5. Estimation for Primary Tunnel Lining Loads

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Estimation for Primary Tunnel Lining Loads

by

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ABSTRACT

Prediction of lining loads due to tunnelling is one of the major issues to be addressed in the design of a tunnel. The objective of this study is to investigate rational and realistic design loads on tunnel linings. Factors influencing the lining load are summarized and discussed. The instruments for measuring the lining loads are reviewed and discussed because field measurements are often necessary to verify the design methods.

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 an underground Light Rail Transit system. The load measurements obtained from these tunnels are compared with the results from the existing design methods. However, none of the existing methods are totally satisfactory. Therefore, there is some room for improvement in the prediction of lining loads.

The convergence-confinement method is reviewed and applied to a case history of a tunnel in Edmonton. The convergence curves are obtained from 2-D finite element analyses using three different material models and theoretical equations. The limitation of the convergence-confinement method is discussed by comparing these curves with the field measurements. Three-dimensional finite element analyses are performed to gain a better understanding of stress and displacement behaviour near the tunnel face.

An improved design method is proposed based on the review of existing design methods and the performance of numerical analyses. A specific method or combination of two different methods is suggested for the estimation of lining loads for different conditions of tunnelling. A method to determine the stress reduction factor is described. Typical values of dimensionless load factors nD/H for tunnels in Edmonton are obtained from parametric analyses. Finally, the loads calculated using the proposed method are compared with field measurements collected from various tunnels in terms of soil types and construction methods to verify the method. The proposed method gives a reasonable approximation of the lining loads.

The proposed method is recommended as an approximate guideline for the design of tunnels, but the results should be confirmed by field measurements due to the uncertainties of the ground and lining properties and the construction procedures.

This is the reason that in-situ monitoring should be an integral part of the design procedure.

1. Introduction

1.1 General

Prediction of lining loads due to tunnelling is one of the major issues to be addressed in the design of a tunnel. However, the problem is not easily solved 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.

It is a well-known fact that tunnel lining seldom carries the full total load of the soil or rock located above the tunnel crown. 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 rational and realistic design loads on tunnel linings.

1.2 Scope and Organization of the Thesis

There are many existing lining design methods available. The response of the ground and support during excavation should be fully understood to review the problem of the existing design methods. Two or three dimensional finite element analyses can be used for this purpose.

An improved design method is proposed based on the review of the existing design methods. The best way to evaluate the validity of the method is to compare the calculated loads with the field measurements. Therefore, loads calculated using the proposed method are compared with field measurements from the case histories collected from many different areas to verify the method.

Chapter 2 presents discussions on the soil response to tunnelling, factors influencing the load on the liner, and available design methods to estimate the lining loads. It is very important to understand factors influencing the lining load to predict the load reliably. The existing design methods are classified based on the calculation procedure on which a particular method is developed. The validity of the design methods is discussed briefly based on literature reviews.

Field measurements are often necessary to verify the design methods or assumptions and improve models of ground behaviour. Therefore, the instruments for measuring the lining loads are reviewed in Chapter 3. The methods for the processing of measurements are also presented for two different lining systems. Finally, certain instruments are recommended for the measurement of lining loads for specific conditions.

The validity of the existing design methods is reviewed in Chapter 4 by comparing the loads calculated using the methods with the field measurements obtained from several tunnels in Edmonton. Conclusions concerning the existing design methods are presented based on the comparison. The use of correction factors for the existing design methods is also discussed in this chapter.

The design of a tunnel liner is a ground and structural problem. Ground-support interaction is a consequence of the resistance with which the liner reacts against the movement of the surrounding ground into the excavated opening. The convergence-confinement method (CCM) is one approach to the analysis of ground-support interaction. In Chapter 5, the convergence-confinement method is reviewed and applied for a case history of a tunnel in Edmonton. The convergence curves are obtained from two-dimensional finite element analyses using three different material models and theoretical equations. The limitation of the CCM is also discussed by comparing these curves with the actual field measurements obtained from a tunnel in Edmonton.

Three-dimensional finite element analyses are performed in Chapter 6 in order to have a better understanding of stress and displacement behaviour near the tunnel face. The influence of the construction sequences in ground-support interaction is investigated using the 3-D analyses. The possible causes of the differences in the final equilibrium stresses and displacements on the liner between the 2-D and 3-D analyses are presented. The difficulties involved in the 3-D analyses are also discussed in this chapter.

An improved design method is proposed in Chapter 7 based on the review of the existing design methods and numerical analyses. A method to determine the stress reduction factors is described in detail. In addition, a specific method or combination of two different methods is suggested to use for each type of tunnelling for the estimation of lining loads. Finally, the results from the proposed method are compared with field measurements of the case histories collected from many different areas to verify the method.

Chapter 8 presents a brief summary of this research and the main conclusions.

Since my original Ph.D thesis is composed of more than 300 pages, it is impossible to include all of the contents in this paper. Therefore, I will summarize it as much as I can to show you what I did for my Ph.D study.

2. Methods for Predicting Lining Loads

The lining is designed to support the weight of the overburden and a horizontal pressure equal to some fraction of it. The primary lining is usually designed to resist all transient loads developed during construction activities as well as the short-term ground loads. The secondary lining is used to ensure a safe support of the tunnel for the any other additional loading resulting from future changes in the overall physical conditions and the possible increased long-term ground loads.

However, tunnel linings seldom carry the full total load of the overburden. The soil above the tunnel is only partly supported by the liner. What occurs in all tunnels is that in-situ stresses are redistributed around the opening due to the inherent shear strength and continuity of the ground. This transfer of pressure from a yielding mass of soil onto adjoining stationary parts is called the arching effect. The lining theoretically has to support only those stresses not arched to the adjacent ground.

The prediction of lining loads for the lining design has always been the goal of tunnel engineers. However, problems exist because of the uncertainties and variations in the ground conditions, the redistribution of the in-situ ground stresses related to the ground deformations before and after lining installation, and construction procedures such as the length of the period during which the excavation is left unsupported. This chapter presents discussions on the soil response to tunnelling, factors influencing the load on the liner and available design methods to estimate the lining loads.

There are several basic requirements of a good design method. First, the design method should be simple to use. Duddeck and Erdmann (1985) reviewed the progress of the development of design models. They concluded that the available design methods are simple enough for practical applications. In other words, if a design method is very complex or time consuming to apply, the method will not be widely used by practical engineers. Second, the design method should consider the stress release occurring before the installation of a liner in some way. Third, the method should take into account the plastic behaviour of the ground as well as that of elastic ground.

The instruments for measuring the lining loads are reviewed and discussed in the following chapter because field measurements are often necessary to verify the design methods. The validity of the existing design methods is reviewed in Chapter 4 by comparing the results from the methods with the field measurements obtained from several tunnels in Edmonton.

3. Tunnel Load Measurements

Prediction of lining loads due to tunnelling is one of the major issues to be addressed in the design of a tunnel. The existing design methods do not consider some of the details of construction and the variation of geology along the tunnel section in longitudinal and vertical directions. Therefore, field measurements are often necessary to verify the design methods or assumptions and improve models of ground behaviour. The results can be used in the design of upcoming tunnel sections for the same tunnel project or for future tunnels working in similar material with the similar construction methods.

The lining stress or load can be measured directly or indirectly using pressure cells, flat jack tests, lagging deflection, rod extensometers, strain gauges, or load cells. The magnitude and distribution of stresses acting on the lining can be obtained from the measurement. In this chapter, the instruments for measuring the lining load are reviewed and discussed in detail.

Pressure cells are not generally recommended for measuring lining loads because of the long and unsuccessful history of attempting to do so. Flat jack tests can be an economical and fast procedure to evaluate the stresses in concrete linings. However, more case histories should be collected to improve the accuracy of the method. Lagging deflection can be used for estimating lining loads in a rib and lagging support system. Again, the method should be applied with caution because the amount of lagging deflection varies depending on several factors as explained in Section 3.2.3. The use of rod or tape extensometers for estimating lining loads requires several assumptions, which make the method only an approximate solution.

It was shown that strain gauges and load cells are the most effective and reliable ways for measuring lining loads. If measurements from both strain gauges and load cells are obtained in combined lining systems, the lining loads shared between two different lining types should be considered when calculating an average lining load. The pattern of strain throughout the lining may be highly variable and difficult to convert into stress without certain assumptions of the stress distribution. Therefore, a large number of strain gauges are generally required to examine the overall behaviour of the lining because a strain gauge provides reliable measurement of strain at only one point in the lining. On the other hand, one or two load cells provide an overall average measurement of normal loads in the lining. Therefore, using load cells is more cost effective than using strain gauges. However, the assumption of stress distribution around a tunnel should be well approximated to calculate reliably the average radial stresses acting on the lining.

