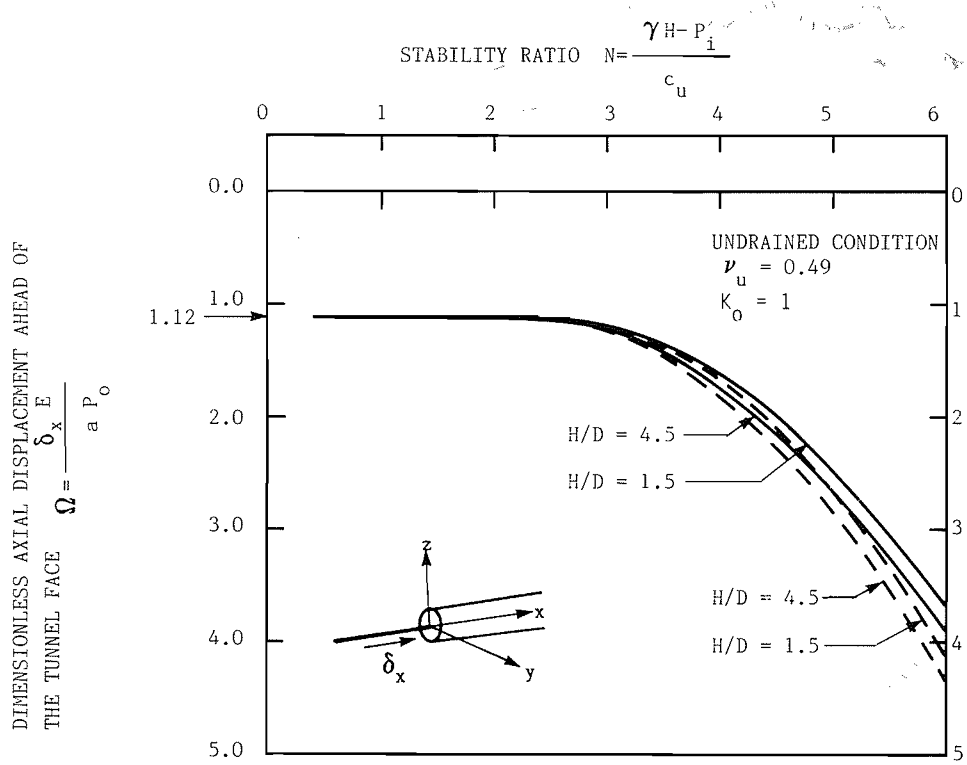

FIG. 1. Dimensionless axial displacement ahead of the tunnel face (after Lee *et al.* 1992). ——, constant soil parameters; - - -, soil parameters increase linearly with depth.

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 owing to (*i*) the difference between the diameter of the tunnelling machine and that of the lining, (*ii*) 3D soil movements ahead of the tunnel face, and (*iii*) alignment problems encountered when steering the shield. The net effect of these factors may be approximately incorporated in a two-dimensional (2D) 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 that will eventually become the crown of the excavated surface; in other words, approximately the maximum crown settlement. In most practical situations the loss of the 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 owing to 3D movement into the face and overcutting owing to alignment problems encountered when steering the shield. Thus the gap parameter (GAP) is defined (Lee *et al.* 1992) as

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

where G_p is the 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), given as

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

where Δ is the thickness of the tail piece, δ is the clearance required for erection of the tail piece, and u_{3D}^* is the three-dimensional elastoplastic deformation, where

$$[3a] \quad u_{3D}^* \leq 0.5\delta_x$$

where δ_x is the face intrusion, given as

$$[3b] \quad \delta_x = \frac{\Omega a P_o}{E}$$

where Ω is a dimensionless displacement factor evaluated from Fig. 1, a is tunnel radius (diameter $D = 2a$), E is Young's modulus (typically the undrained modulus in extension, $E = E_u$), and

$$[3c] \quad P_o = K_o'P_v' + P_w - P_i$$

where K_o' is the effective coefficient of earth pressure at rest, P_v' and P_w are the vertical effective stress and porewater pressure, respectively, at springline of the tunnel, and P_i is the tunnel support pressure. The stability ratio N from Fig. 1 is defined as $(\gamma H - P_i)/c_u$, where γ and c_u are the representative unit weight and undrained shear strength of the soil, respectively, and H is the distance from ground surface to springline of the tunnel. The workmanship factor ω in [1] is defined as

$$[4a] \quad \omega \leq 0.6G_p$$

and

$$[4b] \quad \omega \leq \frac{u_i}{3}$$

where

$$[4c] \quad u_i = a\left[1 \left\{\frac{1}{1 + \frac{2(1+\nu_u)c_u}{E_u}\left[\exp\left(\frac{N-1}{2}\right)\right]^2}\right\}^{1/2}\right]$$

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

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

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 2D (transverse section) finite element analyses or (*ii*) used in conjunction with empirical relationships.

Details concerning the incorporation of the gap parameter into 2D 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 18 case histories. The relationship between settlement ratio δ_{max}/δ_c and dimensionless tunnel parameter $H/2aN$ (where H, a, and N are as previously defined) for tunnels in clays is reproduced in Fig. 2. It is noted that, for soft clays, the average δ_{max}/δ_c ratio is about 0.33, regardless of the dimensionless parameter $H/2aN$. This relationship of $\delta_{max} = \delta_c/3 = \mathrm{GAP}/3$ for soft clays suggests the likely improvement in maximum surface settlement corresponding to a 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 Fig. 2). The recorded ratios varied from 0.1 to 0.7. In general, the absolute magnitude of settlement associated with hard clay is usually small.

Assessment of the proposed procedure using field case histories

To assess the applicability of the proposed approach for estimating the gap and hence predicting settlement, a number of case histories encompassing soft to very stiff clay and various construction techniques have been selected for analysis. In the following sections, each case record will be examined and the results will then be synthesized. Some of the case records reported in this paper have previously been examined by Rowe and Kack (1983) and Ng *et al.* (1986). The major difference between this and the earlier papers is that in the previous investigations the gap parameter was estimated using engineering judgement or back-figuring the gap from the observed surface settlements, whereas in this paper the gap is determined directly from the theoretical technique proposed by Lee *et al.* (1992). It is noted that the soil parameters adopted in the present analysis are previously published values and were not in anyway adjusted to get fits to the observations.

Thunder Bay tunnel

In 1976, a 3.3 km long sanitary trunk sewer was constructed through soft clay in the city of Thunder Bay, Canada. A full-face tunnelling machine was used in the excavation, and the tunnel was supported by an unbolted, precast, segmented concrete lining. Two arrays of instrumentation, referred to as array 1 and array 2, were installed

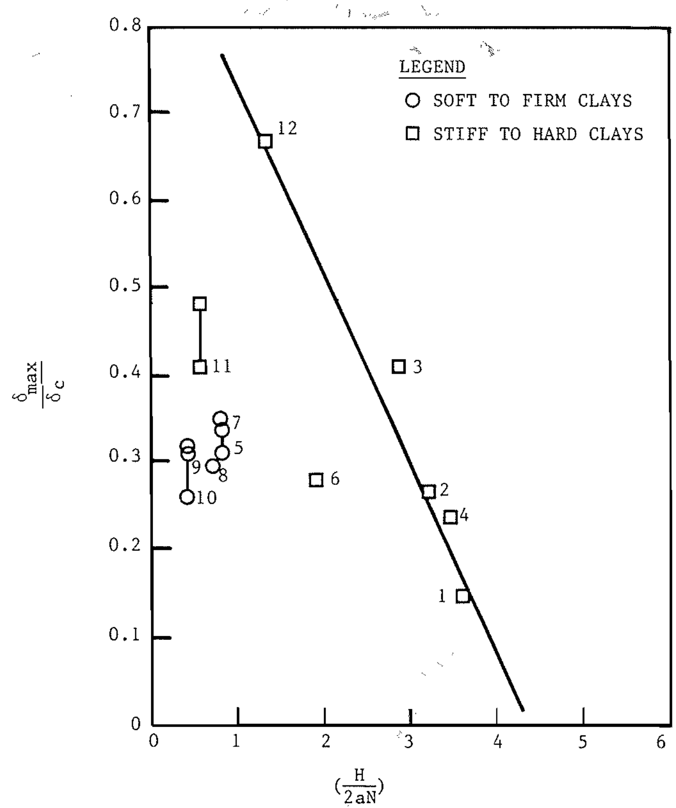

FIG. 2. Relationship between (δ_{max}/δ_c) and ($H/2aN$) (modified from Ng 1991). Numbers correspond to those in Tables 2 and 3.

