한국지반공학회논문집 제18권 1호 2002년 2월 pp. 17 ~ 28

An Investigation of Tunnel Behaviour Using a Time-based 2-D Modelling Method

시간-파라미터법에 의한 터널거동 특성 연구

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요 지

터널의 막장부근에서는 하중전이가 복잡한 3차원적 거동특성을 나타낸다. 일반적으로 3차원 해석은 입력자료의 준비 및 결과 해석에 많은 노력을 요하며, 비선형 압밀해석을 수행하는 경우 컴퓨터 계산수요가 커지는 문제 때문에 실제설계는 2차원 해석법을 주로 사용하게 된다. 3차원적인 터널거동을 2차원적으로 모델링하기 위하여는 터널 굴진과정을 수치해석적으로 표현하기 위한 경험 파라미터가 요구되며, 해석결과는 이 파라미터 값에 따라 크게 변화하는 특성을 나타낸다. 특히 이 값의 평가는 주로 주관적이고 경험적인 판단에 의존하게 되므로 임의성이 커 해석의 신뢰성 문제를 야기할 수 있다. 특히 지반거동이 간극수 등 시간의존성 요인에 의해 영향을 받는 경우 이 파라미터를 평가하기는 더욱 어려워진다. 본 논문에서는 이러한 문제점을 개선하기 위하여 제안된 Time-based 2-D Modeling Method를 사용하여 지하수위가 높은 풍화화강토내 터널굴착 문제에 적용하였고 이를 분할 굴착에도 적용 가능하도록 확장하였다. 해석결과는 실측결과와 잘 일치하는 거동을 보였다. 이 방법을 이용하여 복합막장터널의 거동특성을 조사한 결과, 마제형 터널단면의 역학적 유익성이 확인되었다.

Abstract

Tunnel construction is a complex three dimensional operation. Since, however, it is neither possible nor useful to simulate all conditions and parameters in detail, a simplified two dimensional model is commonly employed in practice. The simulation of three dimensional conditions by a two dimensional model should use empirical parameters. The numerical predictions indicate that analysis results are highly dependent on the parameters. An improved modelling method based on time was adopted to account for three dimensional effect at the tunnel heading and time dependent nature, and used to perform an analysis of tunnelling in decomposed granite. The effects of weathering degree, tunnel shape and multi-drift excavation were investigated by using the method. It is identified that a structural benefit can be obtained by adopting a horse-shoe-shaped cross section with multi-drift excavation in mixed-face ground condition.

Keywords : FEM, Mixed-face tunnel, Time-based 2-D modeling

1. Introduction

Numerical methods have become increasingly popular due to the rapid advancements in computer technology and its availability to engineers. The methods have been applied to the geotechnical problems and gradually successful in both qualitative studies and quantitative predictions. Among several numerical methods, the Finite Element Method(FEM) is the most widely used method. It has been improved, and increasingly replaced

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conventional solutions in tunnel engineering. However, to make use of the full potential of the method considerable expertise is required.

To obtain realistic results from the finite element analysis the actual construction procedures and their sequence must be taken into account. Tunnel construction is a complex three dimensional operation. Consequently any numerical modelling should, in principle, attempt to reproduce this behaviour. Since it is neither possible nor useful to simulate all conditions and parameters in details, a simplified two dimensional model is commonly employed in practice.

Two dimensional modelling of tunnelling is of importance to obtain an insight into necessary design variations, assess the sensitivity of the construction method, study the influence of varying soil conditions or find the proper location of measuring instruments and so on. In this paper existing 2-D modelling methods are discussed. A time-based 2-D modelling approach(Shin & Potts, 2001b) is used to model three dimensionality and time dependent effect at the tunnel heading.

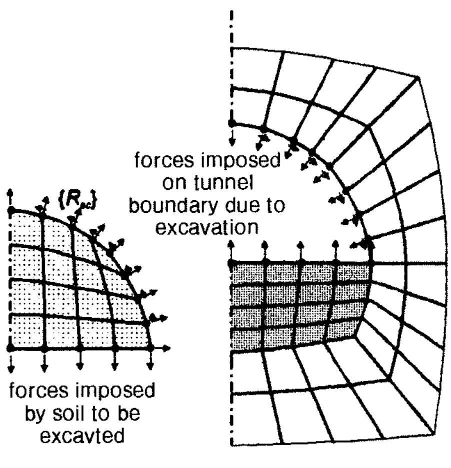
2. An Improved 2-D Modeling Approach

2.1 Geotechnical Consideration of Tunnel Modeling

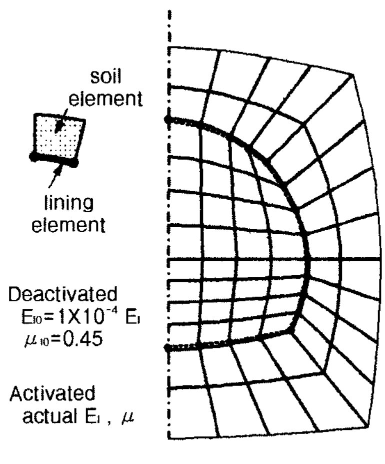
Numerical modelling of tunnelling requires some geotechnical consideration accounting for the modelling of construction sequence, ground condition, and material properties. The main construction process is excavation and construction of lining.

Excavation of a tunnel is generally modelled by physically removing elements from the area to be excavated and imposing the corresponding forces on the newly created boundary surface, see Fig. 1(a). The simulation of the excavation should involve the effect of body force and other surface traction, if any, in the right hand side vectors of the finite element equation. A general expression has been introduced by Ghaboussi and Pecknold (1984) and Brown and Booker(1985).

