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사례연구를 통한 터널 하중의 예측
Estimation of Loads on Tunnel Lining Based on Case Studies

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요약 / ABSTRACT

터널 라이닝에 작용하는 하중을 예측하는 것은 터널설계에서 가장 중요한 문제 중의 하나이다. 하지만 수치해석적 방법외의 기존의 해석적 방법들은 터널건설 방법이나 지질학적인 다양성을 충분히 고려하지 않고 있다. 에드몬톤의 터널에서 실제로 측정된 하중을 기존의 해석적 방법을 이용해 얻어진 하중과 비교하였다. 하지만 기존의 방법들은 터널 하중을 예측하는데 만족하지 못한 결과를 보여 주었다. 터널 라이닝을 설치하기 이전에 터널 전면에서 일어나는 응력 감소를 고려해 주기 위하여 아이젠스타인-네그로의 방법과 기존의 방법을 결합하여 터널 하중을 예측할 것을 제안 하였다.

Estimation of loads on tunnel lining is one of the major issues to be addressed in the design of a tunnel. The existing analytical methods do not consider important details of construction and the variation of geology along the tunnel axis. The measured loads obtained from several sanitary and subway tunnels in Edmonton, Alberta, Canada, are compared with the lining loads calculated using the existing analytical methods. However, the existing methods are determined to be not fully satisfactory for the estimation of lining loads. To account for face and heading effects occurring prior to lining installation, the stress reduction factor determined using Eisenstein and Negro's method is used coupled with an analytical solution for calculation of lining loads.

INTRODUCTION

Prediction of lining loads is not an easy task due to uncertainties and variations of the ground conditions, the redistribution of the in-situ stresses related to the ground deformation before and after lining installation, and the differences in construction procedures. Therefore, most tunnels are often built too conservatively, i.e., more support is used than is necessary.

In practice, various lining thicknesses have been used for similar ground conditions. It has been recognized that such variations reflect the absence of consistent design principles. The objective of this study is to investigate realistic design loads on tunnel linings. The validity of the existing design methods is reviewed by comparing the calculated loads obtained using these methods with the field measurements obtained from several tunnels in Edmonton. An improved design method is proposed based on the review of the existing design methods.

COMPARISON OF FIELD MEASUREMENTS WITH THE RESULTS FROM THE EXISTING THEORIES

Tunnel construction in the City of Edmonton has been very active for storm and sanitary purposes. Since the early 1970's, the city has also been developing a Light Rail Transit (LRT) system. Tunnelling activity has been continuous since then with the growth of the city. The tunnels were mainly bored through till and clay shale, which are generally described as soft ground. Case histories of several sewer and LRT tunnels are summarized as shown in Table 1.

Lining loads calculated using three methods from ring and plate models, i.e., Peck et al.'s, Muir Wood's, and Einstein and Schwartz's, and one from numerically derived methods, i.e., Eisenstein and Negro's, are compared with the measured loads obtained from tunnels in Edmonton. The three methods from ring and plate models were chosen because these methods have been widely used by practical engineers due to their simplicity.

Peck et al.'s and Einstein and Schwartz's methods were described in detail by Peck et al.

Table 1. Primary Liner Loads in Edmonton Tunnels.