(Belshaw and Palmer 1978; Palmer and Belshaw 1989). The tunnelling procedure at arrays 1 and 2 was technically identical; however, array 1 was only 60 m away from the first shaft. Since the crew members were not yet familiar with the construction technique, the tunnel advance at array 1 was slow and intermittent. Both alignment and steering problems of the shield were reported when the excavation proceeded near the instrumented section of array 1. In contrast, at array 2, 21 m of tunnel was completed in one 8-h shift, and no problems were reported.

The mined diameter D, of the tunnel was 2.47, and the depth H of the tunnel axis was 10.7 m at array 1 and 10.5 m at array 2, giving H/D ratios of 4.33 and 4.25, respectively. The tunnel is located in a soft grey silty clay deposit that underlies postglacial fluvial deposits of loose sand and silty sand. The grey silty clay is slightly overconsolidated, vane strength varied from 25 to 50 kPa, with a value of about 40 kPa at the axis of the tunnel. The stability ratio N of the tunnel is 4.8. The undrained modulus E_u determined from anisotropically consolidated undrained triaxial tests is about 10 MPa, and the corresponding ratio of E_u/c_u is 400. Beneath the invert of the tunnel is a layer of varved clay that extends to shale bedrock.

Estimation of the gap parameter

Array 1 — The mined diameter was 2.47 m, whereas the outside diameter of the completed lining was 2.38 m. Thus the physical gap (i.e., clearance ($2\Delta + \delta$)) between the concrete lining and the excavated surface was $G_p = 90$ mm.

The tunnel boring machine employed in the Thunder Bay project was equipped with a rotary excavator that supported the face during excavation. The rate of intrusion of spoil

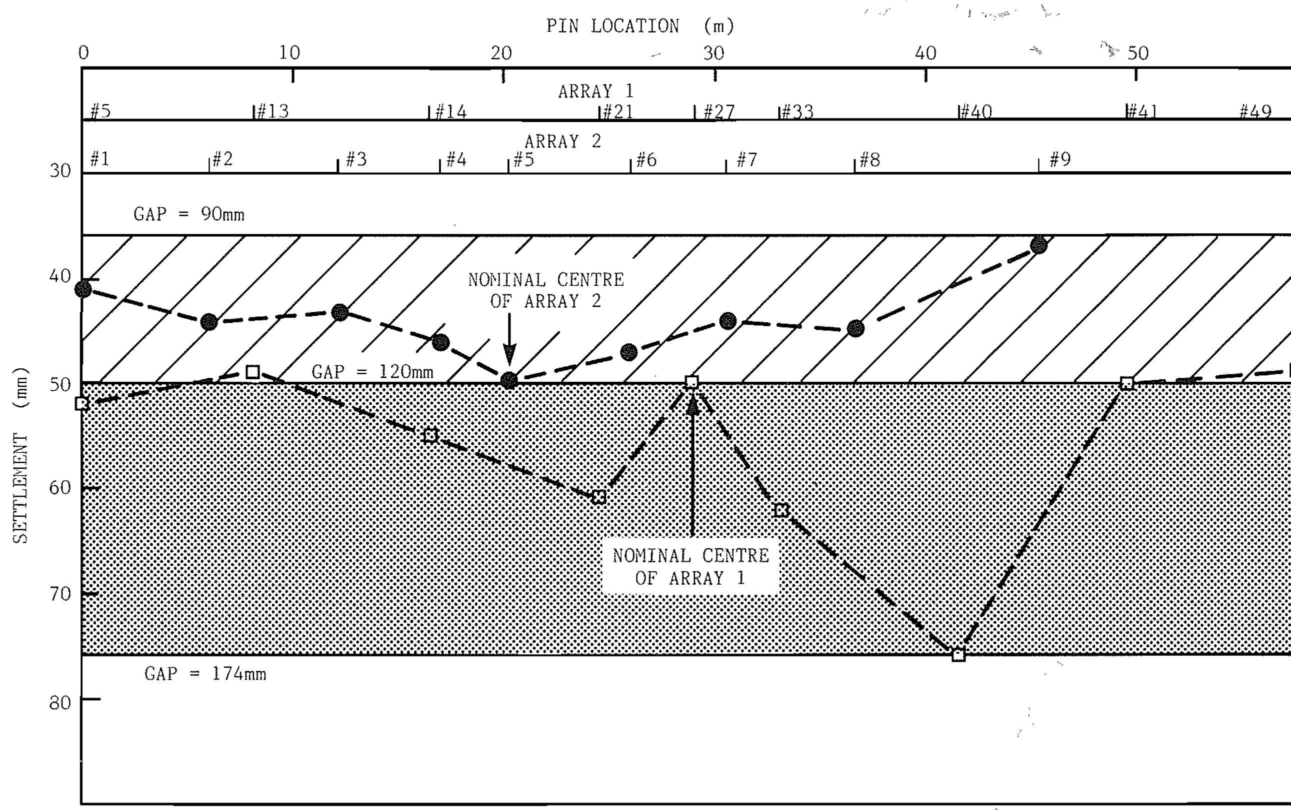

FIG. 3. Ranges of surface settlement predicted along tunnel axis at arrays 1 and 2. (Field data from Belshaw and Palmer 1978 and Palmer and Belshaw 1980).

through the machine head could be controlled by the size of the opening at the face. If the size of this opening is controlled such that the rate of advance of the machine was consistent with the volume of soil excavated, 3D movement at and in front of the shield (u^*_{3D}) would be essentially zero. However, the operators were inexperienced in the use of the machine at array 1 and, as a result of construction difficulties, it is expected that the full 3D ground loss would have occurred ahead of the tunnel face. The equivalent 3D ground loss (u^*_{3D} = 30 mm) can be obtained from Fig. 1 and [3].

An estimate of the maximum radial ground loss over the shield (ω) owing to the alignment and steering problems can be estimated based on the physical gap G_p = 90 mm and the calculated plane strain radial displacement (u_i = 198 mm) from [4]. Thus the maximum likely value of ω is given by $\omega \leq 0.6G_p$ = 54 mm ([4*a*]; G_p = 90 mm from above), provided that $\omega \leq \frac{1}{3}u_i$ = 66 mm ([4*b*]; u_i = 198 mm from [4*c*]), and hence $\omega \leq$ 54 mm. Thus, adopting G_p = 90 mm, u^*_{3D} = 30 mm, and ω = 54 mm, an upper bound value for the total gap parameter at array 1 is estimated (eq. [1]) to be 174 mm. The lower bound gap parameter at array 1 (assuming no overcutting: ω = 0) is taken to be the sum of the physical gap and the 3D movement ahead of the face (i.e., $G_p + u^*_{3D}$) and is equal to 120 mm. Thus the gap parameter for array 1 is expected to lie between 120 (assuming no excessive ground losses owing to overcutting) and 174 mm (assuming the maximum reasonable amount of radial loss over the shield would take place).