The tunnelling procedure includes installation of a lining. The simulation of such an activity in a FEM requires sophisticated book keeping arrangements for keeping track of data relating to deactivation of the elements to be constructed. Elements that are going to be constructed must be included in the original finite element mesh. They must then be deactivated until it is time to construct(or reactivate) them. In this paper the deactivated state is set such that the stiffness is equal to 10^{-4} times the minimum modulus from the prescribed linear material properties, and the Poisson's ratio is 0.45 as illustrated in Fig. 1(b). The construction itself can be simulated by reactivating the elements to be constructed. Reactivation is achieved by assigning the actual stiffness and applying self weight body forces to the constructed.



(a) Simulation of excavation



(b) Simulation of construction

Fig. 1. Simulation of tunnelling

2.2 Review of 2-D Modelling Methods

Excavation and construction described above is a sequential process and causes three dimensional behaviour at the tunnel heading. Simulation of this procedure in two dimensions requires certain simplifications. Numerical experience is vital to achieve appropriate simplification, and for the understanding of the implications due to the simplified modelling. Two dimensional modelling of the three dimensional effect can be achieved by employing simplification using geometrical symmetries, and geotechnical simplification of the ground and construction sequences. Four plane strain approaches adopting empirical parameters, are briefly reviewed and discussed.

- **Percentage Unloading Method**

This approach is also known as the convergence-confinement method(Panet & Guenot, 1982). It is assumed that three dimensional effects are accounted for by replacing the ground to be excavated by a fictitious stress vector around the tunnel periphery:

$$\{\sigma_f\} = (1-\lambda)\{\sigma_o\} \qquad (1)$$

where $\{\sigma_f\}$ is the fictitious stress vector at the excavation boundary, $\{\sigma_o\}$ is the initial total ground stress vector prior to tunnel excavation, and λ is the stress reduction factor which depends on the progression of the tunnel face. Tunnel progression is simulated by introducing the parameter λ which varies from 0.0 at the initial undisturbed state, to λ_D when the lining is installed as shown in Fig. 2. The λ_D is the unloading percentage or stress reduction factor and the determination of λ_D is critical in this approach.

- **Stiffness Reduction Method**

In this approach, it is assumed that the three dimensional stress distribution at the tunnel heading relates to the ground ahead and around the excavated tunnel and that it can be simulated by a stiffness reduction in the soil to be excavated from within the tunnel(Swoboda, 1979). The modelling procedure consists

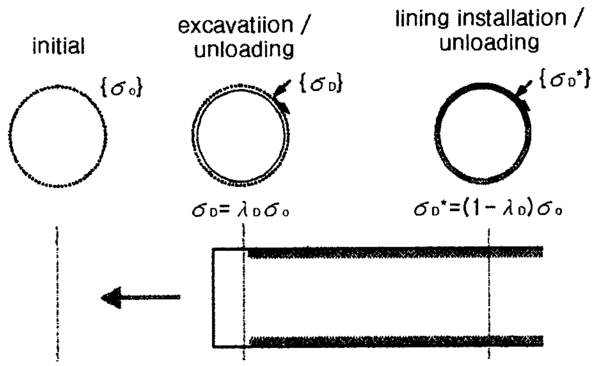


Fig. 2. Percentage unloading method(Panet & Guenot, 1982)

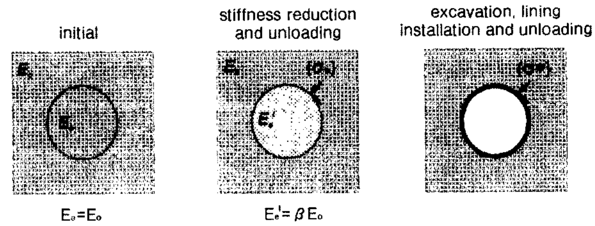


Fig. 3. Stiffness reduction method(Swoboda, 1979)

of two steps as shown in Fig. 3. In the first step, fictitious nodal stresses at the excavation boundary, $\{\sigma_o\}$, are calculated from the initial stress condition. Then these fictitious stresses are applied to the boundary of the tunnel in which the stiffness of the soil is reduced, so that:

$$E_e = \beta E_o \qquad (2)$$

where E_e is the reduced Young's modulus prior to lining installation, E_o is the initial Young's modulus and β is the stiffness reduction factor. In the second step excavation is performed and the lining is installed. After that the resulting stress, $\{\sigma^*\}$, from the first step is applied to the lining. The parameter β controls the amount of stress release produced before lining installation.

- **Volume Loss Method**

It is common in clay soil to assume that the volume lost into the tunnel, V_l, is the same as the volume of the surface settlement trough caused by tunnelling (Addenbrook, 1996), V_s, and also that any additional volume loss after lining installation is negligible(Fig. 4).

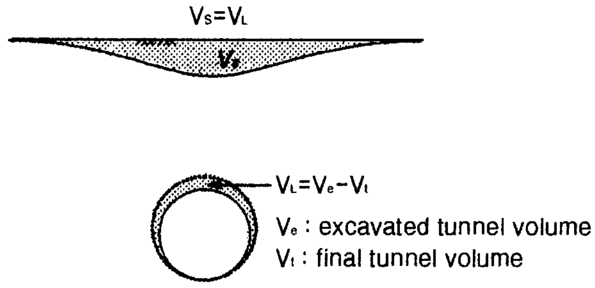


Fig. 4. Volume loss method (Addenbrooke, 1996)

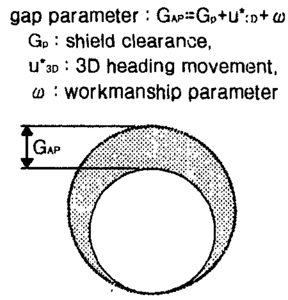


Fig. 5. Gap parameter method (Rowe et al, 1983b)

$$V_l = V_s \tag{3}$$

It is necessary to monitor movements during the excavation procedure and to construct the tunnel lining on the increment at which the required volume loss(V_l) is achieved. After lining installation the loading boundary condition which models excavation is still applied to the excavation boundary up to completion of full release, thus introducing the initial lining stresses.