Tunnel	Method	Ground	Depth (m)	Extern. Dia.(m)	Support Type	n	Meth. of Measure.	References
A. Northeast Line	TBM	Till	10.2	6.1	R & L	1.3	Strain gauges (elect. resis. type)	Eisenstein and Thomson (1978)
B. LRT-South Extension	TBM	Till	11.8	6.2	R & L	0.40 -0.49	Load cells (vibr. wire type)	Branco (1981)
SLRT-Phase II								
C. (Section B2)	TBM	Till	15.8	6.3	R & L	0.79	Load cells	Tweedie et al. (1989)
D. (Section.C2)	SEM	Till	9.7	6.3	S & R	0.68	Flat jack tests	
E. (Section A1)	TBM	Sand	17.2	6.3	R & L	0.35	Load cells	
F. Whitemud	TBM	Clay Shale	47.2	6.05	R & L	1.10	Deformation mea.	Thomson and El-Nahhas (1980)
G. 170th Street	TBM	Till	20	2.56	R & L	3.50	Lagging deflection	
Exper. Tunnel								
H. (Section 1)	TBM	Till	27	2.56	R & L	1.92	Strain gauges (vibr. wire type)	Eisenstein et al. (1979)
I. (Section 2)	TBM	Till	27	2.56	PSL	4.08	Strain gauges (vibr. wire type)	
J. (Section 3)	TBM	Till	24	2.56	PSL	2.50 -2.91	Load cells (vibr. wire type)	
Banks of North Saska. River								
K. (Section 1)	TBM	Till	13.7	3.2	R & L	0.40	Load cells	Corbett (1984)
L. (Section 2)	TBM	Till	13.7	3.2	R & L	0.21	Load cells	
M. (Section 4)	TBM	Sand	16.7	3.2	R & L	3.87	Load cells	

Notes:R & L=Rib and Lagging; S & R=Shotcrete and Rib; PSL=Precast Segmented Lining

(1972) and Einstein and Schwartz (1979) respectively and will not be repeated here.

Muir Wood (1975) recognized a basic error in Morgan's paper in terms of the assumption that plane strain leads to plane stress. In this study, Morgan's equations were used for the calculation of thrust considering Muir Wood's corrected parameters. The change of lining loads due to direct compression of the lining presented by Muir Wood is also considered for the calculation of lining loads.

Eisenstein and Negro (1990) performed 3-D finite element parametric analyses to estimate the radial displacement at the point of lining installation, disregarding the lining's presence and the non-linearstress-strain relationship of the ground. Convergence curves were obtained by parametric 2-D finite element analyses, assuming a hyperbolic elastic material model for varying ground and geometric conditions. Stress reduction factors can be found combining the radial displacements and a full range of normalized convergence curves in a variety of geometric and geotechnical conditions, as presented by Negro (1988). Thrust forces on the lining were obtained using Hartmann's solution. The effects of the delayed lining installation, represented by the stress relaxation and ground stiffness degradation, were accounted for through the use of a reduced unit weight of the soil and a reduced