Array 2 — By the time the tunnel excavation proceeded near array 2, the construction workers had gained considerable experience with the equipment and good tunnelling technique was reported for array 2. Thus, the parameter ω, which accounts for poor workmanship, can be assumed to be zero. Thus a reasonable lower bound estimate of the gap parameter would be 90 mm (i.e., the size of physical gap G_p). However, without very special care, there will usually be 3D movement occurring in front of the shield during face advance; the actual amount of face movement will depend on the size of the opening at the rotary excavator that supports the face during excavation. Provided there is no excessive overcutting, the maximum possible axial intrusion of soil into the tunnel face can be represented by the parameter u^*_{3D} calculated for array 1. Thus a reasonable upper bound value of the gap parameter at array 2 is 120 mm (i.e., $G_p + u^*_{3D}$), giving a range of total GAP at array 2 of between 90 and 120 mm.

Finite element analysis

The selection of soil parameters relevant to the Thunder Bay tunnel has been described by Ng *et al.* (1986), and these values (see Fig. 10 of Ng *et al.* 1986) are adopted in this present paper for the finite element analysis of both instrumented arrays. The soil–structure interaction theory proposed by Rowe *et al.* (1983) was adopted to simulate the closure of the gap and interaction with the tunnel lining.

The surface-settlement data reported by Palmer and Belshaw (1980) and Belshaw and Palmer (1978) are shown in Fig. 3. It may be seen that substantial variations occurred along the centrelines of the two arrays, but the settlements in array 2 are significantly smaller. The ranges of maximum surface settlements predicted by the finite element analyses with the total gap parameter of 90–120 mm for array 2 and 120–174 mm for array 1 are also plotted in Fig. 3. The measured range of settlements lies within the bounds of calcu-

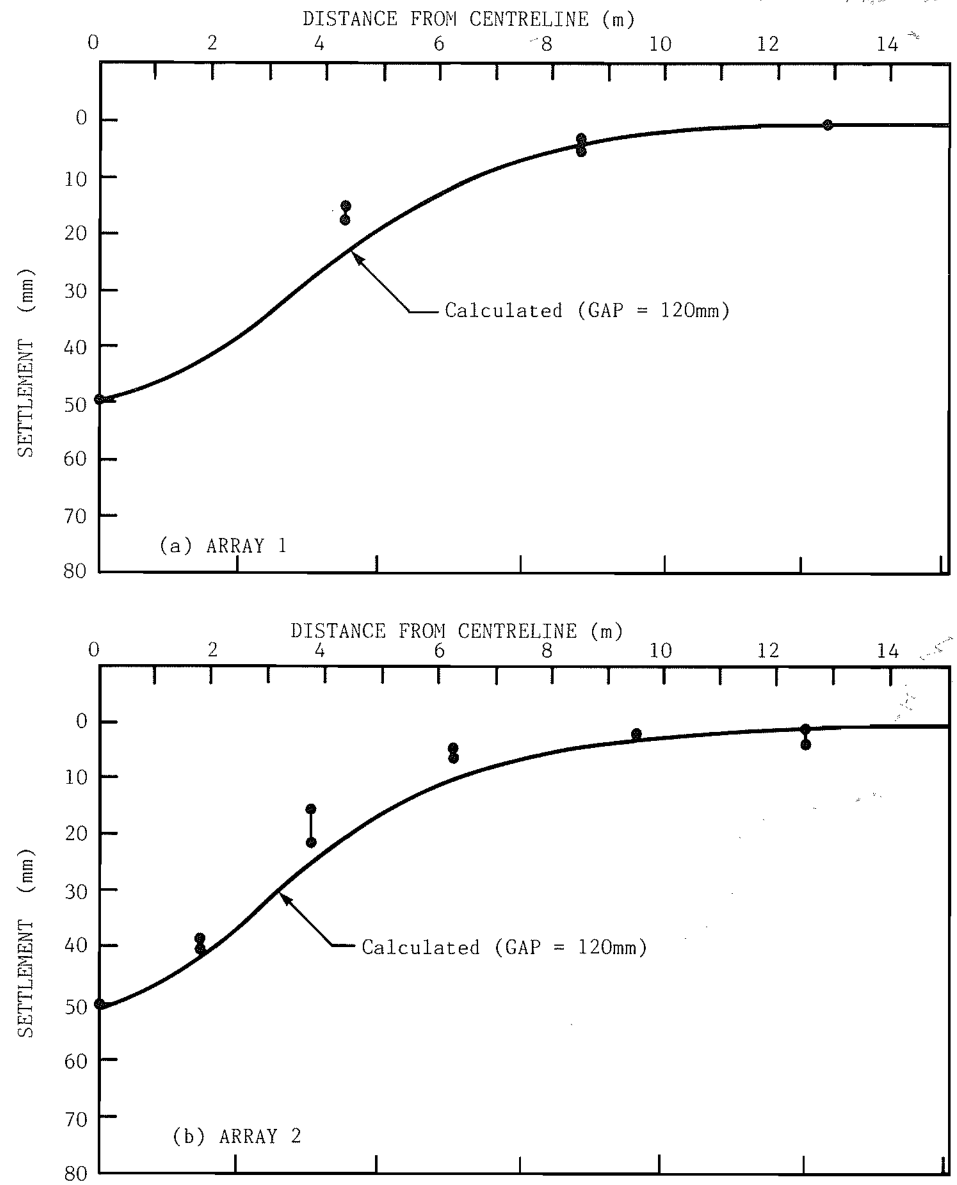

FIG. 4. Observed and calculated undrained surface-settlement profile. (*a*) Array 1: tunnel face at 12.8 m; time = 4 days. (*b*) Array 2: tunnel face at 15.2 m; time = 1 day. (Field data from Belshaw and Palmer 1978 and Palmer and Belshaw 1980.)

lated settlements for both of the arrays. The calculated maximum surface settlement δ_{max} = 76 mm corresponding to GAP = 174 mm is almost identical to the maximum measured settlement at settlement pin 40. This may indicate that, near pin 40, significant overexcavation, resulting from alignment problems, may have occurred. The smallest settlement observed at array 1 is 49 mm. This agrees well within the calculated settlement corresponding to the gap parameter of 120 mm. The variation in settlement between the two arrays, and within each array, may therefore be attributed to the variation of the gap parameter within the range established by the proposed method. It may be noted that, by coincidence, the detailed instrumented section at the nominal centre of array 1 is located in the area where settlement is relatively small, whereas a relatively large settlement (compared with the range of measured settlements within array 2) was recorded at the nominal centre of array 2. This coincidence leads to the use of a gap parameter of 120 mm for analyzing the undrained performance at the nominal centre of both array 1 and array 2.

The calculated undrained surface settlement profiles at arrays 1 and 2 are shown together with the observed settlements in Figs. 4*a* and 4*b*, respectively. The settlement curves for a gap parameter of 120 mm are in good agreement with the observed settlements. (In fact, the calculated results are very similar to those obtained by Ng *et al.* 1986.) Reasonable agreement between the observed and calculated centreline settlements with depth can also be observed for both of the arrays as shown in Fig. 5.

The results of maximum surface settlements predicted by the empirical correlation as described in the proceeding section are compared with the results obtained by field observations and finite element analysis in Table 1. Results predicted by the empirical correlation (δ_{max} = GAP/3) slightly underestimate δ_{max} by about 20% for both arrays 1 and 2. The finite element analysis provided excellent agreement with the

TABLE 1. Estimation of maximum surface settlement for various case histories

Tunnel	Ground condition	H/D	N	GAP (mm)	Surface settlement δ_{max} (mm): Predicted: Empirical	Surface settlement δ_{max} (mm): Predicted: Finite element analysis	Surface settlement δ_{max} (mm): Observed	$\frac{\Delta V}{V}$ (%)
1. Thunder Bay								
Array 1	Soft silty clay	4.3	4.8	120–174	40–58[a]	50–76	49–76	5.5
2. Array 2	Soft silty clay	4.25	4.8	90–120	30–40[a]	36–50	37–50	5.5
3. Manuel Gonzalez Tunnel	Soft, highly compressible, lacustrine clay	4.0	5.0	288	96[a]	103	105	~30
4. Lower Market Street	Recent soft marine silty clay	3.4	4.6–6	70–160	23–53[a]	22–53	25–50	2.7
5. Green Park Underground	Stiff London clay	7.1	2.2	24–27	3–4[b]	6	5–7	1.3

[a]Predicted by empirical relationship δ_{max} = GAP/3.
[b]Predicted by empirical relationship as shown in Fig. 2.

