Stability of D-shaped tunnels in a Mohr–Coulomb material under anisotropic stress conditions

M.A. Meguid and R.K. Rowe

Abstract: The near-face stability of D-shaped tunnels excavated in a Mohr–Coulomb material subjected to anisotropic in situ stress conditions is investigated in the present study. The construction of the intake tunnel of the Darlington Nuclear Generating Station is analyzed using three-dimensional elasto-plastic finite element analysis. The induced displacement and stresses around the tunnel opening as the face advances are compared to the field measurements recorded during the tunnel excavation. The effect of rock mass strength reduction on the tunnel deformation, face stability, and distribution of stresses at the tunnel circumference is investigated for different in situ stress conditions. When the ratio of rock mass strength to overburden pressure falls below 0.5, excessive deformation occurrs and squeezing of the rock mass becomes a problem that can cause instability of both the tunnel circumference and the face.

Key words: weak rock, tunnelling, horizontal stresses, three-dimensional, finite element, excavation, face stability.

Résumé : Dans le présent article, on étudie la stabilité près de la surface de tunnels en forme de D excavés dans un matériau Mohr–Coulomb soumis à des conditions de contraintes in situ anisotropes. On analyse le cas de la construction du tunnel de la Station nucléaire de Darlington au moyen d'une analyse tridimensionnelle élasto-plastique en éléments finis. Le déplacement et les contraintes induits autour de l'ouverture du tunnel alors que la face progresse sont comparés aux mesures sur le terrain enregistrées durant l'excavation du tunnel. Pour différentes conditions de contraintes in situ, on étudie l'effet de la réduction de la résistance du massif de roche sur la déformation du tunnel, sur la stabilité de la face et sur la distribution des contraintes le long de la circonférence du tunnel. Quand le rapport de la résistance du massif de roche sur la pression des terres sus-jacentes tombe sous 0,5, une déformation excessives se produite et l'écrasement du massif rocheux devient un problème qui peut causer une instabilité de la circonférence du tunnel de même que de la face.

Mots clés : roche molle, creusage de tunnel, contraintes horizontales, tridimensionnel, éléments finis, excavation, stabilité de la face.

[Traduit par la Rédaction]

Introduction

In situ stresses play a major role in the design and performance of many subsurface structures including tunnels, underground storage facilities, and power generating structures. In the past three decades, there have been several cases in Canada and worldwide where the presence of high in situ horizontal stresses has caused serious problems during and after tunnel excavations (White and Russell 1982; Lo 1989). Springline closure and invert heave were reported in tunnels constructed in Ontario and parts of the United States.

Several researchers (Lo et al. 1992; Hoek and Marinos 2000; Hoek 2001) investigated the stability of tunnels in weak rock under initially isotropic stress conditions. Hoek and Marinos (2000) showed that a plot of tunnel strain against the ratio of rock mass strength to in situ stress provides a basis for estimating the potential for tunnel instability. Lo et al. (1992) introduced the load sharing ratio concept to evaluate the instability potential for tunnels excavated in a thick fault zone. Both investigations were limited to tunnels excavated in rock mass subjected to isotropic stress conditions.

In this study, the case of constructing the D-shaped intake tunnel of the Darlington nuclear generating station is analyzed using three-dimensional (3D) elasto-plastic finite element analysis. The resulting displacements and stresses are compared with the field measurements taken during the tunnel excavation. The analysis of the Darlington tunnel is used to validate the numerical model and to provide an example for the actual response of the rock mass under given anisotropic in situ stresses.

The objective of this study is to investigate the stability of D-shaped tunnels constructed in anisotropic in situ stress conditions and demonstrate some aspects of the 3D behaviour of the D-shaped tunnels. A detailed 3D finite element analysis, which explores the progressive development and evolution of induced displacements during the advancement of a tunnel face in a wide range of rock mass quality and initial in situ stresses, is conducted. The analyses concentrate on the stability at the face as well as the circumference of D-

Received 14 February 2005. Accepted 6 December 2005. Published on the NRC Research Press Web site at http://cgj.nrc.ca on 8 February 2006.