The loads due to other than earth pressure may affect the measurements of lining loads. Special care should be taken for the measurements of strain gauges in the ribs and precast concrete segmented linings because stresses due to the installation procedure such as outward expansion of the linings by the radially directed rams may affect the readings. Measurements from load cells are not much affected by the expansion of the linings because the load cells are generally activated after the pressure in the expansion jacks is released. The forces resulting from the longitudinal reaction of the mole may be significant for the measurements of lining loads. In order to isolate the effects of the loads from the mole, differential strains can be used rather than absolute strains. In concrete linings, high lining strains may develop locally as a result of creep, shrinkage, and temperature changes. Therefore, strain measurements in concrete linings should be corrected to consider these factors.

Long-term load measurements in a tunnel are important to check the stability of a tunnel because lining loads generally increase as time passes. Therefore, considering the high humidity and dirt conditions in a tunnel, the vibrating wire strain gauges or load cells are recommended for the load measurements in a tunnel. The geology and construction sequence, including the location of the tunnel face with respect to the instrumented lining, should be recorded in detail because they are closely related to the variation of the lining load. Several ribs in a row, usually three or four consecutive ones, should be instrumented to allow for variation in the load due to variations in geology and support installation details. In the following chapter, measured lining loads from case histories are presented and compared with the lining loads from existing theories.

4. Comparison of Lining Loads from Measurements and Existing Design Methods

4.1 Introduction

Methods for predicting and measuring lining loads were reviewed in the previous two chapters. Many factors contribute to the differences in the lining loads of many different tunnels as explained before. The validity of the existing design methods, three from ring and plate models and one from numerically derived methods, is reviewed in this chapter by comparing the loads calculated using the methods with the field measurements obtained from several tunnels in Edmonton. The use of correction factors for the existing design methods is also discussed in this chapter.

4.2 Comparison of Field Measurements with the Existing Theories

Soil pressures on tunnel linings can be expressed either in terms of the percentage of overburden pressure or in the form of:

$$P=n\gamma D$$

where

P= the applied pressure

n= a dimensionless factor

γ= unit weight of soil

D= tunnel diameter.

A summary of the loads carried by the primary support system on Edmonton tunnels is shown in Table 1. Four different design methods, those of Peck et al., Muir Wood, Einstein and Schwartz, and Eisenstein and Negro, were compared with the actual load measurements in this study. In all applications of the methods, no attempts were made to best fit the observed lining performances.

The measured and calculated dimensionless factors (n) for the primary lining in Edmonton tunnels are shown in Figure 1.

4.3 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. In conclusion, Eisenstein and Negro's method gave the closest estimates of the actual lining loads.

4.4 Application of Stress Reduction Factors to the Existing Design Methods

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. For example, Muir Wood presented a closed form solution, recommending a 50 % reduction of the full overburden pressures to account for face and heading effects occurring prior to lining installation. The 50 % stress reduction is an arbitrary value, and various suggestions have been given by others, e.g. about a 33 % stress reduction as suggested by Panet (1973). Einstein and Schwartz (1980) also suggested that the stress reduction factor could be between 15 % and 100 % according to simple analytical and numerical techniques and case study data.

Instead of applying this rather arbitrary reduction, the reduced unit weight of soil found from Eisenstein and Negro's method was used for each tunnel to reevaluate the lining load from the analytical methods. The results are shown in Figure 2. The results encourage the use of the reduced unit weight considering that the stress release occurred before lining installation. The use of stress reduction factors, coupled with other analytical solutions, for the prediction of lining loads is further discussed in detail in Chapter 7.

In conclusion, none of the above methods are totally satisfactory for the estimation of lining loads even though certain methods can give reasonable results under specific conditions. Therefore, there is some room for improvement in the prediction of lining loads. An improved design method will be proposed in Chapter 7 based on the review of existing design methods in this chapter and performance of numerical analyses in the following two chapters.

5. Ground- Support Interaction During Excavation

5.1 Introduction

Tunnel excavation changes the state of stress and stiffness around the opening. The installation of a support system, which interacts with the soil, further alters the stress and stiffness of the ground. Ground-support interaction is a consequence of the resistance with which the liner reacts against the movement of the surrounding ground into the excavated opening. Kuesel (1987) stated that the design of a tunnel liner was not a structural problem but a ground and structural problem, with the emphasis on the ground because the loads acting on a liner are not well defined and its behaviour is controlled by the properties of the surrounding ground.

The transfer of loads from the excavated ground to the tunnel liner depends on the stress-strain-time properties of the ground, the relative stiffness between the liner and the ground, the initial stresses existing in the ground, the method of excavation, type and manner of placement of the liner and the distance between the tunnel face and the point of liner activation. The convergence-confinement method has played a major role in understanding the ground-liner interaction in advancing tunnels. In this chapter, the convergence-confinement method is reviewed and applied for a case history of a tunnel in Edmonton. The convergence curves are obtained from two-dimensional finite element analyses using three different material models and theoretical equations. These curves are compared with actual field measurements.

5.2 Analysis of Experimental Tunnel

An Experimental tunnel was chosen for the present analyses because of the availability of considerable field measurements of tunnel performance as well as soil parameters. The tunnel is described in detail in Sec. 4.2.6. SIGMA/W was used for the two dimensional finite element analysis. SIGMA/W, developed by GEO-SLOPE International Ltd, is a finite element program that can be used to conduct two-dimensional or axisymmetric stress and deformation analyses of earth structures.

The finite element meshes used for the analyses are shown in Figure 3. Initial in-situ stress before excavation was applied to the section assuming 21 KPa of unit weight and a uniform in-situ stress ratio. The deformations and strains from the first step have no relevance and are considered to be zero. Two-dimensional load-deformation analyses were performed through excavation of a tunnel section using three different material models: linear-elastic, hyperbolic and elastic-plastic.

The elements in the tunnel section were deactivated in one step to simulate the excavation. The internal forces, which are equal to in-situ stresses but opposite direction, were applied to the nodes along the excavated tunnel boundary to prevent any soil movement due to excavation. The procedure to calculate the equivalent internal forces for initial stresses is shown in Figure 4. The upper diagram shows the contributing areas for planar two-dimensional elements with width equal to 1 unit. The forces, Fx and Fy, were reduced to zero in 11 different steps with a 10 % deduction of forces for each step.

5.3 Summary and Conclusions

The convergence-confinement method is one approach to the analysis of ground-support interaction. The method was reviewed in detail in this chapter. The CCM has a great advantage in explaining the phenomena governing the ground-liner interaction. However, the method may not be appropriate as a direct design tool except for special cases such as deep tunnels and highly pressurized ground due to the limitations mentioned in Sec. 5.2.2. To have a better understanding of the method, especially the effect of a material model on the convergence curve, two-dimensional finite element analyses were performed.

The procedures to simulate the excavation for two-dimensional analyses were presented in Sec. 5.3.2. The convergence curves were obtained from two-dimensional finite element analyses using three different material models and theoretical equations. The limitation of the use of 2-D finite element methods combined with CCM for the estimation of the final load and displacement around a tunnel was discussed by comparing these curves with the actual field measurements obtained from a tunnel in Edmonton.

Most tunnels clearly show three-dimensional behaviour within the region bounded by one to two diameters ahead of the face to one diameter behind the location of liner activation. The ground displacements occurring ahead of the face could not be considered in the two-dimensional finite element analyses. In addition, two-dimensional analyses can not model the fact that as the tunnel advances, the excavation is done in a zone ahead of the face in which the stress condition has already been changed by the approach of the face.

To gain a better understanding of stress and displacement behaviour near the tunnel face, three-dimensional finite element analyses are performed in the following chapter.

6. Three Dimensional Response of Ground

6.1 Introduction

The plane strain finite element analysis is still widely used for tunnel designs because of its simplicity and low cost compared to three-dimensional finite element analyses. However, in the vicinity of the advancing face, the longitudinal displacements are not zero.

Eisenstein and Branco (1985) observed the longitudinal movement of the ground ahead of the advancing face towards the face using inclinometers. The soil moved back to its original position after the TBM passed by the points that initially moved towards the face. Therefore, the final differential longitudinal displacements from the field measurements were small close to a two dimensional plane strain condition. However, as Branco (1981) pointed out, the response of strain path dependent soil would not be completely described in a two dimensional plane strain representation because the strains were not zero during the tunnel advance.

In addition, as mentioned in the previous chapter, the plane strain analyses have difficulty determining the relationship between the position of liner installation and the amount of ground displacement occurring before installation of the liner. Furthermore, two dimensional analyses cannot model the fact that as the tunnel advances, the excavation is done in a zone ahead of the face in which the stress condition has already been changed by the approach of the face. Therefore, in order to gain a better understanding of stress and displacement behaviour near the tunnel face, three-dimensional finite element analyses were performed and presented in this chapter.

6.2 Three-Dimensional Finite Element Analyses

In order to understand the behaviour of stress and displacement near the tunnel face, three-dimensional finite element analyses were performed using the program SAGE developed by Chan (1985) at the University of Alberta. The Experimental tunnel was chosen for the present analyses because of the availability of considerable field measurements of tunnel performance as well as soil parameters. The tunnel was bored through till and had an excavated diameter of 2.56 m. Each metre of the precast concrete lining consists of four segments with 0.11 m thickness.