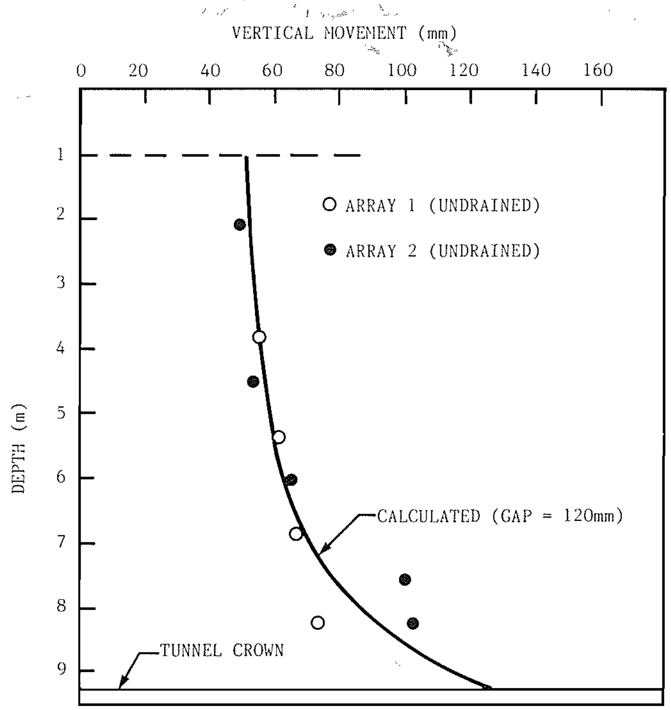

FIG. 5. Predicted and observed vertical displacement along the vertical centre axis with depth, Thunder Bay tunnel (arrays 1 and 2). (Field data from Belshaw and Palmer 1978 and Palmer and Belshaw 1980.)

observed settlement ranges for both arrays. The measured volume of the surface settlement trough (ΔV) was about 5.5% of the volume of soil excavated (V), compared with 1–2% as suggested by Peck *et al.* (1972).

Manuel Gonzalez tunnel, Mexico City

The surface settlement owing to the construction of a shallow sewer tunnel is a soft, highly compressible volcanic lacustrine clay has been reported by Saenz and Utesa (1971). These investigators discussed the behaviour of a number of tunnels constructed in Mexico City at approximately the same time (1969); however, attention will be restricted here to the Manuel Gonzalez syphon tunnel (H/D = 4) because of its geometric similarity to Thunder Bay tunnel (H/D = 4.3) and the significant difference in the surface settlement.

Soil conditions and method of construction

The tunnel was constructed using a 2.95 m diameter shield equipped with a rotating full-face excavator. The oscillating cutters took up one-third of the area of the face. The tunnel lining consisted of steel segments 6 mm thick and 5-cm flange reinforcement as a primary support, followed by a concrete secondary lining that was cast after the excavation was completed. The primary lining, with a diameter of approximately 2.82 m, was installed within the protective shield. A sand–cement grout was injected 8 m behind the shield at a pressure approximately equal to the overburden pressure. The degree of success of the grouting technique was not mentioned, but it may be noted that grouting is seldom effective in soft clays (Lo *et al.* 1984).

The Mexico City clay is very compressible, having a high montmorillonite content, a water content close to 300%, and a plasticity index as high as 400 (Saenz and Utesa 1971). On the basis of published data relating to Mexico City clay

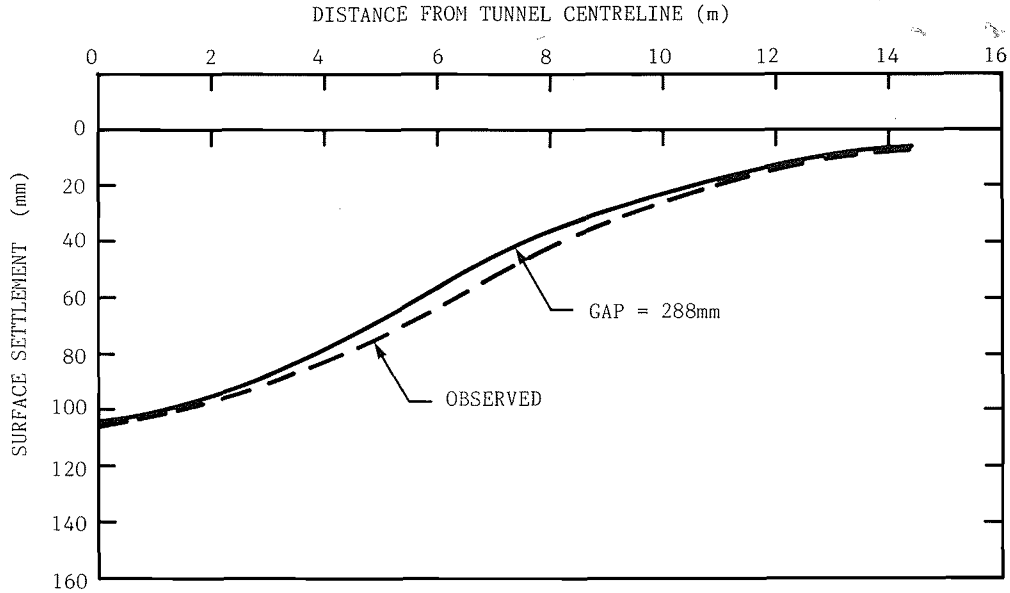

FIG. 6. Comparison of observed and predicted short-term surface settlement, Manuel Gonzalez Tunnel. (Observed response based on Saenz and Utesa 1971.)

(Alberro and Santoyo 1973; Mesri *et al.* 1975), it would appear that the soil is slightly overconsolidated, with a typical overconsolidation ratio of 1.3, an undrained shear strength of 35 kPa, and an undrained elastic modulus of 4 MPa. The stability ratio N is about 5.

The depth of soil cover to the tunnel axis at the measured location was 11.7 m, and the entire tunnel was situated within the soft clay stratum. The diameter of the excavated opening was 2.95 m, resulting in a H/D ratio approximately equal to 4. The soil profile adopted in the present analysis is based on Fig. 6 of Rowe and Kack (1983), with appropriate adjustments being made for the fact that an undrained analysis is being performed in this case, whereas Rowe and Kack (1983) were focussing their attention on a drained analysis.

Estimation of the gap parameter

The theoretical size of the annular void represents a created volume equal to the difference between the diameter of the shield (2.95 m) and the diameter of the lining (2.82 m). This volume is represented by a physical gap $G_p = 130$ mm. However, both field reports and settlement observations indicate that there were considerable difficulties with shield alignment, giving rise to both overcutting and overstressing of the soil. It would appear that these problems result in a total gap parameter composed of G_p, u^*_{3D}, and ω.

Based on Fig. 1 and [3], the equivalent displacement (u^*_{3D}) owing to 3D face loss is estimated as 80 mm. The plane strain displacement at the crown (u_i by eq. [4*c*]) is 530 mm. Thus from [4*a*] and [4*b*], $\omega \leq 0.6G_p = 78$ mm, provided that $\omega \leq u_i/3 = 177$ mm, yielding a value of $\omega = 78$ mm. Thus, the total gap parameter can be determined from [1], viz., GAP $= G_p + u^*_{3D} + \omega = 130 + 80 + 78 = 288$ mm.