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 doi:10.1139/T06-004

shaped tunnels excavated in high in situ horizontal stress conditions. The significance of these effects will be subsequently discussed with respect to potential rock mass instability during and after the tunnel construction.

Geological conditions and rock properties

The tunnel investigated is a nuclear-powered generating station constructed in 1982 and located on the shore of Lake Ontario near Toronto, Ontario. The geological conditions at the site of the tunnel were reported by Lo and Lukajic (1984) and are summarized in the following paragraph.

The tunnel has a D-shaped cross section with a design span of 8 m excavated using the drill and blast method using full face advance. The actual excavated tunnel opening is 9 m in span and 925 m in length. The overburden soil at the site varies in thickness from approximately 21 m to 36 m and consists of a surficial silty clay layer overlying glacial till over bedrock. The first 8 m of the rock is a dark brown shaly limestone underlain by grey limestone of the Lindsay Formation. The grey limestone has 100% recovery with rock quality designation (RQD) ranging from 93% to 100%. The results of uniaxial compression tests showed a linear stress–strain relationship up to half of the failure stress with average uniaxial strength of 110 MPa. The average ratio of the horizontal to vertical modulus E_h/E_v was found to be about 1.2.

To analyze the tunnel behaviour, a method of estimating the properties of the rock mass is required. The system proposed by Hoek and Brown (1980, 1997) is one of the most widely accepted means of assessing rock mass properties and is used in the present study. Given the field description and quality of the rock, a geological strength index (GSI) of 80 is selected, representing a massive in situ rock with few widely spaced discontinuities. Based on this system, the uniaxial compressive strength of the rock mass is estimated to be 80 MPa. The equivalent Mohr–Coulomb parameters are obtained by fitting an average linear relationship to the Hoek–Brown failure envelope (Hoek et al. 1995, 2002). The uniaxial compressive strength of the rock mass, σ_{cm}, is defined as $\sigma_{cm} = (2c \cos \phi)/(1 - \sin \phi)$, where c and ϕ are the effective cohesion and angle of internal friction, respectively. The rock parameters used for the analysis of the Darlington intake tunnel are shown in Table 1.

A series of in situ stress measurements (Lo and Lukajic 1984) were taken at the site using the hydraulic fracturing technique. The results indicated that the state of in situ stress is remarkably anisotropic. The ratio of the horizontal to vertical initial stresses was found to be 10 at the springline of the tunnel.

A field instrumentation program (Lo and Lukajic 1984) was carried out during the tunnel construction to verify the design assumptions and monitor the tunnel performance as the excavation progressed. The instrumentation program included the following:

(*i*) Multiple-point extensometers at three cross sections along the alignment of the tunnel. This facilitated the measurement of displacements at the crown and springlines.

Table 1. Rock mass parameters (data from Lo and Lukajic 1984).

Young's Modulus, E (GPa)	30
Poisson's ratio, ν	0.33
Compressive strength	
Intact rock, σ_{ci} (MPa)	110
Rock mass, σ_{cm} (MPa)	80
Tensile strength, σ_t (kPa)	4
Cohesion, c' (MPa)	20
Friction angle, ϕ' (°)	35
Dilation angle, ψ (°)	0
Initial stress ratio, K_o	10
Unit weight, γ (kN/m^3)	25

(*ii*) Convergence points near the stations where extensometers are installed. The measuring points were located 1 m below springline and 1 m below invert.

(*iii*) Stress-meter at one cross section to measure the radial and tangential stresses as the tunnel face was advanced.