• Gap Parameter Method

In this method, the three dimensional excavation effect is approximately incorporated in a plane strain analysis in terms of a void that represents the volume lost into the tunnel. The magnitude of this void is expressed by a gap parameter, G_{AP}, which is the vertical distance between the crown of the tunnel and the original position of the soil(Fig. 5). It is assumed that the lining rests on the soil directly beneath the invert of the tunnel and that the periphery of the excavation and the tunnel are circular. The deformation of the boundary is monitored during unloading, and once the deformed boundary comes into contact with the tunnel lining, the interaction between soil and the lining is analysed using soil-structure interaction theory(Rowe *et al.*, 1983a and 1983b).

• Discussions

How good these approaches are at modelling reality can only be judged by the evaluation of empirical parameter β, V_l, and G_{AP}. The previous numerical analyses of tunnels in decomposed granite in the reference indicate that surface settlements are highly dependent on the empirical parameters as shown in Fig. 6(Shin & Potts, 1998). The figure shows the significant variation of the surface settlement trough as the stress control factor(λ) changes.

The two dimensional modelling methods explained above summarized in Table 1. These methods can be broadly categorized in terms of factors controlling the three dimensional effect as: stress control method; stiffness control method; displacement control methods. As many authors have indicated, the volume(or gap) of the potential ground loss depends on soil type. In clay, the volume of the surface settlement trough may be equal to the volume of the ground loss, provided that no drainage takes place. Meanwhile in sands volume expansion or contraction may be caused during excavation. Thus the displacement control method is appropriate to tunnelling in clay with a fast rate of advance. The percentage unloading method has some advantages over the stiffness reduction method, because the assumptions made are much clearer from the computational point of view.

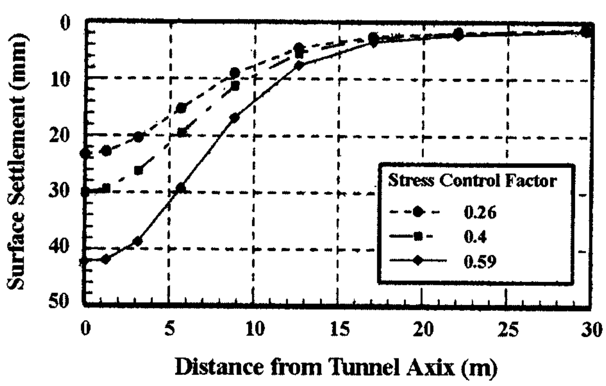


Fig. 6. Influence of stress control factor, λ(Shin & Potts, 1998)

Table 1. Two dimensional modelling methods

Modelling method	Control factor of 3D effect	Description of lining installation	Preferred application conditions	References
Percentage unloading method	stress	$\sigma_D = \lambda_D \sigma_o$ $0.3 < \lambda_D < 0.6$	–	Panet & Guanot (1982)
Stiffness reduction method	stiffness (e.g. Young s modulus)	$E_e = \beta E_o$ $0.2 < \beta < 0.5$	–	Swoboda (1979)
Volume loss method	deformation (volume loss)	$V_l = V_s$	shield tunnel with negligible volume change during construction	Addenbrooke (1996)
Gap parameter method	deformation (gap)	$G_{AP} = G_p + \omega + u^*_{3D}$		Rowe *et al.* (1983a,b)
Time-based method	time	$t^* = \alpha T$		Shin & Potts(2001b)

2.3 An Improved 2-D Modeling Method

Generally, tunnelling in water bearing soils causes time dependent ground behaviour during construction. This type of behaviour can also be found in shallow tunnelling in clays, or tunnelling in permeable soils with thin clay layers. When time dependent behaviour is involved, application of these simplified 2-D modelling methods becomes more complicated, as the empirical parameters are not constant anymore. The involvement of time dependent effects during construction make it difficult to estimate the empirical parameters from field data and may require some modification or additional assumptions.

To account for three dimensional effect and time dependent behaviour, Shin and Potts(2001) proposed a time-based plane strain approach. In this method the whole tunnelling process is represented in terms of time. For instance, the unloading due to excavation was expressed using cumulative Gaussian curve(or error function) as:

$$\{\sigma(t_n)\} = \frac{n\{\sigma_o\}}{2N}\left\{1 + \frac{2}{\sqrt{\pi}}\int_0^{t_n} e^{-x^2}dx\right\},$$
$$n = 1,2,3,\ldots..N \ and \ -\frac{T}{2} < t_n < \frac{T}{2} \qquad (4)$$

where N is the total number of increments, n is the n^{th} current increment and t_n is the time required to reach the n^{th} increment.