The same cross section as for the 2-D analyses was used for the transversal section of the 3-D mesh in order to compare the results between them. The longitudinal cross section of the 3-D mesh is shown in Figure 5. A total of 10 slices with 1220 finite elements and 5101 nodal points were used for the analyses. The final mesh was finer than most of three-dimension meshes found in the literature.

6.3 Convergence Curves

The concept of convergence-confinement method was explained in detail in the previous chapter. However, the previous convergence curves do not include the actual load transfer mechanisms existing at the tunnel face.

The ground reaction curves or convergence curves are drawn for slices 5 and 6 by combining radial stresses and displacements obtained in the middle section of each element. The displacements are obtained for nodes at the excavation line. The stresses for the crown, springline and floor are obtained from the points located about 2 cm, 1.5 cm and 1 cm respectively from the excavated boundary, which are close enough from the nodal points used to get the displacements.

Figure 6 showed the convergence curves of the springline for slices 5 and 6 without the installation of the liner. The radial stresses are normalized to the in-situ stress. The convergence curves for slices 5 and 6 showed similar trends. The displacements started to occur ahead of the face with a slight increase of radial stresses from the in-situ stress due to the arching effect along the longitudinal direction of the tunnel. The dimensionless radial stresses decreased to about 0.91 when the face advanced 0.325 m behind the reference point A. The radial stresses droped close to zero at point B after the face passed the points. The radial stresses of unsupported section between points B and C did not drop to zero because the stresses were obtained about 1.5 cm inside of the excavated boundary and the mesh was not fine enough to get zero stress in the boundary. The convergence curves between points B and C did not coincide because the excavation round length after the passage of reference points of slices 5 and 6 were slightly different. However, the final stresses and displacements for those two points were exactly the same when the face advanced far away from those points as shown at point C.

Figure 7 presents the convergence curves obtained from two-dimensional linear elastic analyses with the results of 3-D analyses for slice 5 without liner. The final equilibrium points are almost the same for the both analyses but the ground responses are completely different. The conventional plane strain convergence curve could not consider the 3-D arching effect occurring ahead of the face and the fast stress reduction in the unsupported section.

The convergence curves of slice 5 with the installation of the liner are shown in Figure 8. The curves clearly show the influence of lining installation on the stress and displacement distributions at the crown, springline, and floor. There were increases of radial stresses without much displacement as the tunnel face advances further after the liner was installed at slice 5 in step 9. The stresses and displacements at those points were finally stabilized when the tunnel face advanced about 2 diameters away from the points. The final loads and displacements on the lining depend on the relative stiffness of the lining and the ground and the length of unsupported section at the time of lining installation.

6.4 Summary and Conclusions

In this chapter, 3-D finite element analyses have been reviewed and performed to gain a better understanding of stress and displacement behaviour near the tunnel face. The radial displacements and the radial stresses of the ground have been obtained from the 3-D analyses with and without the liner. The results were summarized in the previous sections. The stress distribution along the tunnel clearly showed the influence of the construction sequences in ground-support interaction. The stresses on the liner may vary because of the length of the lining and the distance between the lining and the tunnel face even though all the other material properties of the ground and the liner, the tunnel geometry and the in-situ stresses are the same.

The convergence curves were obtained by combining the radial stresses and the displacements. The final radial stresses from the 2-D analyses with the confinement curve and from the 3-D analyses did not coincide probably because the convergence curve of 2-D was obtained from the analysis without the liner installation. The difference was larger as the distance of the unsupported section was reduced in the sequence of tunnel construction. Therefore, the convergence-confinement method should be used carefully when the excavation has a shorter length of unsupported section throughout the construction.

The responses of the ground around an advancing tunnel certainly show three-dimensional nature and cannot be considered properly by a two-dimensional model. However, 3-D analysis is not an easy task to perform because of the difficulties involved in the preparation of the input data and the handling of the output data. This is one of the reason that a simple design method should be available considering the three-dimensional nature of the ground around an advancing tunnel. In the following chapter, an improved design method is proposed to have better approximation for the behaviour of the tunnel without applying complex analyses.

7. Validation and Recommendations

7.1 Introduction

The validity of the existing design methods was reviewed in Chapter 4 by comparing the loads calculated using the methods with the field measurements obtained from several tunnels in Edmonton and other areas. However, none of the existing methods were totally satisfactory for the estimation of lining loads because they could not consider all the factors affecting lining loads. Because certain methods could give reasonable results under specific conditions, an improved design method is proposed in this chapter using the existing design methods.

A design method for the prediction of lining loads should include the decrease of lining loads due to the stress release before lining installation as explained in the previous two chapters. The use of stress reduction factors, coupled with other analytical solutions, for the prediction of lining loads was discussed briefly in Sec. 4.7. The results encouraged the use of the reduced unit weight considering that the stress release occurred before lining installation. Therefore, a method to determine the stress reduction factors is described in detail in this chapter. In addition, a specific method or combination of two different methods is suggested for each type of tunnelling for the estimation of lining loads. Finally, the proposed method is compared with field measurements from the case histories collected from many different areas to verify the method.

7.2 Estimates of Stress Reduction Factors

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 author.

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.

As a result, Table 2 is suggested for the estimates of the stress reduction factors instead of the three tables for its simplicity. 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:

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

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

where

ϕa = adjusted friction angle

ϕe = equivalent friction angle

c = cohesion of soil

R_f = failure ratio (generally from 0.7 to 1.0)

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

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.

7.3 Suggested Methods for the Prediction of Lining Loads

Tunnels are divided into several types, depending on the soil behaviour around an excavation, a factor which is mainly controlled by construction methods, soil types, and tunnel depths. Tunnels are divided into two main groups depending on the tunnel depth. The classification is based on the fact that a certain minimum thickness is required for the construction of the lining up to a certain depth regardless of the soil loads. The deeper tunnels of the two, having a depth to centerline to diameter ratio greater than three, are further subdivided into three groups depending on whether the tunnels have positive face control or not.

A specific method or combination of two different methods is suggested for each type of tunnelling for the estimation of lining loads. The proposed method is strictly valid for tunnels in soil or soft rock with a ratio of tunnel depth to diameter up to 6 for tunnels constructed without face control and up to about 8 for tunnels with face control. However, the method may be used for slightly deeper tunnels than these as shown later in this chapter. The depth generally covers most urban tunnels. A point to be mentioned is that the calculated lining load is an average one. Therefore, to calculate lining loads for a combined lining system, lining loads should be divided according to Young's modulus and the area of each of these linings as explained in Sec. 3.3.1.

7.3.1 A Depth to Centerline to Diameter Ratio Up to Three

Stress reduction factors cannot be used reliably in such a shallow tunnel because arching may not be fully developed due to the shallow soil cover. Furthermore, the whole soil section located above the tunnel crown can directly exert pressure on the tunnel lining because the plastic zone may be extended to the ground surface. 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.

In addition, the minimum thickness for cast-in-place concrete linings is generally controlled by the clearance between the initial support ribs and the interior form required for the concrete.

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.

Lining loads for tunnels constructed using the cut and cover method can also be estimated using Peck's method without taking into account stress reduction factors because the ground acts passively as a dead load, a loading condition which is similar to that of Peck's.

7.3.2 A Tunnel Depth to Diameter Ratio Greater than Three

7.3.2.1 No Face Control

Eisenstein and Negro (1985) suggested that the stress reduction factor (α) found using their method could be used, coupled with any analytical solution, for calculation of thrust forces and bending moments. The stress reduction factor can be accounted for through the use of a reduced unit weight of the soil. However, the method is strictly valid for stable ground due to the assumption of linear elastic ground and an unlined opening for the finding of the stress release factor as mentioned in Sec. 7.2.

The stress reduction factor can be obtained from Table 2 and combined with Einstein and Schwartz's method to calculate the lining loads for tunnels constructed according to the modern tunnelling philosophy and without the positive face control.

The average support thrust T_{av} can be expressed as follows according to Einstein and Schwartz's method:

$$T_{av} = \frac{1}{2} PoR(1 + Ko)(1 - a_o)$$

$$a_o = \frac{CF(1 - \nu)}{C + F + CF(1 - \nu)}$$

$$C = \frac{ER(1 - \nu_l^2)}{E_l A_l (1 - \nu^2)} \qquad F = \frac{ER^3(1 - \nu_l^2)}{E_l I_l (1 - \nu^2)}$$

where

C = compressibility ratio

F = flexibility ratio

A_l = average cross-sectional area of the support per unit tunnel length and

Po = vertical ground stress at the centre line of a tunnel.

It is customary to express soil loads on tunnels either in terms of the percentage of overburden pressure or in the form of:

$$P = n\gamma D$$

or $$n = \frac{P}{\gamma D}$$

where

n = a dimensionless factor that depends on soil type and tunnelling method.