Finite element analysis

The surface-settlement profile, observed 28 days after tunnel construction, is given in Fig. 6. The volume of the settlement trough was approximately 2.0 m^3/m. This represents 30% of the excavated tunnel volume and is at least 10 times

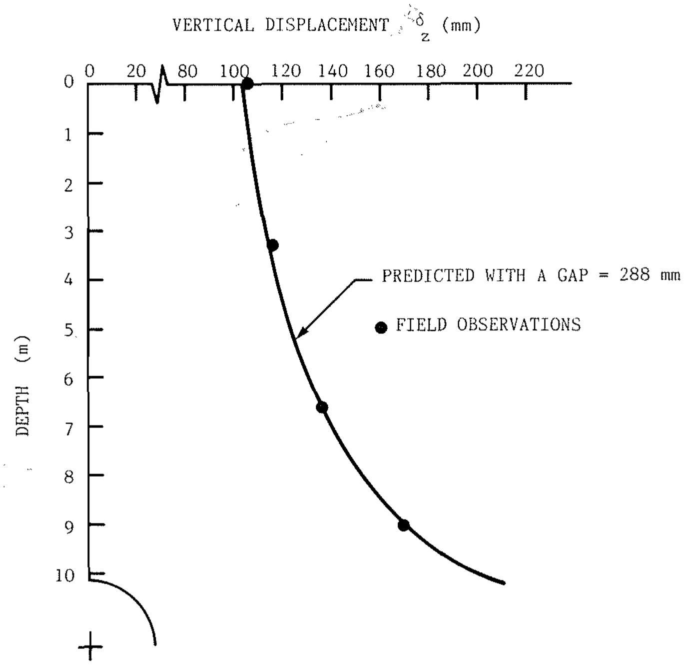

FIG. 7. Observed and predicted vertical displacement along the vertical centreline axis with depth, Manuel Gonzalez Tunnel. (Field data from Saenz and Utesa 1971.)

the volume that would normally be expected for soft firm clays based on Peck (1969). The surface-settlement trough predicted with the total gap parameter of 288 mm is also illustrated in Fig. 6. It may be seen that the predicted settlement is in good agreement with the observed settlement. The predicted and observed vertical centreline displacements with depth are shown in Fig. 7; as in the case of surface settlement, good agreement between the predicted and field-observed values is obtained.

The lateral movement of soil at the axis level predicted with the total gap parameter of 288 mm is 14.5 cm. This predicted lateral movement is consistent with the maximum

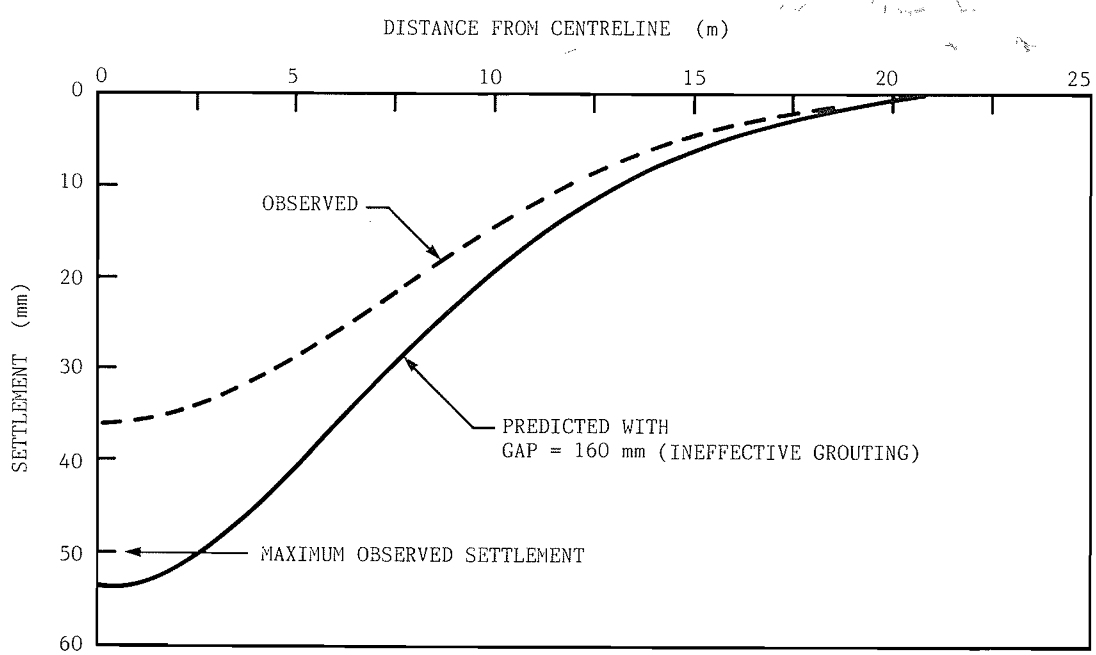

FIG. 8. Comparison of observed surface-settlement trough with predicted results (GAP = 160 mm), BART Project. (Observed response based on Kuesel 1972.)

value of 15.8 cm measured using inclinometers in the field.

The maximum surface settlement predicted by the proposed empirical procedure (δ_{max} = GAP/3) is given in Table 1. Reasonable agreement between the predicted and field-observed values is also obtained.

Lower Market Street, Bay Area Rapid Transit Subway System

The first tunnel at Lower Market Street that forms part of the Bay Area Rapid Transit (BART) system in San Francisco, reported by Kuesel (1972), will be examined in this section.

Soil conditions and tunnel geometry

The Lower Market Street tunnel was constructed by a closed-face tunnelling shield of 5.65 m in diameter. The tunnel axis is located 19 m below the surface (H/D = 3.4). The section of the tunnel under consideration was excavated in San Francisco Bay mud, a recent soft marine silty clay with decaying vegetation and shells. Apart from being slightly more compressible, this material has similar engineering characteristics to the silty clay that was tunnelled in Thunder Bay.

The San Francisco Bay mud has been defined as a soft, plastic, normally consolidated clay; however, an undrained strength in excess of 50 kPa at the tunnel axis has been reported (Kuesel 1972). Consolidated isotropically undrained (CIU) triaxial tests on recent San Francisco Bay mud obtained by Duncan and Dunlop (1969) indicated that the elastic modulus for this relatively soft, plastic clay lies within the range of 1–20 MPa over a depth of 33 m, with a modulus value of about 10 MPa at the tunnel axis level. This range of modulus values was confirmed by Clough and Denby (1980) by the use of self-boring pressuremeter tests. The 33-m clay deposit is underlain by a layer of dense fine sand and approximately 30 m of firm silt and clay. Bedrock is found at a depth of 70 m. The soil profile adopted in the present analysis is the same as that adopted by Rowe and Kack (1983).

Method of construction

Very conservative construction techniques, designed to largely reduce ground loss, were adopted in this project. A detailed account of the method of construction and the field observations at Lower Market Street have been reported by Kuesel (1969, 1972), and only a brief summary will be given here.

The Lower Market Street tunnel was constructed using a 5.65-m mechanically operated closed-face shield. The machine had narrow slots or doors through which the soil was brought in as it was scraped from the tunnel face; care was also taken to ensure that the machine advance was kept equal to the volume of excavated soil. These measures permitted the face to be firmly supported. In addition, compressed air at 83 kPa was being used which reduced the value of stability number N from about 6 to 4.6. The tunnel was lined with a bolted, segmental lining with an outside diameter of 5.49 m. Advancing the shield left an annular void (i.e., physical gap) of approximately 160 mm, and special attention was given to filling the tail void as promptly and fully as possible. Two optional methods for filling the tail void were specified: either ring-by-ring grouting or a two-shot process by injecting pea gravel immediately behind the shield, with grouting following within 45 m behind the heading. The grouting details were left to the contractor's discretion, but both methods were used successfully.

Settlement observations

The settlement owing to tunnelling was reported to be generally less than 50 mm and often less than 25 mm. The generally small magnitude of settlement was attributed to the good tunnelling technique employed. The surface-settlement trough observed at the instrumentation section is shown in Fig. 8. The trough was observed to have a maximum centreline settlement of 36 mm and spread to a lateral distance of 18 m on either side of the tunnel centreline, giving an average slope of the final trough of approximately 1:500 (Kuesel 1972).