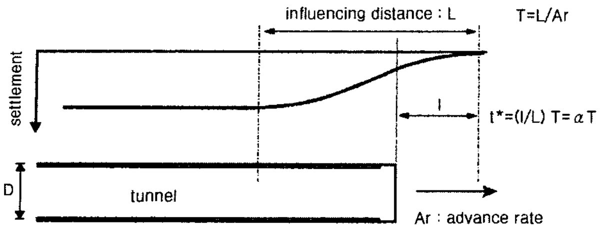
Lining installation can also be considered in the context of time. The time to introduce the lining can be assumed as:

$$t^* = \alpha T \qquad (5)$$

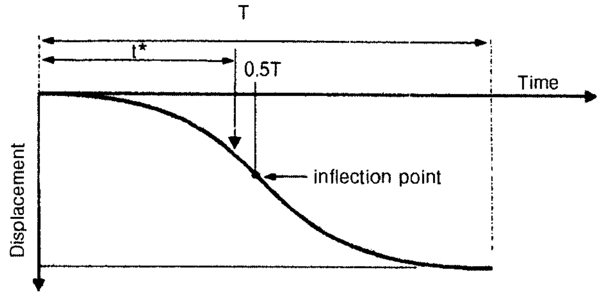
Then α can be defined as the time-based confinement factor. The lining is introduced at the increment corresponding to t^*. Two parameters, T and t^*, must be defined for the analysis. T is the time required for the stress changes induced by excavation to reach the final stress condition at the tunnel boundary. In other words it is the time period over which the three dimensional effect is taking place. The parameter t^* represents the time before lining installation and can be defined as the actual time when the lining installation takes place. T can be determined by using the tunnel advance rate which can be obtained from the construction plan, as :

$$T = \frac{\eta D}{A_r} \qquad (6)$$

where A_r is the advance rate, D is the tunnel diameter and η is the empirical factor. The field data and numerical analyses in the literature indicate that the influencing range is$(1\sim2)$ Z_0 in which Z_0 is the tunnel depth(Attewell *et al.*, 1985). In the present case, Z_0=2.2 D and consequently a η of 3.5 has been adopted for use in this paper. Parameter t^* is the empirical value to be evaluated by field data or other rigorous analyses such as three dimensional modelling method. Evaluation of t^* can be made from the longitudinal ground surface settlement. The conservative estimation of settlement just



(a) Evaluation of parameters from a longitudinal settlement trough



(b) Evaluation of parameters from a time-settlement curve

Fig. 7. Evaluation of time parameters

above the tunnel face has been recommended(Lake *et al.*, 1992) as $0.5S_{max}$ in undrained conditions. Accordingly t^* =0.5 T can be assumed. However, as Nyren(1998) pointed out, the assumption is too conservative, and causes too large displacements in the time-based approach. This indicates $t^*<0.5\ T$, i.e $\alpha<0.5$. Fig. 7 show the evaluation scheme for parameters T and t^* from longitudinal settlement troughs and time-displacement curves respectively from previous experience under similar conditions.

3. A Typical Analysis

3.1 Tunnel Profile and Finite Element Modeling

A NATM tunnel in decomposed granite soil is considered. The horse-shoe-shaped tunnel of measuring section E of the Seoul Subway Line 3(Shin & Yoo, 1986) was taken as a model for a typical analysis. This shape

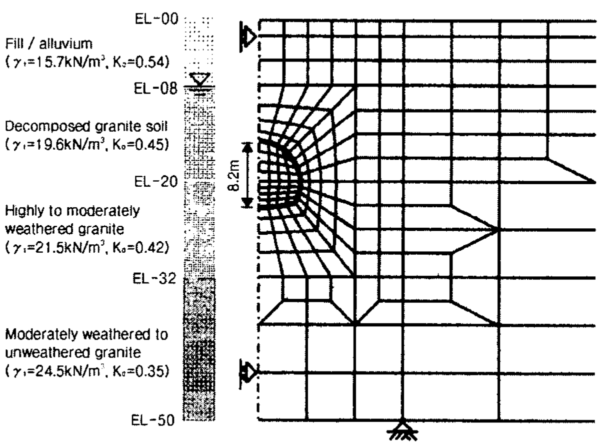


Fig. 8. Soil profile and finite element mesh

of tunnel cross section was commonly employed on the project. The tunnel was constructed beneath a 35m wide road and green field conditions were assumed for the analysis. The ground water table is located at 8m below the ground surface. The ground profile and the finite element mesh used for typical analysis of a single tunnel are shown in Fig. 8. One distinctive feature in this tunnelling practice is the short-term time dependent behaviour due to the ground water movements and the sprayed lining. This is the case where the time-based modelling method is required. Biot's(1941) three dimensional coupled equations are used for the analyses presented in this paper. Fig. 8 also shows the finite elements used. The symmetrical geometry of a single tunnel in a green field site was assumed and therefore only half of the problem was analysed.

A constant water level was assumed throughout the analysis. Flow of water into the tunnel depends on the hydraulic boundary conditions on the excavation boundary. As the soil is permeable, a drained condition(i.e. zero pore water pressure along the excavation boundary) was assumed.

The top fill/alluvium stratum was modelled as isotropic linear elastic. Small strain nonlinear elasticity model (Jardine, 1885) combined with the L,P&P model(Lagioia *et al*, 1996; Shin & Potts, 2001a) was employed to represent the decomposed granite soil and cross isotropic linear elasticity combined with Mohr-Coulomb model was used for the other materials. Material parameters are shown in Table 2.

The nonlinear log law permeability model proposed by

Table 2. Material parameters

Decomposed granite soil(EL−08～EL−32)

(a) Small strain nonlinear elastic model

shear modulus	A	B	C(%)	α	γ	$E_{d\ min}$(%)	$E_{d\ max}$(%)	G_{min} (kPa)
	1,515	1,485	$2\text{x}10^{-4}$	0.955	0.818	$9.0\text{x}10^{-3}$	0.35	9,706
bulk modulus	**R**	**S**	**T(%)**	δ	λ	$\varepsilon_{v\ min}$(%)	$\varepsilon_{v\ max}$(%)	K_{min} (kPa)
	475	465	$2\text{x}10^{-4}$	0.848	0.872	$5.0\text{x}10^{-3}$	0.50	6,438

(b) L,P&P model

plastic potential parameters	yield surface parameters	hardening parameters
α_g : 0.1 μ_g : 0.9999 M_g : 1.5	α_f : 0.0001 μ_f : 2.3 M_f : 1.16	B_p : 0.02, ξ : 0.01, p_m: 2000 p_t : 1000, p_{so}=exp(5+0.07z) p_{mo}=30+0.3z , p_{to}=10+0.1z