Then

$$n = \frac{2T_{av}}{R(1+Ko)\gamma D}$$

$$= \frac{Po(1-a_0)}{\gamma D}$$

$$= \frac{\gamma_{red} H(1-a_0)}{\gamma D}$$

where

γ_{red} = reduced unit weight of the soil caused by stress release around a tunnel

H = depth to tunnel centerline.

Since

$$\frac{\gamma_{red}}{\gamma} = (1-\alpha)$$

therefore

$$n = \frac{H(1-a_0)(1-\alpha)}{D} \qquad (7.1)$$

7.3.2.2 Positive Face Control with Compressed Air or Fluids

Positive face control is necessary to maintain face stability and to prevent any excessive deformation at the face by allowing a limited amount of stress release to occur. Therefore, compressed air or slurry pressure is generally applied near the face. The pressure should be lower than the overburden pressure to prevent any possible blow-out. Eisenstein and Ezzeldine (1992) recommended applying about 40 % of the overburden pressure for a tunnel in Edmonton constructed using a hydroshield boring machine. Since less than overburden pressure is usually applied to the face, there should be a certain amount of stress release at the time of lining installation.

Deere et al. (1969) presented a ground reaction curve which showed the effect of air pressure. They assumed that the in-situ stress was reduced by the magnitude of the air pressure and that the lining load would increase again upon removal of the air pressure. However, they did not show how to evaluate the radial displacements of the tunnel wall at the time of lining installation. Negro (1988) suggested reducing the tunnel depth equivalent to the magnitude of the air pressure. However, he did not consider the stress increase on the lining after the removal of the air pressure.

It is obvious that the amount of stress release at the tunnel wall is reduced at the time of lining installation due to the positive face control compared to that of a tunnel constructed without face control. However, the amount of stress release is unknown. Deere er al. and Negro's concepts can be combined to evaluate the effect of positive face control on the lining loads. First, in-situ stresses are reduced by the magnitude of the air or slurry pressure. The reduction of in-situ stresses is achieved by reducing the soil cover depth above a tunnel. The stress reduction factor can be found as explained in the previous section but using the reduced soil depth. The stress acting on the lining before the removal of the air or slurry pressure can be found by multiplying the reduced unit weight of the soil by the reduced soil depth. The air or slurry pressure should be added to the stress found above to obtain the final stress acting on the lining upon removal of the air or slurry pressure.

Eq. (7.1) should be modified to take into account all of these factors described above. It was shown that n can be expressed as follows:

$$n = \frac{Po(1 - a_o)}{\gamma D}$$

Since

$$\frac{P_i}{\gamma} = H_{eq}$$

Therefore

$$\begin{aligned} n &= \frac{[\gamma_{red}(H - H_{eq}) + P_i](1 - a_o)}{\gamma D} \\ &= \frac{\gamma_{red}(H - H_{eq})(1 - a_o)}{\gamma D} + \frac{P_i(1 - a_o)}{\gamma D} \\ &= \frac{(1 - \alpha)(H - H_{eq})(1 - a_o)}{D} + \frac{P_i(1 - a_o)}{\gamma D} \qquad (7.7) \end{aligned}$$

where

Pi = air or slurry pressure

Heq = soil depth equivalent to Pi.

Therefore, the dimensionless factor n for a tunnel constructed using air or slurry pressure can be estimated using Eq. (7.7).

7.3.2.3 Positive Face Control using Jet-Grouted Piles

If a tunnel is driven through unstable material such as loose sand, jet-grouted piles should be used prior to tunnelling to enhance face stability during the tunnelling operation. In this case, higher soil strength should be used, especially in terms of Young's modulus, for the calculation of the lining loads considering the use of jet-grouted piles. The low lining loads may be a result of the fillcreting operations used in the tunnel. Calculated lining loads will be lower than those measured if the increase of soil strength is not considered for the prediction of lining loads. In summary, Eq. (7.1) is applicable for tunnels belonging to this group to estimate the dimensionless factor n but with the increased Young's modulus considering the use of jet-grouted piles.

7.4 Verification of the Proposed Method Using Case Histories

There are several basic requirements of a good design method. First, the design method should be simple to use. Second, the design method should consider the stress release occurring before the installation of a liner in some way. Third, the method should take into account the plastic behaviour of the ground as well as that of elastic ground. The proposed method totally satisfies the first two conditions and partly satisfies the third conditions due to the assumption of stable ground used for the development of the stress reduction factors. However, the most important factor to be a good design method should be the accuracy of the predicted lining loads. Therefore, the measured and calculated lining loads are compared in this section to check the validity of the proposed design method.

Fourteen case histories were collected to verify the proposed method. These case histories are summarized in Appendix 1. The ratios of tunnel depth to tunnel diameter, H/D, considered were from 1.2 to 17.9, even though the method was not recommended for use in tunnels having a H/D greater than 6.

The proposed lining loads were compared with measured loads as shown in Figure 9. The stress reduction factor was not considered for tunnels having a ratio of H/D of less than three as suggested in Sec.7.3.1. The predicted lining loads for these shallow tunnels, i.e., Tunnel Nos. 2, 8, 9, and 11, were conservative, as expected. However, the conservative estimate does not create problems because the minimum practical thickness of the lining usually governs the tunnel design. The figure also shows that the lining loads in Nipawin drainage tunnel No. 7 are very much underestimated. One of the possible reasons for this could be related to the fact that the space between the soil and lining was not fully filled. Matheson et al. (1986) stated that the volume of pea gravel injected into the gap between the soil and lining was about 80 % of the annular void. If the soil moved into the lining due to the existence of the gap behind the lining, the assumption of stable ground used for the development of the stress reduction factors could not be justified. However, the proposed method generally gives good approximations of the lining loads for most of the tunnels located in places other than Edmonton as shown in Figure 9.

7.5 Summary and Conclusions

An improved design method was proposed in this chapter using the existing design methods. A method for determining the stress reduction factors was also described. The values of the stress reduction factors for the different soil strengths, tunnel depths, and in-situ ratios were presented as tables. A specific method or combination of two different methods was suggested for the estimation of lining loads for various conditions of tunnelling.

Tunnels were divided into two main groups depending on the tunnel depth. For a shallow tunnel having a depth to centerline to diameter ratio up to three, Peck's or Einstein and Schwartz's method without taking into account stress reduction factor was suggested for the prediction of lining loads. Lining loads for tunnels constructed using the cut and cover method can also be estimated using Peck's method without taking into account stress reduction factors.

Deep tunnels, having a depth to centerline to diameter ratio greater than three, were further subdivided into three groups depending on whether the tunnels have positive face control or not. For a tunnel constructed without face control, the stress reduction factors could be used coupled with Einstein and Schwartz's method for the estimates of lining loads. The stress reduction factors can be easily obtained using Table 2. Typical values of nD/H for tunnels in Edmonton were obtained from parametric analyses. The calculated values of n from the table were slightly more conservative than those from the suggested equation. However, the table can still be used for quick estimations of lining loads for tunnels in Edmonton.

For a tunnel constructed with compressed air, the reduced soil depth, considering the air pressure, was used for the calculation of the stress reduction factors. Einstein and Schwartz's method was also suggested for estimating the lining loads. The air pressure should be added to the stress found above to obtain the final stress acting on the lining upon removal of the air pressure. It was also observed that increased soil strength, especially in terms of Young's modulus, should be used for the calculation of the lining loads for a tunnel constructed with positive face control using jet-grouted piles.

Finally, the loads calculated using the proposed method were compared with field measurements collected from various tunnels in terms of soil types and construction methods to verify the method. The proposed method gave reasonable approximations of the lining loads for tunnels in Edmonton and other areas, considering the accuracy of the load measurements, limited knowledge of ground and lining properties, and the existence of possible gaps between the lining and ground. Therefore, the proposed method can be used reliably for the estimation of lining loads of a tunnel. 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 modern tunnelling philosophy in which displacements are controlled.

There have been no absolute methods which can be used for estimating lining loads for all the various conditions of tunnelling in terms of the ground and construction methods. The author does not believe that such a method will be developed even in the future because 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. In other words, tunnelling is really an art rather than a science due to the nature of the ground and construction procedures. Therefore, the proposed method is recommended as an approximate guideline for the design of tunnels, but the results should be confirmed by field measurements. This is the reason that in-situ monitoring should be an integral part of the design procedure.