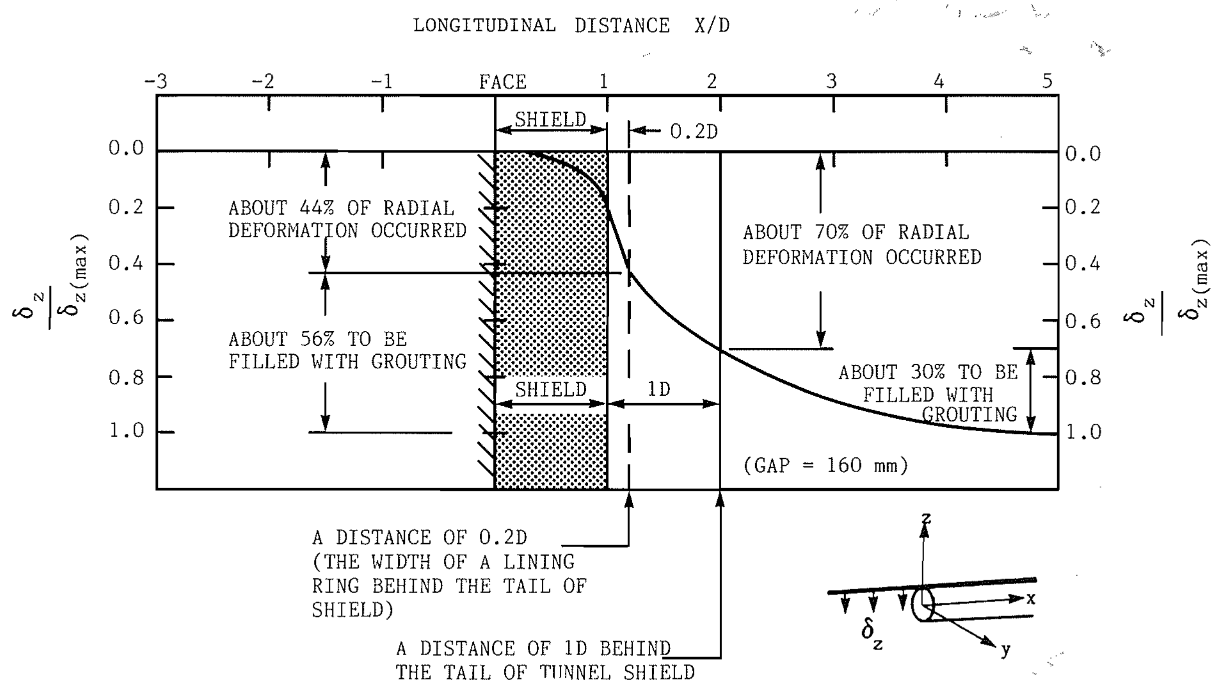

FIG. 9. Development of 3D radial displacement along the longitudinal crown axis, BART Project.

Estimation of the gap parameter and finite element analysis

The construction technique employed on this project may be expected to have minimized 3D movements ahead of the face, and hence the equivalent radial displacement u^*_{3D} can be assumed to be 0. For design purposes, a reasonable initial estimate of the gap parameter would be equal to the physical gap of 160 mm, which is the exact difference between the diameter of the excavated surface and the tunnel lining (and neglecting the effects of pea gravel and grout). The predicted settlement obtained for this gap is shown in Fig. 8. It may be seen that in this case the adoption of physical gap gives rise to an upper bound settlement prediction. The calculated centreline settlement is about 54 mm compared with about 37 mm observed at the instrumentation section and the maximum observed settlement of 50 mm.

The foregoing results suggest that the introduction of pea gravel and grout (under compressed air of 83 kPa) into the tailpiece void may have significantly decreased the annular void. Rowe and Kack (1983) simulated the effect of compressed air under 2D plane strain conditions and concluded that the compressed air was not particularly useful from a 2D standpoint.

For the purposes of the present study, a 3D finite element analysis was performed to simulate the tunnel excavation. The compressed air was simulated by decreasing the traction removed during excavation by 83 kPa. A gap parameter of 160 mm was simulated using the technique described by Lee (1989).

The normalized radial deformation profile of soil ($\delta_z/\delta_{z(\max)}$) along the crown axis of the tunnel is illustrated in Fig. 9. Since pea gravel – grouting was immediately injected behind the shield under a compressed air pressure of 83 kPa, this construction technique may allow the insertion of a reasonable quantity of grout – pea gravel into the annular void prior to movement of the soil onto the tunnel lining. Under 3D conditions, the effectiveness of the grouting process will depend on the actual amount of radial intrusion of the soil into the annular void prior to the injection of grout. If the soil were allowed to move into full contact with the lining, then the grout injection would not be effective. If there is still a void between the soil and lining, then the grout – pea gravel injection will have an effect. Thus the potential effectiveness largely relies on the 3D nature of the tunnel heading. In the BART tunnel the hydraulic rams pushed the shield forward off the newly erected lining ring, leaving a 0.2*D* length of ground unsupported behind the lining until such time as the void is filled by pea gravel or grout. This distance of 0.2*D* behind the tunnel face is plotted on the normalized curve shown in Fig. 9. It may be seen that the total 3D radial deformation developed up to this distance is approximately 44% of the final maximum deformation. In other words, the intrusion of grout into the tailpiece void would reduce the volume of the void by about 56%. This reduction of tail void can be treated as a negative contribution to the workmanship parameter ω. This implies a value of ω of -90 mm (i.e., 0.56×160 mm) and gives rise to a gap of 70 mm (i.e., 160–90 mm), which may be regarded as a reasonable lower bound estimate of the gap and assumes perfect workmanship. The predicted maximum surface settlement with a gap of 70 mm is 22 mm, which is consistent with the observation that the field settlement was often about 25 mm.

The foregoing discussion relates to ideal conditions of complete grouting in the ring behind the shield. However, if grouting is delayed until after several rings have been placed, then the effectiveness will be reduced. With the benefit of hindsight, consideration may be given to the observed surface settlement at the instrumented section. If the grout was not injected until a distance about 1*D* behind the tail of the tunnel shield, then Fig. 9 shows that the total 3D radial deformation developed up to this distance is about

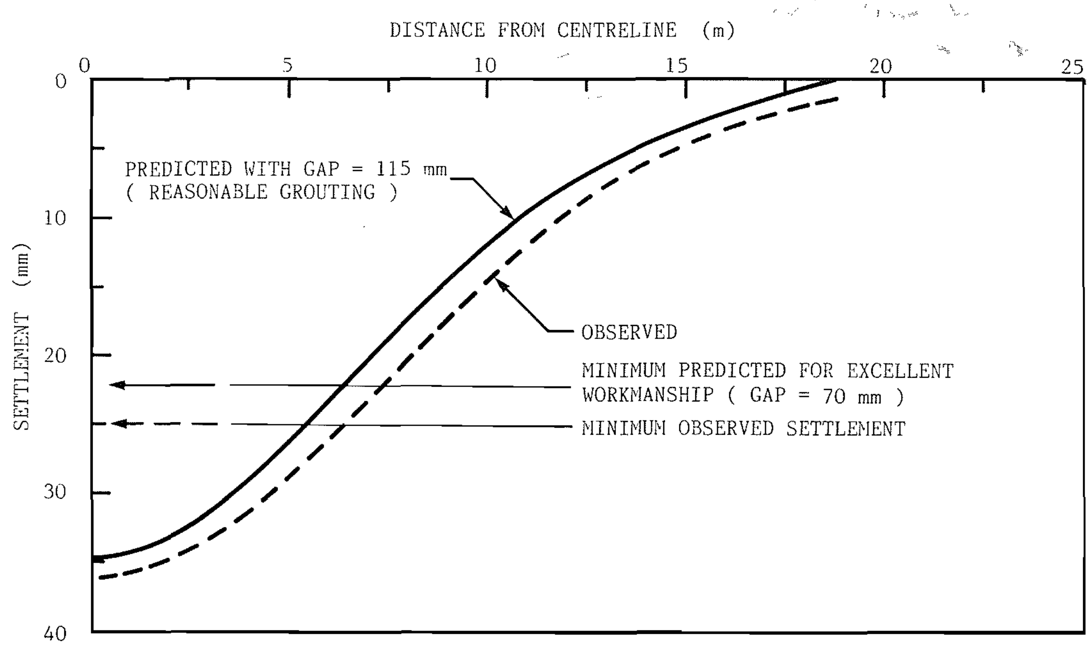

FIG. 10. Comparison of observed surface-settlement trough with predicted results (GAP = 115 mm), BART Project. (Observed response based on Kuesel 1972.)