Other soils

(a) Isotropic linear elastic model

	Young's modulus (kPa)	Poisson's ratio	coefficient of lateral earth pressur	unit weight (kN/m^3)
fill/alluvium	E = $1.47\text{x}10^4$	0.35	0.54	15.7
moderately to unweathered	E = $1.00 \text{ x } 10^6 + 1.06\text{x}10^6 z$	0.28	0.35	24.5

(b) Mohr−Coulomb model

	cohesion intercept	angle of shearing resistance	angle of dilatancy
fill/alluvium	(linear elastic)	−	−
moderately to unweathered	100 +500z (kPa)	56.0	28.0

Vaughan(1989) is used to represent the flow behaviour in decomposed granite soil.

$$\log_e k = -Bp' \log_e k_o \quad or \quad k = k_o e^{-Bp'} \tag{7}$$

where k_o is the coefficient at zero mean effective stress, p' was the mean effective stress, and B is the material properties (m^2/kN). Model parameters are evaluated from the assumption that the in-situ depth versus permeability relationship is analogous to the in-situ mean effective stress versus permeability relationship. B=0.0043m^2/kN and k_o=1.9x10^{-6}m/s are used for the analyses presented in this paper. A NATM lining is represented by the continuous elastic beam model. The lining profile was described in Fig. 9.

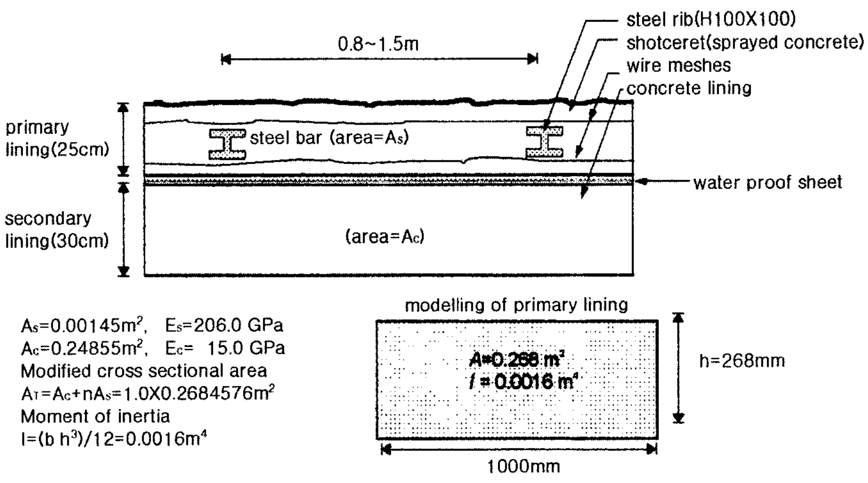


Fig. 9. Modeling of lining

The excavation sequence was modelled with the time-based two dimensional modelling method. Model parameter T=15days was used which implies 2.0 m/day of advance rate, α=0.459 was used, i.e. t^*=0.459 T. Full face excavation was assumed.

3.2 Results of Analysis

All the analyses presented in this section were coupled analyses, with the tunnel lining simulated as a permeable boundary. Consequently the results presented in this paper correspond to a long term condition.

The maximum settlement and volume loss obtained were 27.2mm and 1.02% respectively. The inflection point of the settlement trough is located 11m away from the tunnel centre line. The calculated settlements can be represented by a Gaussian normal distribution curve. To obtain the best fit Gaussian curve, three combinations of parameters were considered, namely S_{max}- i, $V_l - i$ and S_{max}- V_l. Consequently three Gaussian curves were obtained, using the approximate equation V_s=2.5 i S_{max}. These are shown in Fig. 10. Although the three alternative combinations of fitting parameters do not give significant difference in predictions, the commonly used S_{max}- i curve produce the best likely fit to both the calculated and field data.

The results are also plotted as 2 i/D against Z_o/D in Fig. 11, where Z_o is the depth to tunnel axis level, i is the distance to the inflection point from the tunnel centre line, D is the tunnel diameter and compared with field

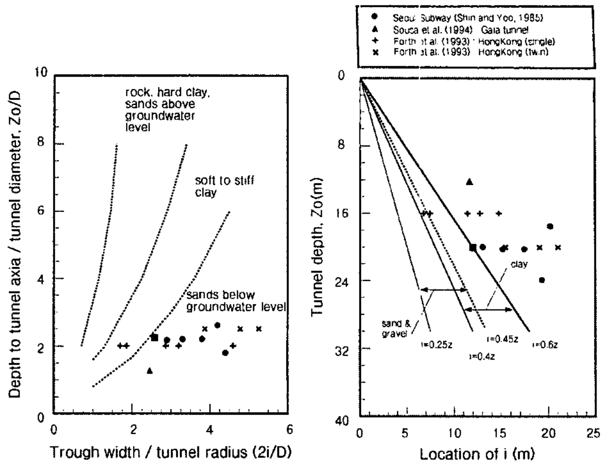


Fig. 11. Comparison of width of settlement troughs

data. Wide settlement troughs were shown.

The resulting hoop thrusts and bending moments are presented in Fig. 12. The maximum hoop thrust occurred around the spring line and was 880kPa, which is equivalent to 49% of the full overburden load at the tunnel axis level. A significant change occurred at the corner with a peak bending moment of 200kNm/m being predicted at this location. Bending moments above the spring line were quite small. This indicates that the stress distribution in the horse-shoe-shaped tunnel lining is not uniform in this situation.