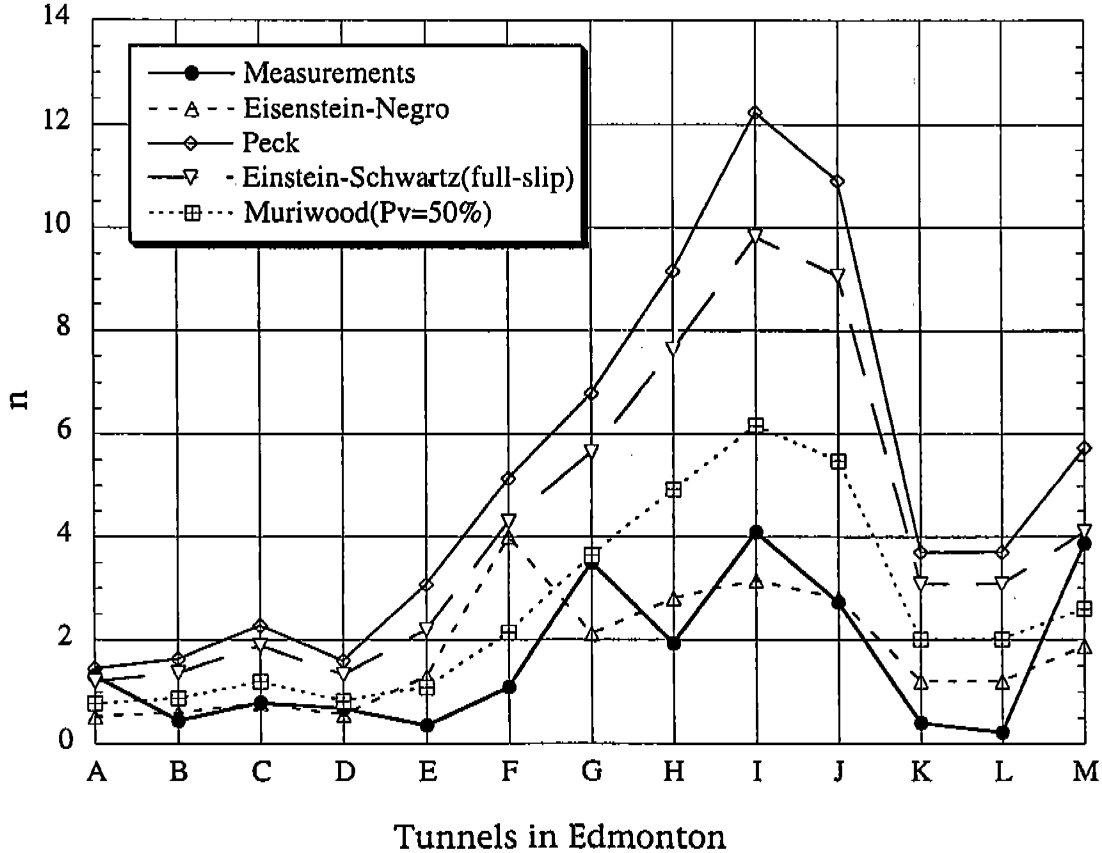

Fig. 1. Measured and Calculated n for the Linings in Edmonton Tunnels

ground stiffness. A full range of charts and diagrams facilitating the use of this method was presented by Negro (1988).

Soil pressures on tunnel linings can be expressed in the form of :

$$P=n\gamma D \qquad (1)$$

where P=the applied pressure, n=a dimensionless factor, γ = unit weight of soil, and D = tunnel diameter. The calculated lining loads from the four different design methods are compared with the actual load measurements as shown in Fig. 1. In all applications of the methods, no attempts were made to best fit the observed lining performances.

The figure clearly shows that Peck's method overestimates the lining load consistently except in tunnel A. Einstein and Schwartz's method also overestimates the lining load except in tunnels A and M. Muir Wood suggested taking only 50% of the overburden pressure. Muir Wood's method still overestimates the liner load by an average of 37% but underestimates in tunnels A and M.

Eisenstein and Negro's method gives the closest estimates, an average of 28% more than the actual lining loads measured. However, this method underestimates the lining loads in several tunnels. The following section describes the possible causes of these discrepancies between the measurements and predictions.

CONCLUSIONS VERSUS EXISTING DESIGN METHODS

Lining loads calculated using four different design methods were compared with actual load measurements in the previous section. Peck's method consistently gave much higher lining loads than those from measurements as expected. Peck's method was calculated based on the assumption of an overpressure loading condition, which implies that the tunnel opening has been excavated and supported even before the full overburden pressure is applied. As a result, Peck's method consistently overestimates the lining loads.

Einstein and Schwartz's method also generally overestimated the lining loads even though the method gave a better approximation than Peck's method. Their method assumes an excavation unloading condition, which indicates that the tunnel opening is excavated and supported after the full overburden pressure is applied. Therefore, stress redistribution induced by the opening was considered in their method. However, the opening is simultaneously excavated and supported in one step without consideration of the stress reduction occurring prior to lining installation.

Muir Wood suggested taking only 50% of the overburden pressure, taking into consideration some stress reduction of the ground around a tunnel opening before the lining was placed. Muir Wood's method gave a better approximation than the previous two methods, with an average of 37% overestimation of the lining load. Actually, the stress reduction factor of 50% worked reasonably well for soils in Edmonton but can be misleading for certain geologic materials.