8. References

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Table 1 Primary Liner Loads in Edmonton Tunnels

Tunnel	Method	Ground	Depth* (H)	Dia.* (D)	H/D	Pv (KPa)	PL (KPa)	PL/Pv x 100 (%)	h (m)	n=h/D	Method of Measurements
A. Northeast Line	TBM	Till	10.2	6.1	1.7	214	169	79	8.05	1.3	(e)
B. LRT-South Extension	TBM	Till	11.8	6.2	1.9	248	52-64	21-26	2.48-3.07	0.40-0.49	(a)
SLRT-Phase II											
C. (Section B2)	TBM	Till	15.8	6.3	2.5	332	105	32	5.0	0.79	(a)
D. (Section C2)	SEM	Till	9.7	6.3	1.5	204	89.37	44	4.3	0.68	(b)
E. (Section A1)	TBM	Sand	17.2	6.3	2.7	361	46	13	2.19	0.35	(a)
F. Whitemud Creek	TBM	Clay Shale	47.2	6.05	7.8	991	140	14	6.67	1.10	(c)
G. 170th Street	TBM	Till	20	2.56	7.8	420	188	45	8.95	3.50	(d)
Experimental Tunnel											
H. (Section 1)	TBM	Till	27	2.56	10.5	567	103	18	4.9	1.92	(e)
I. (Section 2)	TBM	Till	27	2.56	10.5	567	219.6	39	10.46	4.08	(f)
J. (Section 3)	TBM	Till	24	2.56	9.4	504	135-156	27-31	6.40-7.44	2.50-2.9	(a)
Banks of North Saska. River											
K. (Section 1)	TBM	Till	13.7	3.2	4.3	288	26.9	9	1.28	0.40	(a)
L. (Section 2)	TBM	Till	13.7	3.2	4.3	288	14.2	5	0.68	0.21	(a)
M. (Section 4)	TBM	Sand	16.7	3.2	5.2	351	259.85	74	12.37	3.87	(a)

Notes: (a) From load cells
(b) From flat jack tests in the shotcrete liner
(c) From the deformation of the ribs
(d) From lagging deflection
(e) From the measurements of strains in the rib
(f) From the measurements of strains in the precast segmented liner

* units in meters

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

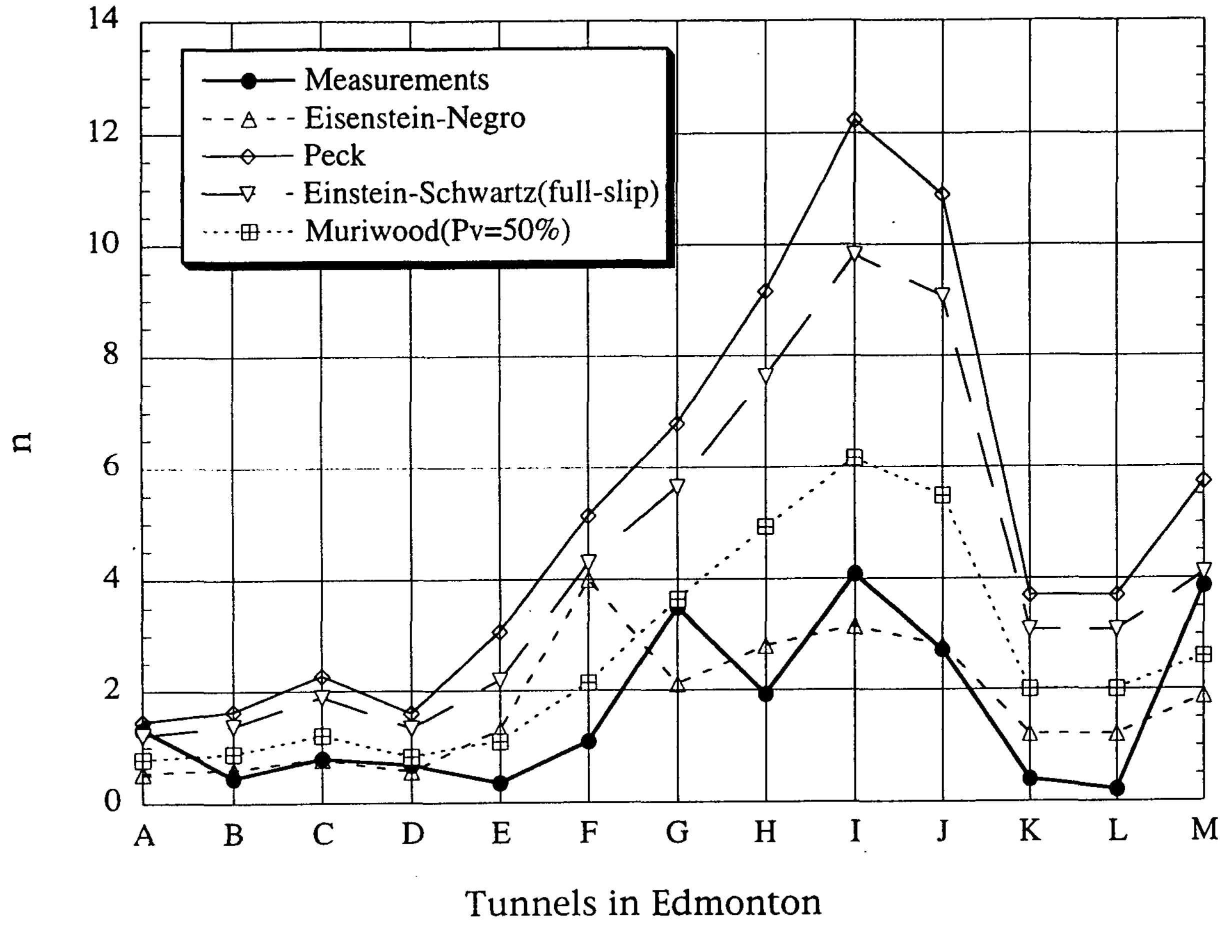

Figure 1 Measured and Calculated n for the Linings in Edmonton Tunnels

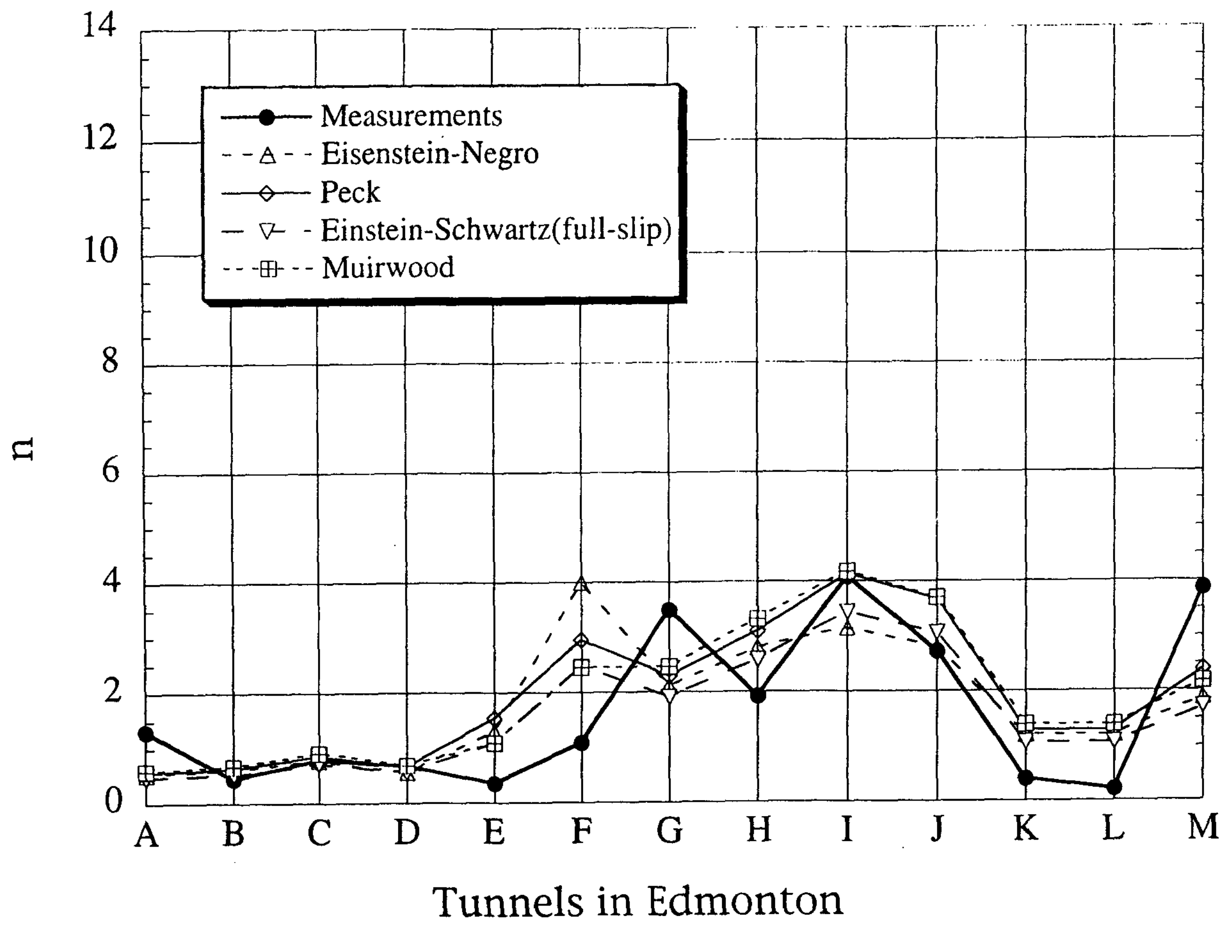

Figure 2 Measured and Calculated n for the Linings in Edmonton Tunnels Using Reduced Unit Weight