4. Behavior of Mixed-face Tunnels

4.1 Effect of Weathering Profile

Lee(1987) indicated two distinctive weathering profiles of Seoul Granite; shallow weathering and deep gradual

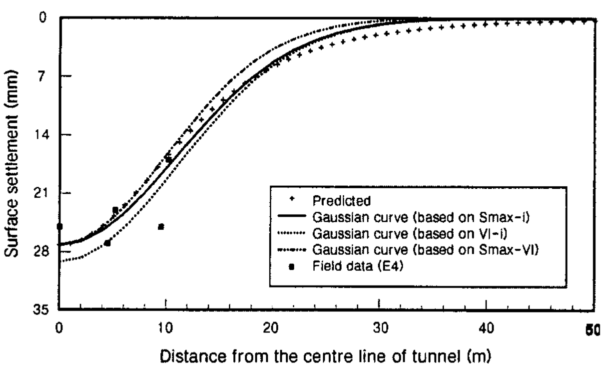


Fig. 10. Representation of surface settlement troughs

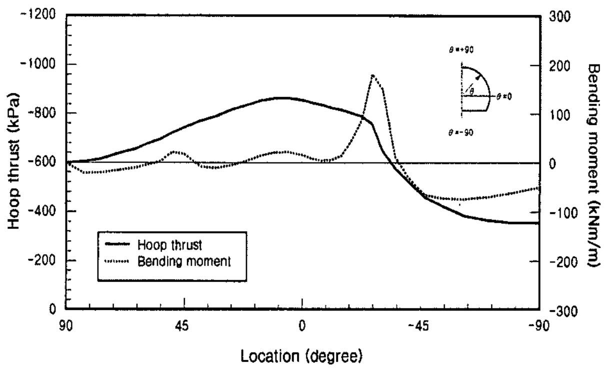
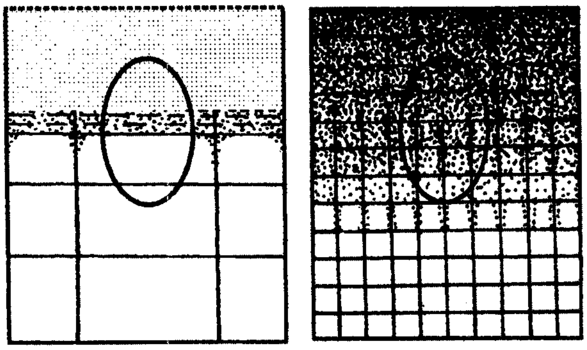


Fig. 12. Comparison of hoop thrusts and bending moments



(a) Shallow weathering (b) Deep weathering

Fig. 13. Weathering of decomposed granite and mixed-face tunnels

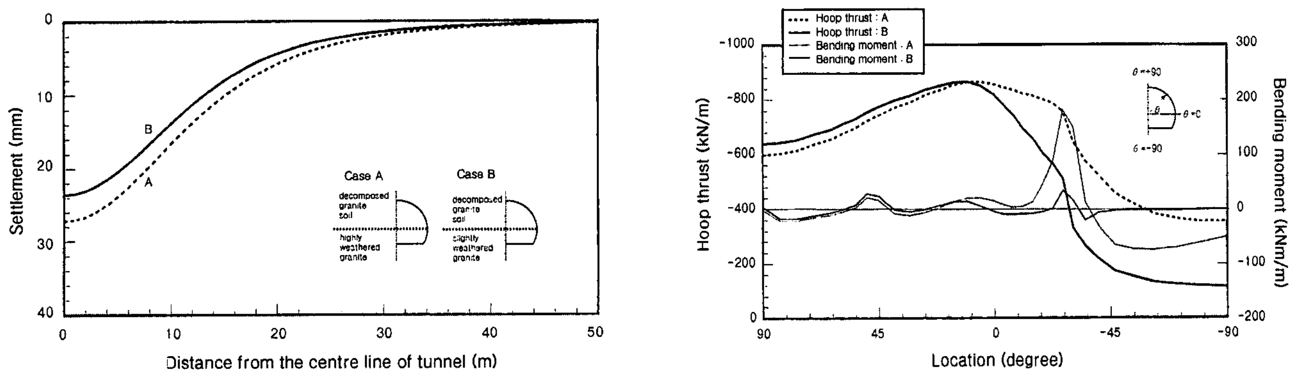
weathering as illustrated in Fig. 13. The effect of differences in weathering on tunnel behaviour is examined by considering the following two cases of mixed-face conditions: (a)case A - mixed-face of decomposed granite soil and moderately to slightly weathered granite(soil-weathered rock); (b)case B - mixed-face of decomposed granite soil and highly weathered granite(soil-hard soil). Face condition of case B is represented using continuous soil properties, meanwhile to represent case A residual soil is followed by moderately weathered granite whose material properties are shown in Table 2.

Fig. 14(a) compares the results of surface settlement troughs. Although the effect is not significant, the increase of stiffness in the soil-weathered rock face condition reduces the maximum settlement by 12%. Fig. 14(b) compares lining responses. The hoop thrusts in the lining above the spring line hardly change, but below the spring line decrease considerably. In addition, an appreciable decrease in bending moments has occurred in the soil-weathered rock face condition. It is clear that a structural benefit can be obtained by adopting a horse-shoe-shaped cross section for the soil-weathered rock face condition.