70% of the final maximum deformation (i.e., leaving about 30% to be filled with grout). This implies a value of ω of -45 mm (i.e., 0.3×160 mm) and gives rise to a gap of 115 mm (i.e., 160–45 mm). The predicted settlement profile with a gap of 115 mm is shown in Fig. 10,. which is consistent with the observed behaviour at the primary monitoring section.

On the basis of the foregoing, it is considered that the construction techniques adopted in the BART project successfully allowed the insertion of a reasonable quantity of pea gravel or grout into the annular void prior to movement of the soil onto the tunnel lining. This effect may be approximately simulated in a plane strain analysis by adopting a smaller gap (i.e., ω may be negative). The amount of reduction of ω can be approximately obtained by 3D finite element analysis or the simplified 3D approach described by Rowe and Lee (1992).

The estimated gap parameters for the BART subway tunnels are in the range of 70–160 mm. By adopting the empirical relationship of $\delta_{max} = GAP/3$, the maximum surface settlements are predicted to be in the range of 23–53 mm. These values agree reasonably with the observed values as shown in Table 1. The observed volume of surface-settlement trough is about 2.7% of the volume of tunnel excavation.

Green Park Underground

The Green Park Underground tunnel was shield driven in stiff, heavily overconsolidated London clay. The shield was 3.5 m long and 4.15 m in diameter, and the depth to tunnel axis level was 29.3 m ($H/D = 7.1$). A detailed account of the method of construction and the field observations has been reported by Attewell and Farmer (1974).

Soil conditions and method of construction

The tunnel was mined by hand excavation. The upper part of the face was excavated 0.6 m ahead of the shield and was "boxed-in." The shield was then jacked forward and the lower part subsequently excavated. The lining was a seven-segmented, bolted, cast-iron lining, 4.07 m external diameter and 0.6 m in width erected inside the shield tail.

Grout was injected into the space between the cut clay surface and the lining ring when there was a 1.2-m unsupported length of tunnel between the rear of the tail and the last grouted portion. Grout injection progressed from bottom upwards at low pressure.

The undrained shear strength obtained from unconsolidated undrained (UU) triaxial tests on tube samples at the axis level is 266 kPa, which is consistent with the quoted stability ratio of 2.2. The elastic modulus of the clay was not reported, but this may be assumed to be 200 times c_u. This E_u/c_u ratio was suggested by Lo *et al.* (1984) by examining the data obtained from UU tests on block samples from a shaft in London clay at Ashford Common (Ward *et al.* 1965) at approximately the same depth as the Green Park Underground tunnel. Since the clays at these locations are similar, reasonable accuracy may be expected using the extrapolated soil data from Ashford Common.

Estimation of the gap parameters and finite element analysis

The maximum physical gap is equal to the size of the bead (6.5 mm) plus the tail void (76 mm), i.e., 82.5 mm. Grouting was injected into the tail void. As reported by Attewell and Farmer (1974), the rate of soil settlement over the tunnel crown was in the order of 0.33 mm/h. The average rate of advance of the shield was 0.13 m/h. Thus, it is clear that the grouting should have been effective (i.e., the effective tail void size can be assumed as zero). However, the movement of soil into the space created by the bead would have already occurred, and so the physical gap approximately corresponds to the size of the bead (i.e., $G_p = 6.5$ mm). From the construction method described, and the effective grouting procedure adopted, it is clear that ground displacement is restricted mainly to the 3D movement at the face. The dimensionless axial displacement Ω determined from Fig. 1

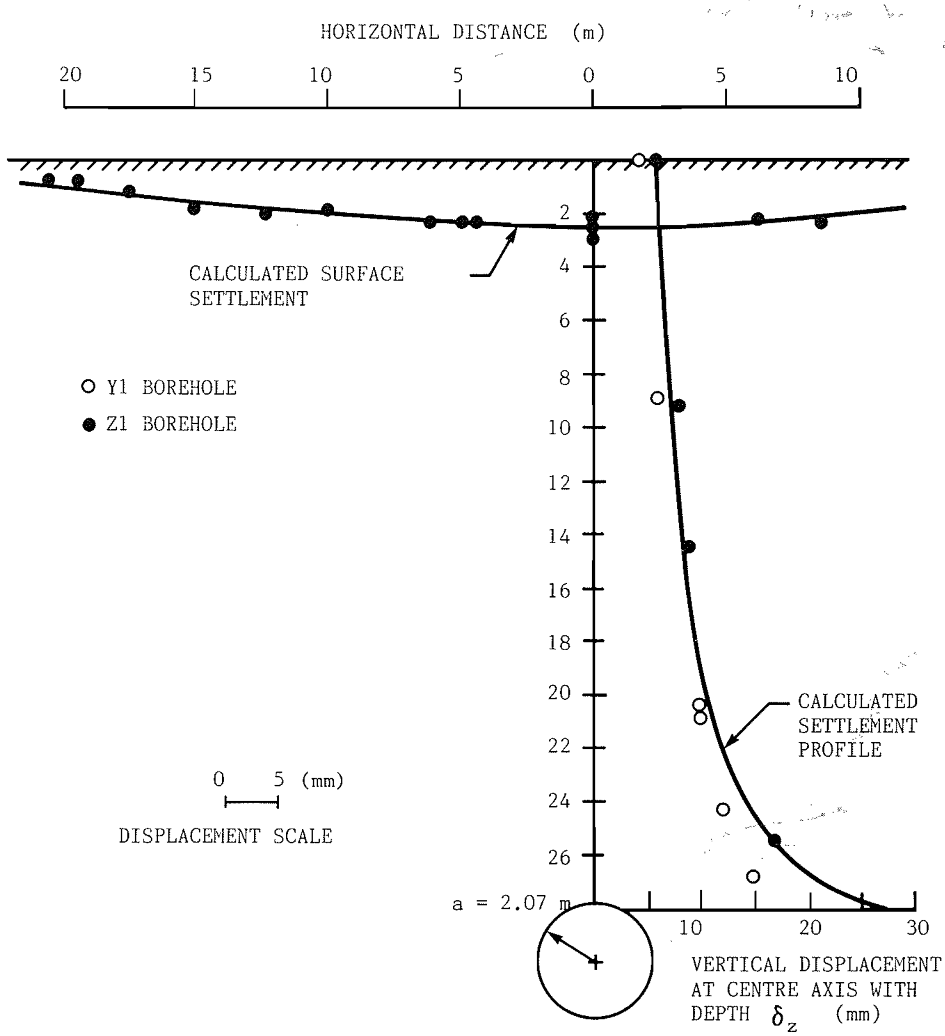

FIG. 11. Comparison of predicted and observed settlement trough and vertical displacement with depth, Green Park Underground. (Field data based on Attewell and Farmer 1974.)

is 1.12. Since at K_0' value of 1.65 at tunnel axis depth may be assumed from the work of Skempton (1961), the equivalent 3D displacement u^*_{3D} is estimated to be 17.5–20.5 mm. (The effect of $K_0' \geq 1$ is considered by [3c].) Taking G_p = 6.5 mm and u^*_{3D} = 17.5–20.5 mm, the total gap parameter is then found to be about 24–27 mm.