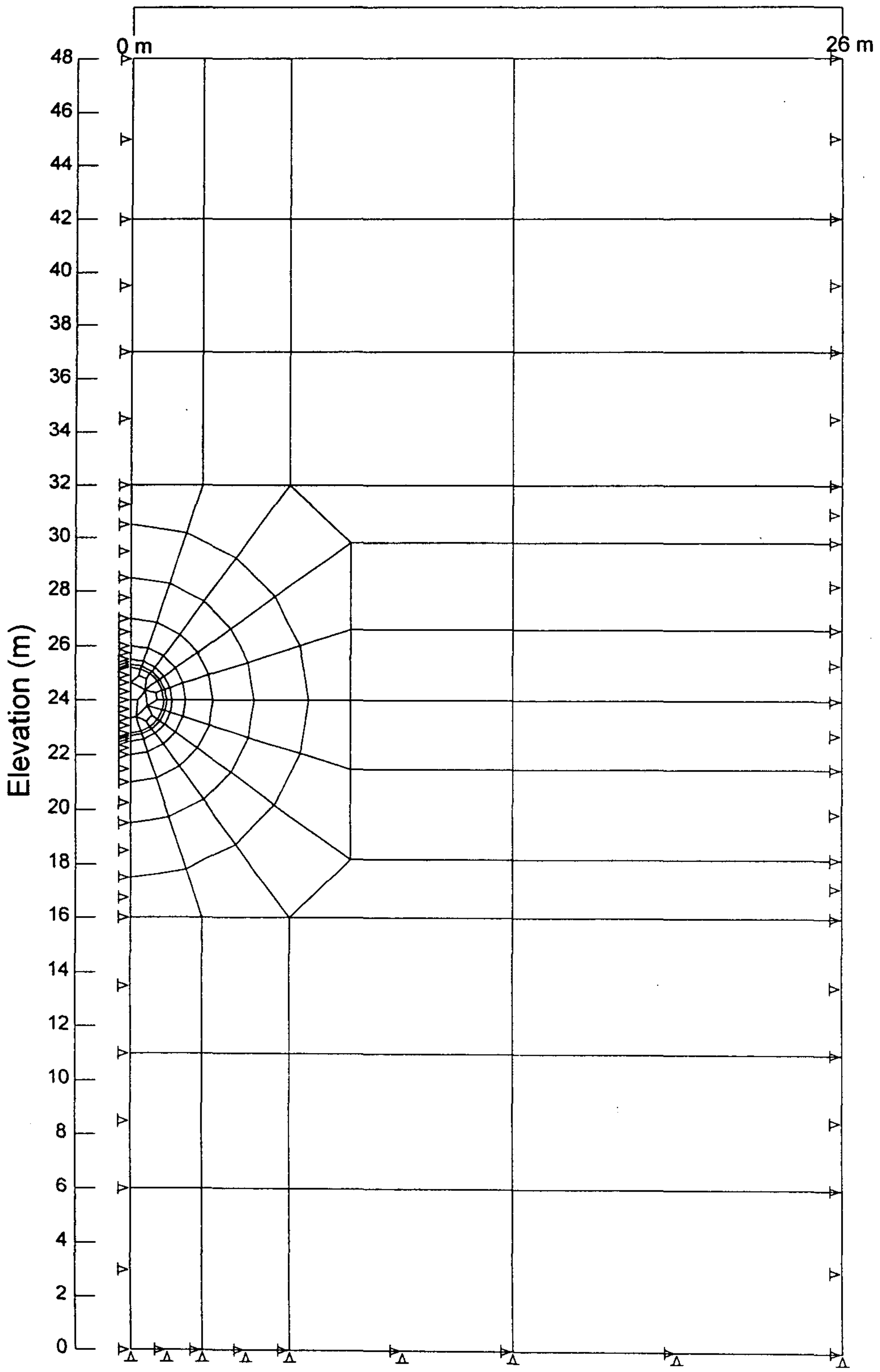

Figure 3 Two-Dimensional Meshes of the Experimental Tunnel

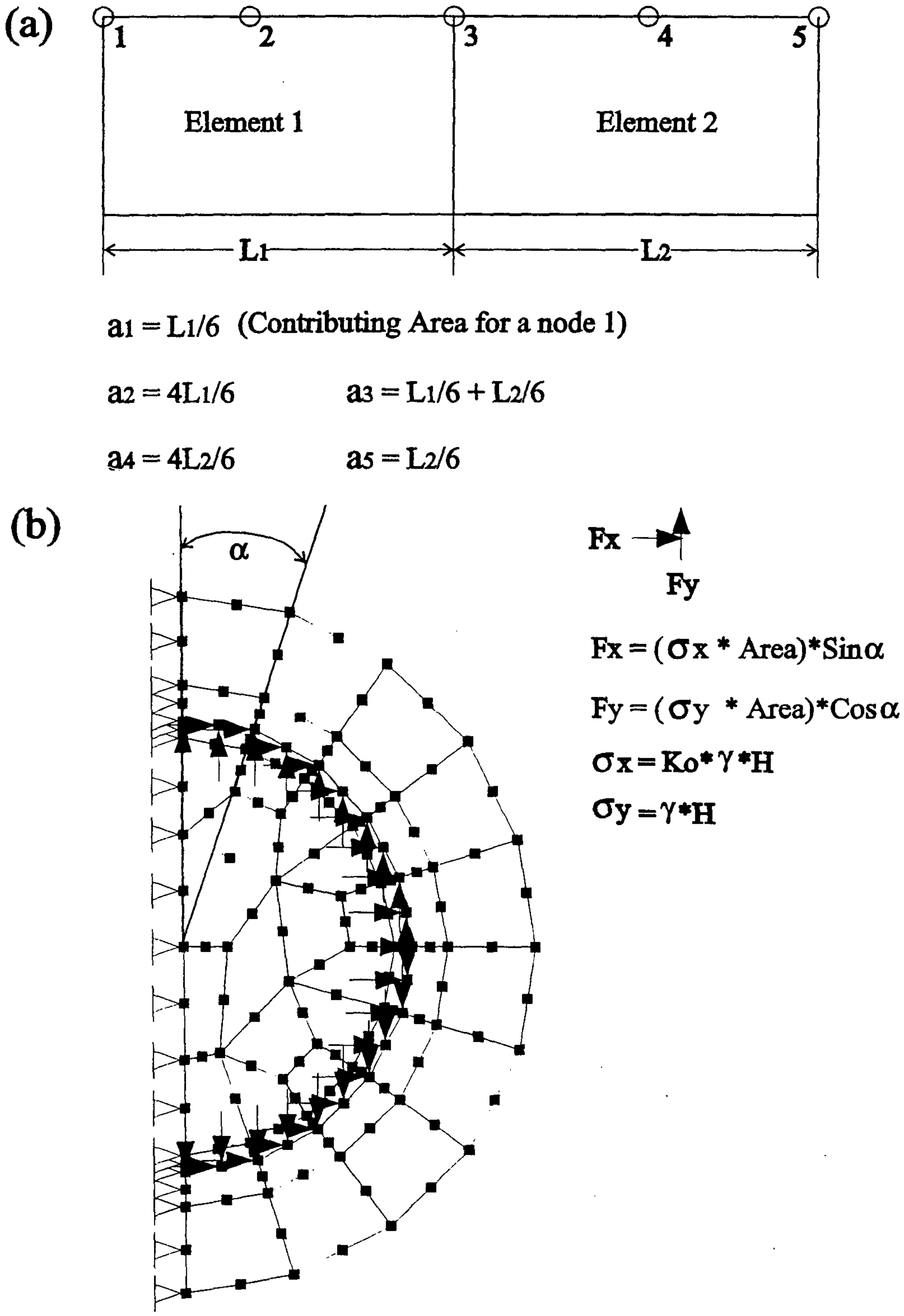

Figure 4 (a) Contributing Areas for Uniform Stresses
(b) Calculation of Equivalent Loads for Initial Stresses