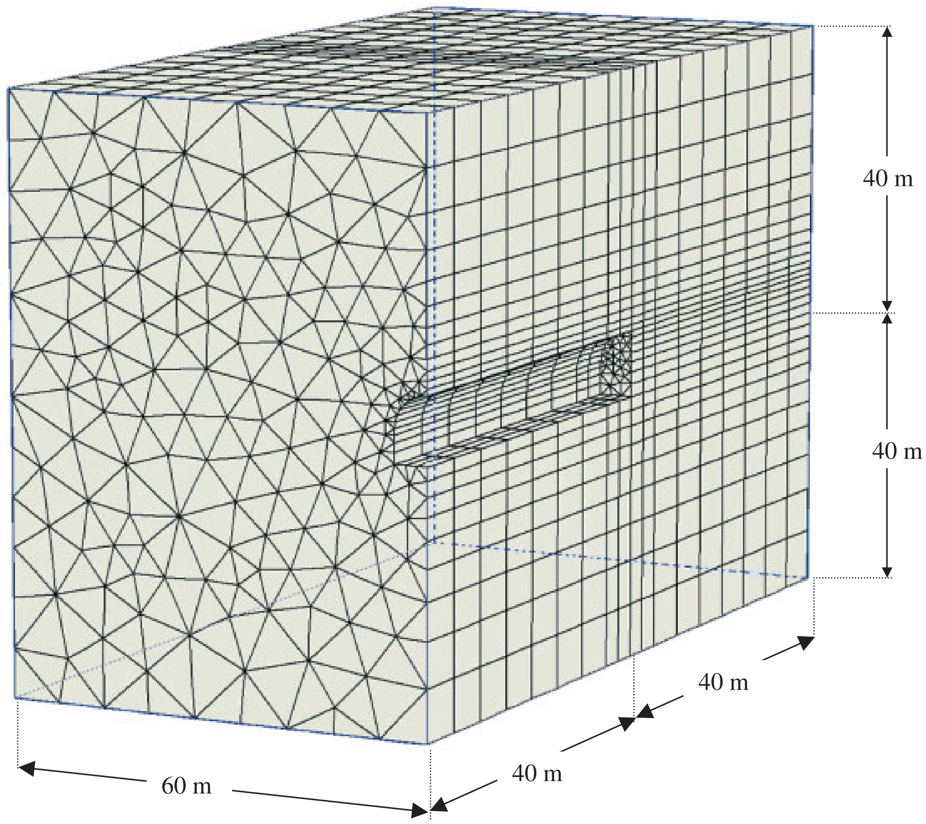
These field measurements were taken at locations away from the tunnel face where plane strain conditions prevail. These measurements will help to understand the two dimensional behaviour of D-shaped tunnels in similar geological conditions.

Method of analysis

The analyses were performed using the Plaxis 3D-Tunnel finite element program (Plaxis 2004) employing 15-noded wedge elements. The rock mass is assumed to behave as an elastic – perfectly plastic material where failure occurs because of overstressing of the rock around the tunnel opening. For the analysis of the Darlington tunnel (rock mass parameters are given in Table 1), excavated in a strong rock mass (σ_{cm} = 80 MPa), tunnel excavation may result in the development of a minimum or no plasticity in the close vicinity of the tunnel and therefore, the Mohr–Coulomb criterion would define the onset of compressive or tensile failure. For tunnels excavated in weaker rocks, a plastic zone will generally develop around the tunnel, and the onset of plastic failure of the rock mass is defined by the Mohr–Coulomb criterion. Consideration of rock mass structure (joints, bedding planes, faults, etc.) is beyond the scope of this study. For the nonlinear finite element analyses, material is removed in several stages so that the final stress data is appropriate to the sequence of excavation that would be performed.

A parametric study was conducted (Meguid et al. 2002) to examine the influence of the rigid boundary locations on the finite element results for tunnels constructed under different in situ stress conditions. It was found that the minimum distance to the lateral boundaries of the mesh, which would not significantly influence the predicted displacement of the tunnel, depends on the in situ stress to rock mass strength ratio. In the present study, a distance of 60 m (6.5D) from the tunnel centerline in the x direction and 40 m (4.5D) in the z direction was selected to minimize the boundary effects for a wide range of rock mass strength to in situ stress ratios. Behind the tunnel face, a distance of 40 m was necessary to en-

Fig. 1. Finite element mesh used in the 3D analysis.



sure that plane strain conditions were reached based on the observed stress and displacement patterns.

The 3D finite element analysis was performed using 7020 fifteen-noded isoparametric wedge elements with a total of 19 991 nodes arranged as shown in Fig. 1. Nodes along the vertical boundaries of the mesh may translate freely along the boundaries but are fixed against displacements normal to these boundaries. The nodes at the base are fixed against displacements in both directions.