The common problems with the above three methods are that they make assumptions about only concerning linear elastic ground, and there is uncertainty about the determination of the stress reduction factor. Eisenstein and Negro's method not only considered non-linear ground behaviour but also was capable of calculating the stress reduction factor. Therefore, the method gave the closest estimates of the actual lining loads. However, this method underestimated the lining loads in several tunnels, especially tunnels A, G, and M. In the case of tunnel A, a cover to diameter ratio, Z/D, is 1.17. Therefore, the ratio of 1.5 was used for the calculation of the lining loads because Negro (1988) provided charts and diagrams facilitating the usage of this method for a Z/D from 1.5 to 6. The increase of the ratio to 1.5 for the calculation of lining loads in tunnel A could increase the stiffness of soil and consequently decrease the load calculated. The lining load of tunnel G is underestimated, probably due to the inaccuracy of the measurements because the lagging deflection is not a reliable way to measure lining loads. However, this assumption cannot be confirmed. The high lining loads in tunnel M could be related to poor ground conditions coupled with increased overburden at the section due to the recent construction of an embankment on the ground surface. In fact, observed settlement in the tunnel was much greater than those of other sections due to poor ground conditions. The load increase caused by an embankment simulates the overpressure loading condition, which was assumed in Peck's method. This may be the reason that Peck's method gave a better approximation of lining loads for tunnel M than for other tunnels. In other words, Peck's method works well for a tunnel having a cover to diameter ratio less than 1.5, having poor ground conditions, or experiencing the change of overburden pressures.

In conclusion, Eisenstein and Negro's method gave the closest estimates of the actual lining loads. However, the method can be used reliably only for stable ground due to the assumptions of linear elastic ground and an unlined opening for the findings of stress reduction factors. The method also should be used carefully for very shallow tunnels because the method was developed based on the parametric finite element analyses within certain ranges.

ESTIMATES OF STRESS REDUCTION FACTORS

Eisenstein and Negro (1985) suggested that the stress reduction factor found using their method could be used coupled with any analytical solution for calculation of thrust forces and bending moments. The results from Eisenstein and Negro's method showed that the amount of stress release at lining activation for Edmonton tunnels ranged from 43% to 66%, with an average of 59%. This value compares favorably with Muir Wood's arbitrary reduction by 50% of the overburden stresses suggested for lining design. However, the 50% stress reduction is an arbitrary value, and various suggestions have been given by others.

Stress reduction factors can be found combining the radial displacements and a full range of normalized convergence curves in a variety of geometric and geotechnical conditions, as presented by Negro (1988). However, obtaining stress reduction factors using the table and diagrams can be tedious and time consuming even though it is not difficult. Therefore, using the table and diagrams, final three tables, which were not presented in this paper, were obtained for the stress reduction factors (α) as a function of the support delay length X for three different in-situ stress ratios by the current authors.

The tables clearly show several characteristics of stress reduction factors. First, stress reduction factors (α) do not increase much from an X/D of 1 to an X/D of 2 even though more stress is released as X/D increases. As a result, the stress reduction factors obtained for an X/D of 1 can be used for an X/D of greater than 1 without much error. Second, the stress reduction factors also were not sensitive to the values of Z/D, especially with a Z/D between 3 and 6, even though slightly more stress was released as Z/D increases. Therefore, the stress reduction factors for a Z/D of 3 can be used for a Z/D of 6.

Stress reduction factors cannot be used reliably in a shallow tunnel because arching may not be fully developed due to the shallow soil cover. Furthermore, O'Rourke (1984) suggested that the minimum practical thickness is usually enough for tunnels with a depth to centerline up to three diameters. Therefore, Peck's or Einstein and Schwartz's method without taking into account stress reduction factors is probably good enough for the prediction of lining loads in a shallow tunnel.

The predicted lining loads may be too conservative if arching develops around an excavation. However, because the soil behaviour is rather uncertain due to the shallow soil cover, and the minimum practical thickness is usually enough for tunnels, the conservative estimate does not create problems. Therefore, the stress reduction factors for an H/D of less than 3 do not have to be considered. As a result, Table 2 is suggested for the estimates of the stress reduction factors instead of the three tables for its simplicity.