4.2 Effect of Tunnel Shapes in a Mixed-face Ground Condition

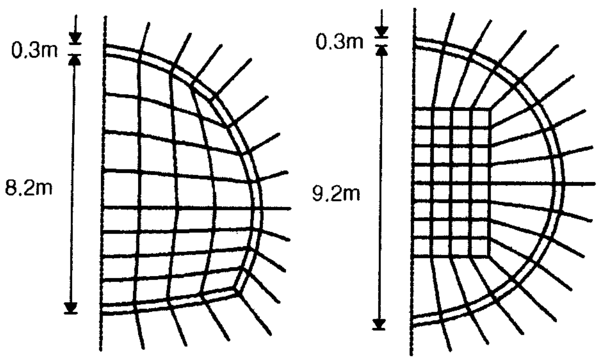
A horse-shoe-shaped tunnel section has some advantages in tunnel construction, e.g. reduces excavation volume required and provides better working conditions in comparison with a circular cross section tunnel. There were ground conditions in which the circular shape was more suitable in part, for implementation. However, in the construction of the Seoul Subway Lines 3&4, the majority of the ground conditions warranted the use of a horse-shoe-shaped tunnel and therefore for practical reasons circular(or elliptical) tunnels were not constructed very much.

An investigation of the effect of tunnel shapes was made. An equivalent circular tunnel, having the same cross sectional area as that of the horse-shoe-shaped tunnel is considered. This consideration does not fulfill the design equivalence. In the design of an underground railway system, a circular section requires a little larger area than a non-circular tunnel to provide the same operational and architectural clearance required. Thus the practical equivalence can be obtained by taking a larger tunnel volume than that of horse-shoe shaped one. Tunnel cross sections modelled are shown in Fig. 15. All



(a) Surface settlement troughs (b) Hoop thrusts and bending moments

Fig. 14. Effect of mixed-face ground condition



(a) Horse-shoe shape (b) Circular shape

Fig. 15. Modelling of tunnel cross sections

modelling methods for a circular tunnel were the same as those of the horse-shoe-shaped tunnel.

Fig. 16 shows the surface settlement trough for the two tunnels. It is evident that the trend of settlements is the same in both cases. The maximum settlement of a circular tunnel is 23.6mm and is about 87% of that for the horse-shoe-shaped tunnel. The volume loss is 93% of that for the horse-shoe-shaped section. The difference in the location of the maximum slope, indicating zero horizontal strain, is not significant.

An interesting difference in ground loading was found and shown in Fig. 17. The ground loading on the circular tunnel lining is quite uniform, varying slightly from 100kPa to 200kPa, whilst as discussed previously, the horse-shoe-shaped tunnel shows a peak value of 430kPa at the corner of the tunnel, i.e. θ=-30.

Resultant hoop thrusts and bending moments in the circular tunnel lining are shown and compared with those for the horse-shoe-shaped tunnel in Fig. 18(a). The hoop thrust in the circular tunnel changes smoothly without a significant reduction at the tunnel invert. Fig. 18(b) also shows the distribution of bending moment, indicating small and smooth changes. The variation of hoop thrust and bending moment along the lining in a circular tunnel are smoother and far less than in a horse-shoe-shaped tunnel.

However, as discussed previously, the assumption of the same cross sectional area as that of the horse-shoe-shaped tunnel does not satisfy structural equivalence. In other words, to establish the same structural envelop in the tunnel, a circular tunnel commonly requires a larger area in comparison to a horse-shoe-shaped tunnel. The increase of tunnel area for a circular tunnel may reduce

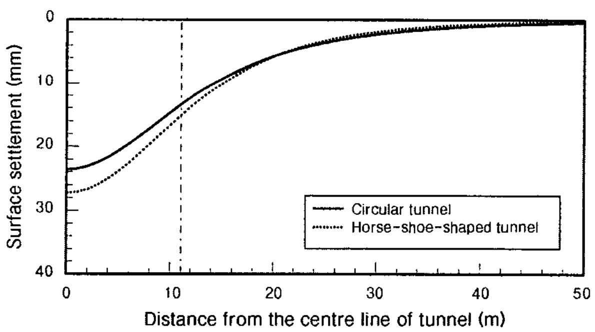


Fig. 16. Effect of tunnel shape : surface settlement troughs

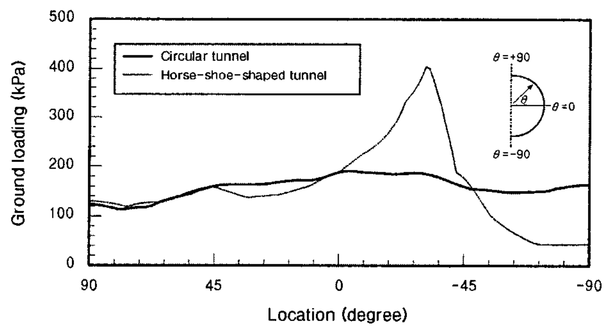
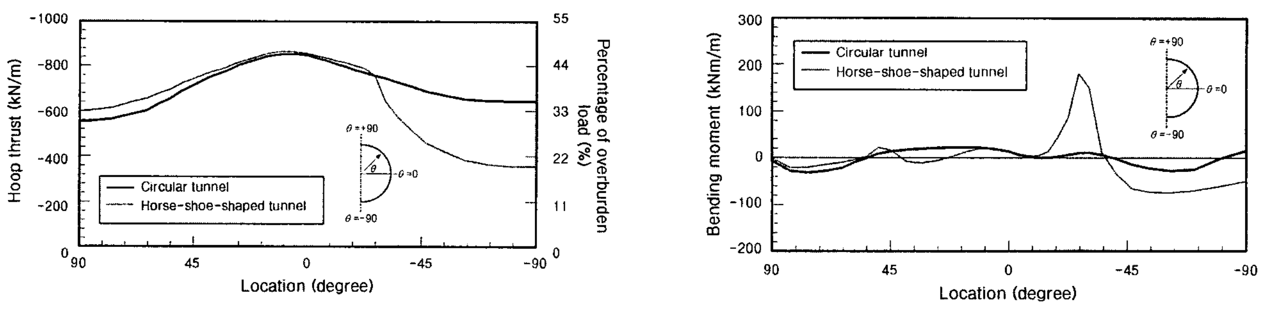


Fig. 17. Effect of tunnel shape : ground loadings



(a) Hoop thrust (b) Bending moments

Fig. 18. Effect of tunnel shape : lining behaviour

the differences in ground behaviour between the two tunnels. Consequently the effect of tunnel shape is not significant.