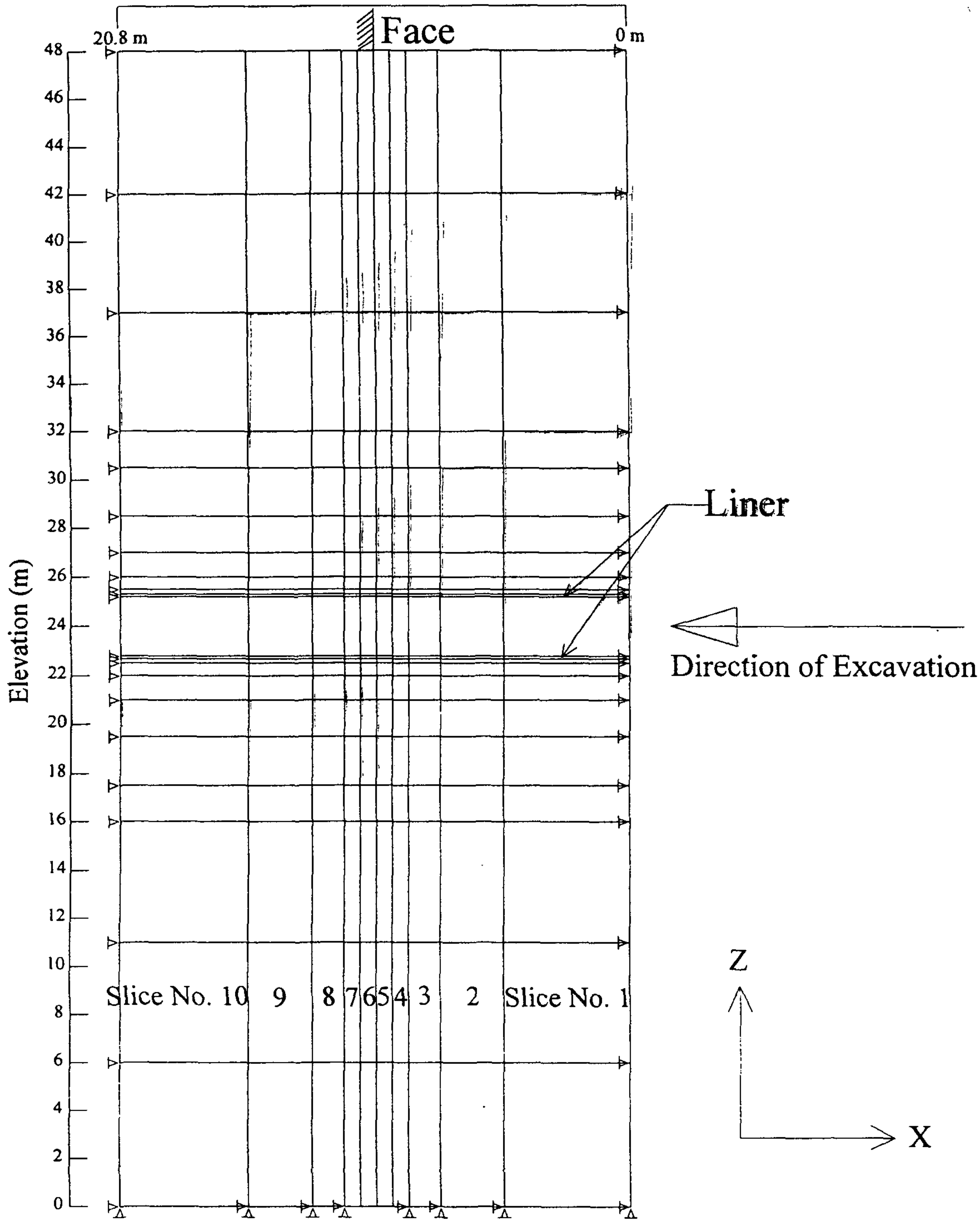

Figure 5 Longitudinal Cross Section of the 3-D Mesh of the Experimental Tunnel

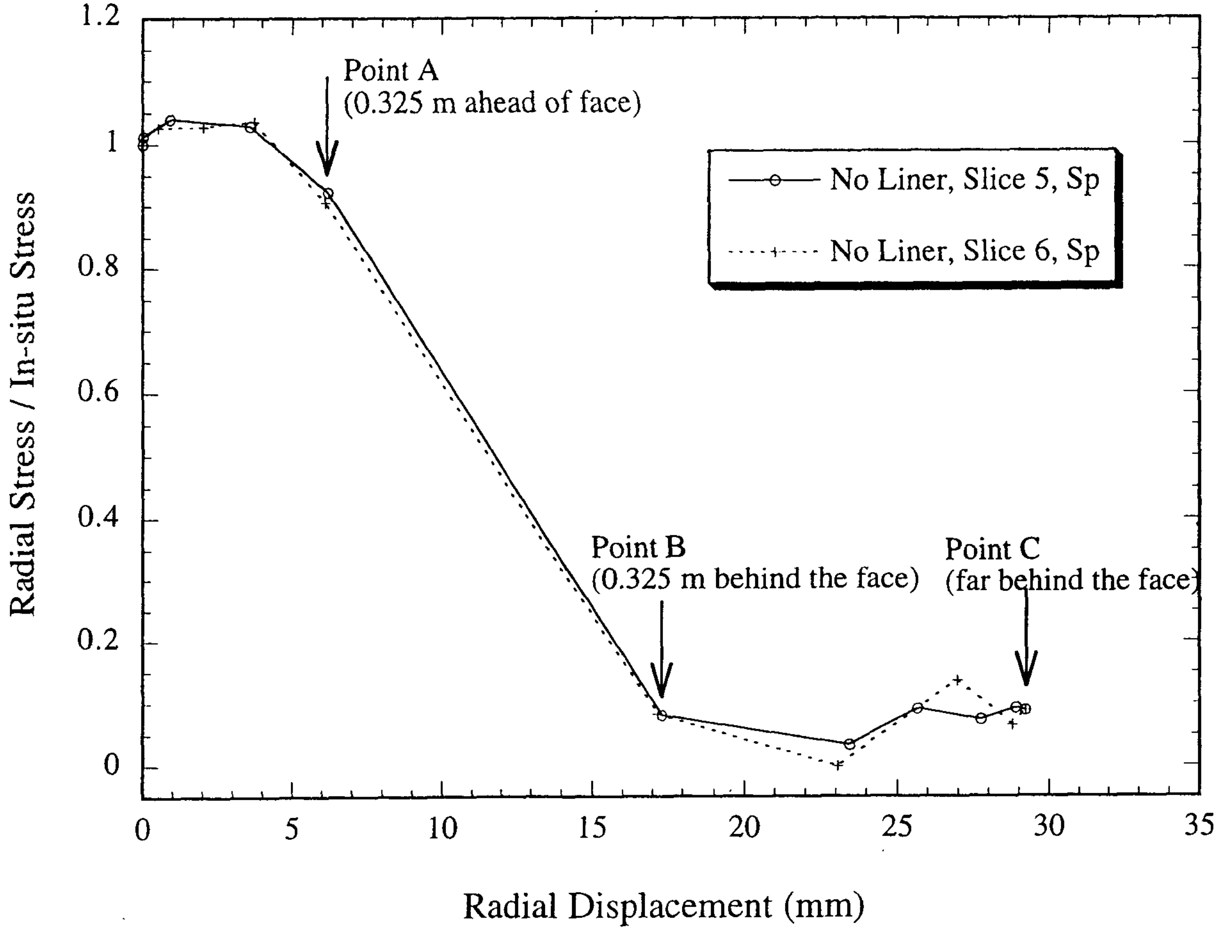

Figure 6 Comparison of Convergence Curves of Springline for Slices 5 and 6 without Liner