Table 2. Stress Reduction Factors Suggested for the Proposed Design Method($3 \le Z/D \le 9$)

Ko	ϕ_e	X/D											
		0	0.1	0.2	0.25	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0≤
0.6	20	0.225	0.296	0.351	0.375	0.381	0.394	0.405	0.410	0.414	0.418	0.423	0.427
	30	0.268	0.379	0.466	0.504	0.514	0.535	0.554	0.561	0.568	0.575	0.582	0.589
	40	0.275	0.409	0.527	0.579	0.594	0.622	0.650	0.660	0.670	0.680	0.689	0.699
0.8	20	0.241	0.325	0.391	0.418	0.426	0.440	0.453	0.458	0.462	0.467	0.471	0.475
	30	0.255	0.361	0.448	0.487	0.497	0.518	0.537	0.544	0.551	0.558	0.565	0.572
	40	0.259	0.375	0.478	0.525	0.538	0.564	0.589	0.598	0.608	0.617	0.626	0.635
1.0	20	0.215	0.285	0.341	0.365	0.372	0.384	0.396	0.400	0.404	0.409	0.413	0.417
	30	0.229	0.314	0.388	0.422	0.432	0.450	0.468	0.475	0.482	0.488	0.495	0.501
	40	0.237	0.329	0.414	0.454	0.465	0.488	0.510	0.519	0.528	0.536	0.545	0.553

According to the study of case histories collected in places other than Edmonton, the table may be used for a Z/D of up to 9 or more. The study of case histories also showed that the in-situ stress ratio of unity could be used for tunnels having a ratio greater than 1.

Calculation of equivalent friction angles is needed for soils with a non-zero cohesive strength component and with failure ratios different from unity using the following equations :

$$\phi_a = \arcsin\left[\frac{1+(\frac{\sigma_3}{c})\tan\phi}{1+(\frac{\sigma_3}{c})\sec\phi}\right] \tag{2}$$

$$\phi_e = \arcsin(1 - R_f + R_f \csc\phi_a)^{-1} \quad (3)$$

where

φa = adjusted friction angle

φe = equivalent friction angle

c = cohesion of soil

R_f = failure ratio (generally from 0.7 to 1.0)

$$= \frac{(\sigma_1 - \sigma_3)_{failure}}{(\sigma_1 - \sigma_3)_{ultimate}}.$$

The support delay length X, which is the distance from the tunnel face to the leading edge of the lining, should be taken from the face of the tunnel rather than from the tail of the shield due to the existence of gap between the shield and soil. Ward (1969) and Belshaw and Palmer (1978) also observed that the use of a shield did not prove to be effective to reduce ground movements. However, the support delay length can be taken from the tail of the shield such as the case of Ashford tunnel because the average diameter of the shield was only 0.5 cm larger than that of the lining (Tattersall et al., 1955). The support delay length for the SEM tunnel is suggested taking from the face to the location where the invert is closed.

Stress reduction factors can be obtained easily and reliably if Table 2 is used. The method is strictly valid, though, for stable ground due to the assumption of linear elastic ground and an unlined opening for the finding of the stress release factor. From a large number of case histories, Negro (1988) concluded that the method gave good results in NATM tunnels regardless of the soil type, provided the tunnel face was stable and in TBM tunnels constructed in firm ground or even in less stable ground if the lining was activated in full contact with the soil at a short distance from the face.

ESTIMATES OF LINING LOADS USING THE STRESS REDUCTION FACTORS

The stress reduction factor found using Table 2 can be used coupled with Einstein and Schwartz's method for calculation of the lining loads. The dimensionless factor n can be expressed as follows according to Einstein and Schwartz's method combined with the stress reduction factors :

$$n = \frac{H(1-a_0)(1-a)}{D} \quad (4)$$

$$a_0 = \frac{CF(1-\nu)}{C+F+CF(1-\nu)} \quad (5)$$

where

C, F = compressibility and flexibility ratios as defined by Einstein and Schwartz (1979)

H = depth to tunnel centerline.

The dimensionless factor n is mainly a function of H, D, C, and α according to parametric analyses. The predicted values of n obtained using Eq. (4) are compared with the measured n for tunnels in Edmonton as shown in Fig. 2. One more case history from a tunnel in Edmonton is added to verify the proposed method.