4.3 Effect of Multi-drift Excavation

A multi-drift excavation is frequently adopted in mixed-face NATM construction. General practice for the construction of the Seoul Subway is that firstly the upper half(tunnel area above spring line) is excavated, followed by the lower half(tunnel area below tunnel spring line), with the bench length varying from 5 to 30m. The mechanical benefit from multi-drifts seems to come from the longitudinal support provided by an existing lining behind the tunnel face. This effect can not be considered in two dimensional analysis without further assumptions. Sequential excavation is likely to cause more complicated three dimensional effects at the tunnel heading than full-face excavation

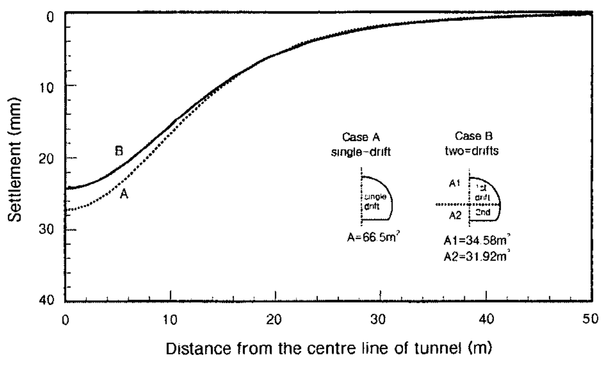
The inset figure in Fig. 19(a) shows a typical two-drift excavation emplcyed in the construction of Seoul Subway, where the total area of tunnel is A=66.5m^2, the area of the first drift is A_1=34.58m^2 and the area of the second drift is A_2=31.92m^2. Two dimensional modelling of this procedure may require some simplification. Although there have been several attempts to simulate multi-drift NATM tunnelling and the need for a variation of the empirical parameters has been noted, any suggestions for the choice of empirical parameters have rarely been made. A modification to the empirical parameters can be a device to examine the multi-drift sequence. In the time-based modelling method, this can be achieved by reducing the empirical parameter, α. One possible approach is proposed here assuming that the empirical factor for each drift is related to the area to be excavated. The assumption made is:

$$\alpha_i = \alpha\left(\frac{A_i}{A}\right)^n \tag{8}$$

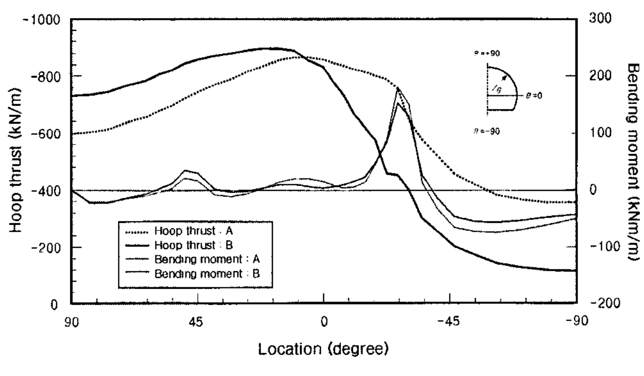
where α_i is the time control factor for the i^{th} drift, α is the time control factor for full-face excavation, A_i is the excavation area of the i^{th} drift, A is the total excavation area, n is a constant (n<1) and i is the number of drift.

This equation implies that the smaller the excavated area, the shorter the time required to excavate it, thus the placement of the lining will be faster than that of full-face excavation. By carrying out a series of analyses for different n values, a slight reduction in volume loss was obtained for n=1/3 and consequently α_1=0.37, and α_2 =0.36(where α=0.459).

The results for n=1/3 are shown in Fig. 19. The two-drift excavation gave about 10% less volume loss than the single-drift excavation. Similar trends for hoop thrust and bending moment are obtained for both cases, however, in the case of two-drift the lower lining below the spring line carries considerably less hoop thrust than for the single-drift case.



(a) Surface settlement troughs



(b) Hoop thrusts and bending moments

Fig. 19. Effect of multi-drift excavation

5. Conclusions

The existing 2-D modelling methods of tunnelling were reviewed. An improved modelling method based on time was adopted to account for three dimensional effect at the tunnel heading and time dependent effect. Typical analyses were performed for a single horse-shoe-shaped tunnel with the time-based method. Calculated surface settlements show excellent agreement with field data. Various analyses using with the 2-D time-based method identified the behaviour of the tunnel in decomposed granite soil:

(a) It is confirmed that generally the surface settlement trough due to a single tunnel can be represented by a Gaussian normal distribution curve defined by the parameters S_{max}- i ;

(b) A mixed-face condition where decomposed granite soil and weathered rock appear at the tunnel face, results in smaller hoop thrusts and bending moments in the tunnel lining;

(c) The general trends of ground behaviour around the circular tunnel are the same as those for the horse-shoe-shaped tunnel and the difference is not significant;

(d) For multi-drift excavation the empirical parameters for the two dimensional simulation can be evaluated from the equation: $\alpha_i = \alpha(A_i / A)^n$

Acknowledgement

The work presented in this Paper was sponsored by Seoul Metropolitan Government, an Overseas Research Award from Committee of Vice-chancellors Principals of Universities of United Kingdom and a British Chevening-Ministry of Science and Technology Joint Scholarship during the author's research at Imperial College. Their support is gratefully acknowledged.

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(received on Jul. 24, 2001, accepted on Dec. 18, 2001)