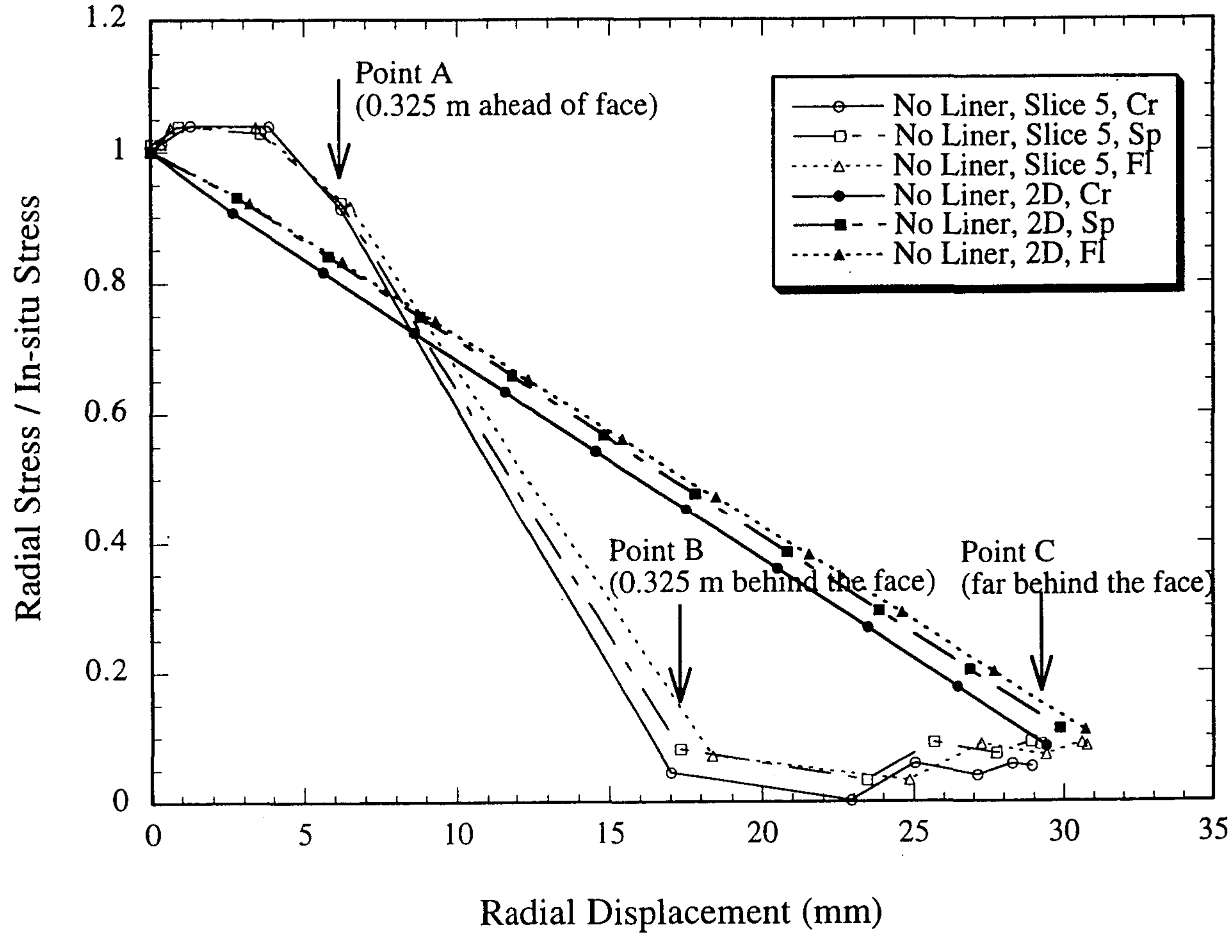

Figure 7 Comparison of Convergence Curves of Slice 5 for 2-D and 3-D Analyses

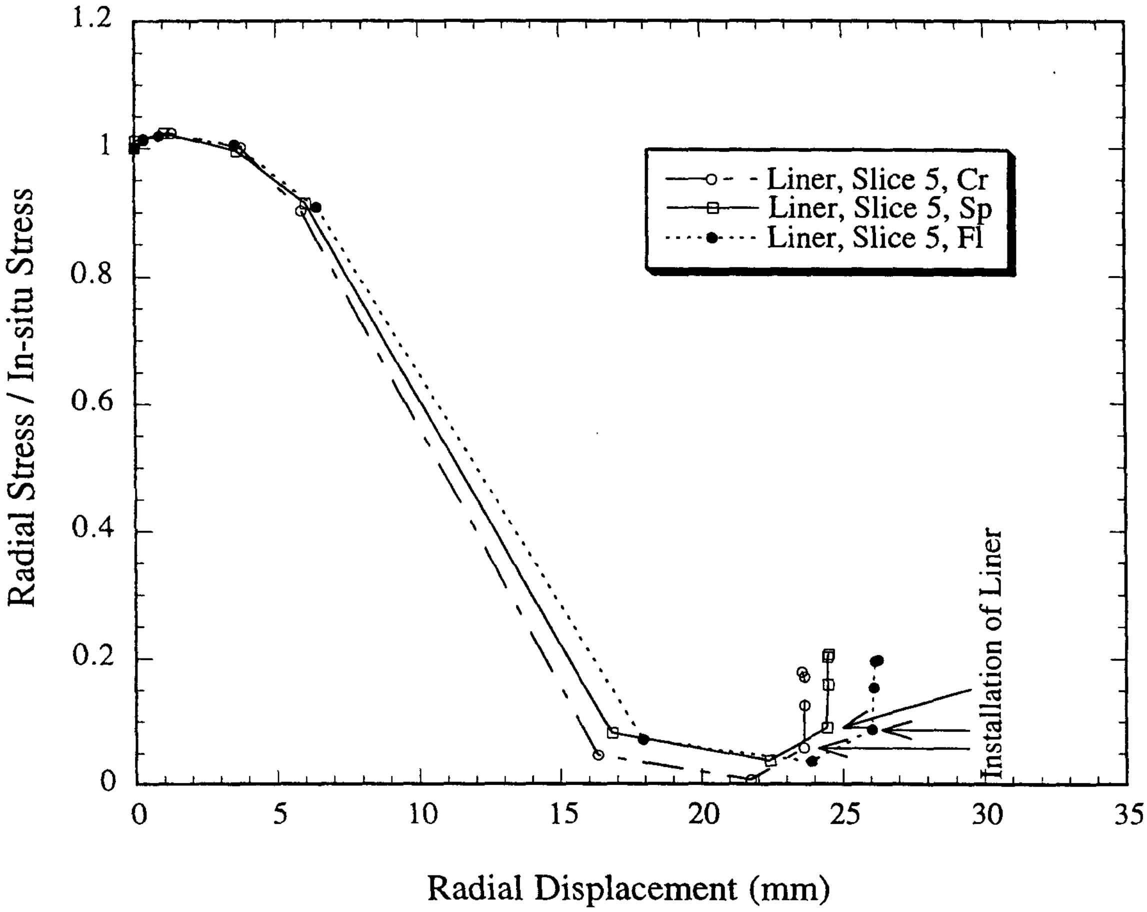

Figure 8 Convergence Curves for Slice 5 from 3-D Analyses with Liner Installation

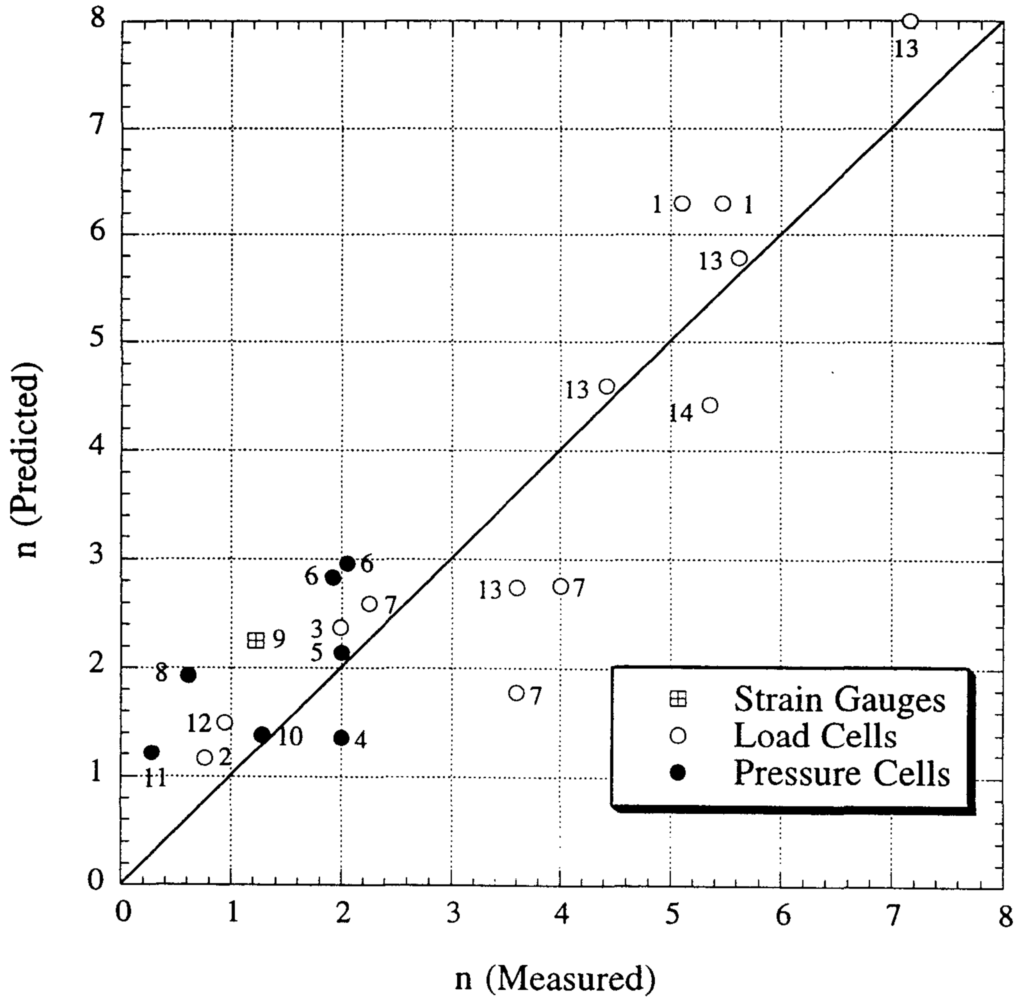

Figure 9 Comparison of Predicted n Calculated using the Proposed Method with Measured n for the Tunnels Located other than Edmonton