The twin tunnels which are named Southbound (SB) and Northbound (NB) were excavated from the South Portal to the University Station. The excavated diameters of both tunnels are approximately 6.3m, and both tracks are parallel with approximately 10m between centres in typical tunnel sections. Steel ribs and two layers of shotcrete were used for a

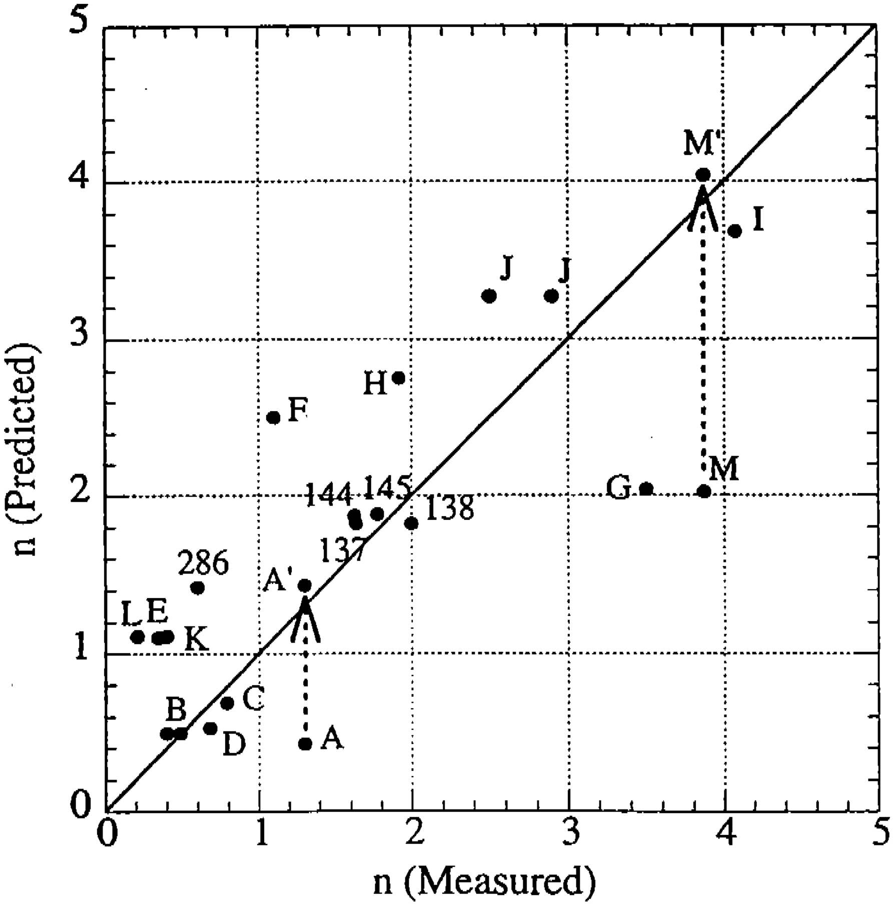

Fig. 2. Comparison of Predicted n Calculated using the Proposed Method with Measured n for Tunnels in Edmonton

primary lining system for both tunnels. Five sets of electrical resistance load cells were installed in the ribs of the two tunnels: ribs No.137, No.138, and No.286 in the SB tunnel and No.144 and No.145 in the NB tunnel. The tunnels were driven mainly in claystone bedrock except in rib No.286. In rib No.286, the tunnel was mainly driven through clay till except for the crown area which is composed of sand. The measured loads on ribs in the SB tunnel before the passage of the NB tunnel and loads on ribs in the NB tunnel were compared with the calculated loads. The load share between shotcrete and steel ribs was considered for the calculation of lining loads as suggested by Eisenstein et al.(1991).

The lining loads in rib No.286 were overestimated as shown in the figure probably due to the use of steel spiling near the crown of the section which has the same effect as increasing soil strength in the area. However, the existence of spiling can be disregarded for the purpose of the tunnel lining design because it is a safe-side approximation.

The proposed method is actually not suitable for tunnels A and M as explained before.

Tunnels A and M can be replotted as shown in the figures if Peck's method is applied without considering stress reduction factors. The lining load of tunnel G is underestimated, probably due to the inaccuracy of the measurements because the lagging deflection is not a reliable way to measure lining loads.