To investigate the effects of rock mass strength reduction on the stability of tunnels constructed in anisotropic rock mass conditions, the Darlington tunnel geometry and material properties are adopted, however, the rock mass strength has been incrementally reduced using the *phi-c reduction* method. The strength reduction is established by reducing the cohesion and tangent of the friction angle in the same proportion:

$$c/c_{\mathrm{r}} = \tan\phi/\tan\phi_{\mathrm{r}} = \textit{strength reduction ratio}$$

where c and ϕ are the input strength parameters for the Mohr–Coulomb failure criterion and c_{r} and ϕ_{r} are reduced strength parameters that are just large enough to maintain equilibrium. The reduction of strength parameters is controlled by the strength reduction ratio, which is increased in a step-by-step procedure (by reducing c_{r} and ϕ_{r}) until failure occurs.

Based on this approach, the rock mass strength was incrementally reduced allowing the investigation of tunnel stability for a wide range of rock mass strength to in situ stress ratios.

The analysis of Darlington tunnel

The rock mass response to the excavation of the Darlington intake tunnel was analyzed. The induced displacements and stresses are examined in this section and compared to the field measurements.

Displacements

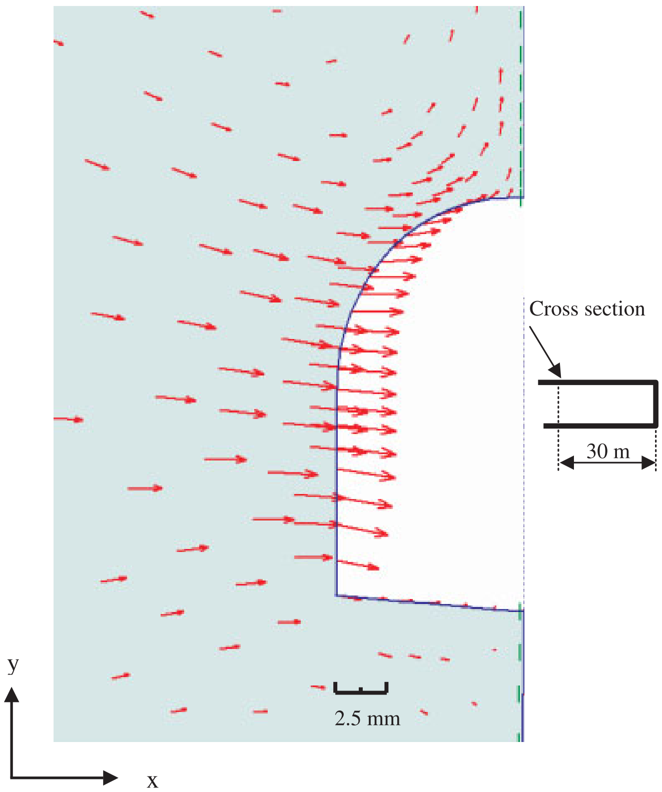
Displacement vectors surrounding the tunnel circumference at a plane strain cross section are shown in Fig. 2. The deformation pattern is dominated by high inward horizontal displacement at the tunnel walls and small upward displacements at the crown and invert. The directions and relative magnitudes of these displacements are consistent with those observed at the Darlington tunnel and other case histories in southern Ontario (e.g., Lo et al. 1984 and Lo 1989).

The distribution of radial displacement at the tunnel circumference with distance from the tunnel wall is shown in Fig. 3. The recorded displacements at six measuring points around the tunnel circumference are also plotted for comparison purposes. The distribution indicates an inward displacement at the springline and upward displacement at the crown, which decreases with distance from the tunnel wall. The stress redistribution zone extended to a distance of about 45 m (5D) around the tunnel circumference. Displacement results agreed well with the field measurements.