The predicted settlement trough and vertical displacement with depth are shown in Fig. 11. It may be seen that the predicted results for the gap of 27 mm are in good agreement with the observed data. A gap of 24 mm would give a better agreement for the data at borehole Y1.

The empirical data for stiff clays as shown in Fig. 1 were used to predict the maximum surface settlement. The $H/(2aN)$ ratio for the Green Park Underground is 3.2; this gives rise to a δ_{max}/δ_c ratio of 0.13. Taking δ_c = GAP = 24–27 mm, the resulting δ_{max} is predicted to be about 3–4 mm, which is a little below the observed range of values of 5–7 mm but of a similar order. The observed volume of surface-settlement trough is 1.3% of the excavated tunnel volume.

Long-term settlement was monitored by Attewell and Farmer (1974). The maximum surface settlements, however, were found to slightly decrease with time. They concluded that, because of the high K_0' value (K_0' = 1.65), postshield inward decompressions would cause slight upward movement of the tunnel crown and consequently slight decreases of the surface settlement with time. This effect cannot be simulated by the proposed theory; however, the effect of this postshield inward recompression under high K_0 conditions is small and may be considered to be of secondary importance.

Additional case records

Four case records of tunnelling in clays have been analyzed in the previous section where comparison between measured and calculated displacements have been made. The calculated overall pattern of ground displacements predicted by detailed finite element analysis is in good agreement with the field observations. Moreover, the proposed empirical procedure also seems capable of predicting the maximum surface settlement δ_{max} to the degree of accuracy of engineering interest.

To access the applicability of the proposed empirical approach, additional case histories will be examined within this section. The procedure is to calculate the gap parameter as described by Lee *et al.* (1992) and then predict δ_{max} by the proposed empirical relationship. Nine additional case

TABLE 2. Pertinent data for additional case histories

Tunnel	Soil type	Reference	a^a (m)	H (m)	H/D	c_u (kPa)	E_u/c_u	N
1. Mississauga sewer	Dense glacial till	Delory *et al.* 1979	2.14	13.1	3.1	360	280	0.9
2. Ottawa outfall sewer	Overconsolidated Leda clay	Eden and Bozozuk 1969	1.52	18.3	6.0	90	300	3.2
3. Mexico City siphon	Soft, highly compressible clay	Tinajero and Vieitez 1971	1.48	11.7	4.0	35	115	5
4. Grangemouth sewer	Soft silty clay	Henry 1974	1.30	10.0	3.9	22	500	5.4
5. Tyneside Tunnel	Stiff laminated clay	Attewell and Farmer 1975	1.01	7.5	3.7	75	200	2
6. Buenos Aires	Soft silty clay	Moretto (1969)	2.35	16.4	3.5	35	800	8.6
7. Regents Park (northbound)	Stiff London clay	Barratt and Tyler 1976	2.07	20.1	4.9	230	140	1.7
8. Regents Park (southbound)	Stiff London clay	Barratt and Tyler 1976	2.07	34.1	8.3	280	200	2.4
9. Heathrow Cargo	Stiff London clay	MuirWood 1969	5.45	12.8	1.2	165	165	1.5

[a] $D = 2a$.

TABLE 3. Estimation of gap parameter for additional case histories

Tunnel	u^*_{3D} (mm)	G_p (mm)	ω (mm)	GAP (mm)	Surface settlement δ_{max} (mm): Predicted	Surface settlement δ_{max} (mm): Observed	Remarks
1. Mississauga sewer		$(u_{\rm l} < G_p)$ $u_i = 11$, $G_p = 100$		11	2.5[a]	0–3	Full plane strain conditions developed behind the face
2. Ottawa outfall	12	0	0	12	4[b]	6	Physical gap eliminated by jacking of lining
3. Mixico City siphon	90	130	78	298	100[b]	105	Significant steering and alignment difficulty reported
4. Grangemouth sewer	16	0	54–90	70–106	23–35[b]	25	Physical gap eliminated by grouting and compressed air of 62 kPa; G_p assumed to be 90–150 mm for estimation of ω
5. Tyneside Tunnel		$(u_i < G_p)$ $u_i = 19$, $G_p \sim 90$–150		19	9[a]	8	Full plane strain conditions developed behind the face
6. Buenos Aires	~90	90–150	54–90	234–330	90–110[b]	130–180	G_p assumed to be 90–150 mm
7. Regents Park (northbound)	14	3	0	17	6[a]	7	3-mm bead; physical gap eliminated
8. Regents Park (southbound)	14	3	0	17	3.5[a]	5.5	expanded lining
9. Heathrow Cargo	28	0	0	28	22[a]	11	G_p eliminated by expanded lining

[a] Predicted by empirical relationship as shown in Fig. 1.
[b] Predicted by empirical relationship δ_{max} = GAP/3.

histories emcompassing soft to very stiff clay and various construction techniques have been chosen for analysis. The pertinent data and references for these cases are given in Table 2. The establishment of gap parameters and the predicted and observed surface centreline settlements for these case records are then given in Table 3. It can be seen that the proposed empirical approach does provide a reasonable estimate of the range of the maximum surface settlement for all cases except the Heathrow Cargo tunnel case, where the deformations were overestimated. A closer examination of the Heathrow Cargo case indicates that the size of the tunnel opening (diameter 10.9 m) is much greater than the rest of the cases analyzed, and the tunnel was constructed at a much shallower depth (H/D = 1.2). It may follow that very large diameter shallow tunnels such as the Heathrow Cargo tunnel may present a different response and that the present analysis may lead to an overprediction of settlement.

Conclusions

A theoretically based procedure for estimating potential settlement above tunnels constructed in soft ground has been proposed in a companion paper by Lee *et al.* (1992). This theory is based on the assumption that the surface settlement is largely related to the volume of lost ground, which is represented by a gap parameter. This so-called gap parameter is the sum of the physical gap (G_p), which is related to the machine, shield, and lining geometry, the 3D deformation at the face (u^*_{3D}), and the workmanship parameter (ω). The gap parameter can then be incorporated with plane strain finite element methods (such as that developed by Rowe *et al.* 1983) to predict the resulting surface settlement. Alternatively, the gap parameter can be considered as the vertical displacement above the crown of the tunnel (δ_c). The maximum surface settlement (δ_{max}) can then be estimated from the settlement ratio δ_{max}/δ_c established through empirical correlation. The application of these theoretical and empirical techniques has been discussed and the range of applicability assessed by reference to 14 case histories that encompass a wide range of soil conditions and construction techniques.

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. For the four case histories examined by detailed finite element analysis, it would appear that the proposed theory does provide a reasonable estimation of the range of the gap and consequently the surface settlement.

On the other hand, the gap parameter can be used in conjunction with the empirical relationship as established by Lo *et al.* (1984) and Ng (1991). Using this approach, analyses of 14 case histories involving tunnelling in clays of widely different strengths and geometries have been performed. It is found that the calculated surface displacements agree reasonably well with measured displacements.

This paper has demonstrated that the technique proposed by Lee *et al.* (1992) does provide reasonable estimates of the possible "bounds" of gap parameters for a given shield and lining geometry and construction technique. It is suggested that the simplified procedure developed for predicting the gap and therefore the surface settlement may be used for preliminary design purposes provided that the undrained shear strength (c_u) and modulus of the clay (E_u) can be reasonably determined. This gap can also be used in conjunction with more sophisticated numerical methods to predict the variation in settlement with position and depth at critical sections of the tunnelling project. This paper has focussed on predicting short-term (undrained) deformations. A similar approach may be adopted for predicting long-term deformations but, in addition to the factors considered here, also requires consideration of the size of the remoulded zone surrounding the tunnel and the consolidation and creep characteristics of the soil (e.g., see Yi *et al.* 1992), which are beyond the scope of the present paper.

Acknowledgements

Funding for this research has been provided by the Natural Sciences and Engineering Research Council of Canada by means of grant A1007 and a Steacie Fellowship awarded to R.K. Rowe and grant 0046681 to K.M. Lee.

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