The proposed method has a tendency to give conservative results if these tunnels are excluded. However, the proposed method gives a reasonable approximation of n, considering the accuracy of the load measurements, limited knowledge of ground and lining properties, and existence of possible gaps between the lining and ground. Therefore, the proposed method can be used reliably for the quick estimation of lining loads of a tunnel.

CONCLUSIONS

An improved design method was proposed using the existing design methods. A method for determining the stress reduction factors was also described. The stress reduction factors can be included for the estimates of the lining loads if the tunnel has a stability ratio less than 2 or is constructed according to the modern tunnelling philosophy in which displacements are controlled. Therefore, the assumption would be mainly applicable to stiff or dense soils, where good ground control construction is accomplished. It could also be applied to softer or weaker ground if an internal pressure, such as compressed air or slurry, is applied to the tunnel walls and face, a condition which will ensure a limited degree of yielding around the tunnel.

The prediction of lining loads requires accurate information on the ground and lining properties, which is not always possible to obtain and which often varies along the tunnel section. Furthermore, the lining loads are affected by construction procedures, which also vary from one project to another depending on the tunnelling practice of the region and the skill of the tunnel builders. Therefore, the proposed method is recommended as an approximate guideline for the design of tunnels, and the results should be confirmed by field measurements. This is the reason that in-situ monitoring should be an integral part of the proposed design method.

REFERENCES

Belshaw, D. J. and Palmer J. H. L., 1978, Results of a program of instrumentation involving a precast segmented concrete-lined tunnel in clay. Canadian Geotechnical Journal, Vol. 15, 573-583.

Branco Jr., P., 1981, Behavior of a shallow tunnel in till, M. Sc. Thesis, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, 351 p.

Corbett, I., 1984, Load and displacement variations along a soft ground tunnel, M. Sc. Thesis, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, 246p.

Einstein, H. H. and Schwartz, C. W., 1979, Simplified analysis for tunnel supports, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT4, 499-518.

Eisenstein, Z. and Thomson, S., 1978, Geotechnical performance of a tunnel in till, Canadian Geotechnical Journal, Vol. 15, 332-345.

Eisenstein, Z., El-Nahhas, F., and Thomson, S., 1979, Pressure- displacement relations in two systems of tunnel lining, Proceedings of 6th Pan-American Conference on Soil Mechanics, Lima, 85-94.

Eisenstein, Z., Kuwajima, F. M. and Heinz, H. K., 1991, Behaviour of shotcrete tunnel linings. Proceedings, Rapid Excavation and Tunnelling Conference, Seattle, 47-57.

Eisenstein, Z. and Negro Jr., A., 1990, Integrated design method for shallow tunnels in soft ground, Presented at the JTA Tunnel Symposium in Tokyo, Japan, Vol. 2, 27-36.

Muir Wood, A. M., 1975, The circular tunnel in elastic ground, Geotechnique, 25, No. 1, 115-127.

Negro, A. Jr., 1988, Design of shallow tunnels in soft ground. Ph. D. Thesis, Department of Civil Engineering, University of Alberta, Edmonton, 1480 p.

O'Rourke, T. D., 1984, Guidelines for Tunnel Lining Design, American Society of Civil Engineers, 82 p.

Peck, R. B., Hendron, A. J., and Mohraz, B., 1972, State of the art of soft-ground tunneling, RETC Proceedings, Vol. 1, 259-286.

Tattersall, F., Wakeling, T. R. M., and Ward, W. H., 1955, Investigations into the design of pressure tunnels in London Clay, Institution of Civil Engineers Proceedings, London, Vol. 4, Part 1, 400-471.

Thomson, S. and El-Nahhas, F., 1980, Field measurements in two tunnels in Edmonton, Alberta, Canadian Geotechnical Journal, Vol. 17, 20-33.

Tweedie, R. W., Harris, M. C., Gerber, G. E., and Eisenstein, Z., 1989, Ground and structure monitoring for the Edmonton SLRT-Phase II Tunnels, Canadian Tunnelling, 61-72.

Ward, W. H., Peck, R. B., 1969, Deep excavations and tunneling in soft ground, Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, 320-325.

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