Stresses

The distributions of radial and tangential stresses at the tunnel crown with distance from the tunnel wall are shown in Fig. 4. A compressive stress concentration of 28 MPa in

Fig. 2. Displacement vectors at a cross section of the Darlington tunnel.



the tangential (horizontal) direction was calculated near the tunnel wall. This maximum compressive stress is well below the estimated compressive strength of the rock mass (80 MPa). The stresses in the radial direction were zero at the tunnel wall and increased to 3 MPa at a distance of 2 m and decreased to the overburden pressure (1 MPa) at a distance of 40 m from the tunnel wall.

The distributions of tangential and radial stresses at the tunnel springline are shown in Fig. 5. The tangential (vertical) stress is tensile within a distance of about 5 m from the tunnel wall and changes to compressive stress of 2 MPa (twice the overburden pressure) at a distance of 10 m from the tunnel wall. The tangential stress gradually decreases from 2 MPa to the initial vertical stress (1 MPa) at greater distance. The radial (horizontal) stress is zero at the wall and gradually increases to approach the initial in situ horizontal stress (10 MPa). The calculated stresses are in agreement with the measured stresses at the six locations surrounding the tunnel circumference as shown in Fig. 5.

Stability analyses of D-shaped tunnels

The influence of rock mass strength reduction on the stability of D-shaped tunnels is investigated in this section by examining the induced displacements at four different locations, namely, the face centre, springline, crown, and invert. The results are presented for initial stress ratio, K_o, values of 5, 10, 15, and 20 covering a wide range of anisotropic in situ stress and rock mass strength conditions.

The face centre

A plot of tunnel strain at the face centre against the ratio of rock mass strength to the overburden pressure is shown in Fig. 6. In this context, strain is defined as the percentage ratio of tunnel displacement to tunnel radius. Strains increase asymptotically when the ratio of rock mass strength to the overburden pressure falls below 0.5. This indicates the onset of severe face instability, and without adequate support the tunnel face would collapse.

Based on the relationship shown in Fig. 6, an elastic line representing the response of the rock mass in the elastic range is established for $K_o = 10$. The line can be obtained by extending the linear part of the curve towards the y axis. For tunnelling in weak rock masses, however, a plastic zone will generally develop around the tunnel, and the response between the elastic state and onset of failure is nonlinear as shown in Fig. 6. A design line can then be suggested for different K_o values to limit the strains to within tolerable limits. The suggested design line in this study is based on an additional 50% of the elastic strain. Strains between the limits bounded by the elastic and design lines are considered to be acceptable and will not cause instability at the tunnel face. For the case of $K_o = 10$, the maximum elastic strain, represented by the elastic line, was found to be 0.10% indicating a safe strain range of 0.10–0.15%. These results provide a preliminary assessment for the face stability of D-shaped tunnels excavated in rock masses subjected to anisotropic in situ stress conditions.

The springline

The induced strains at the tunnel springline for different rock mass strength and K_o values are shown in Fig. 7. For rock mass strength to overburden pressure greater than 1, strains linearly decreased as rock mass strength increased for the investigated range of K_o values. It was found that strains increased asymptotically when the ratio of rock mass strength to overburden pressure fell below 0.5. At this ratio severe instability and possibly failure is expected at the springline unless immediate tunnel support is provided.

The elastic and suggested design lines were established for $K_o = 10$ as shown in Fig. 7. The acceptable strain was found to range from 0.14% to 0.21%. These values represent the maximum elastic strain and the suggested strain range that ensure the stability of the springline. For the Darlington tunnel ($K_o = 10$ and the rock mass strength to overburden pressure ratio is greater than 8), the measured strain at the springline was found to be 0.05%, which is well below the maximum elastic strain, indicating that the tunnel is responding elastically and no stability problems are expected at the tunnel springline.

The crown and invert

The effect of rock mass strength reduction on the induced displacements at the tunnel crown is shown in Fig. 8. Displacements changed from outward to inward as the ratio of rock mass strength to overburden pressure decreased. Strains increased asymptotically when the ratio of rock mass strength to overburden pressure fell below 0.5 and increased significantly as the value of K_o increased from 5 to 20. For the investigated range of K_o, instability was observed to correspond to inward radial displacements. The elastic and sug-

Fig. 3. Radial displacements at the circumference of the Darlington tunnel.

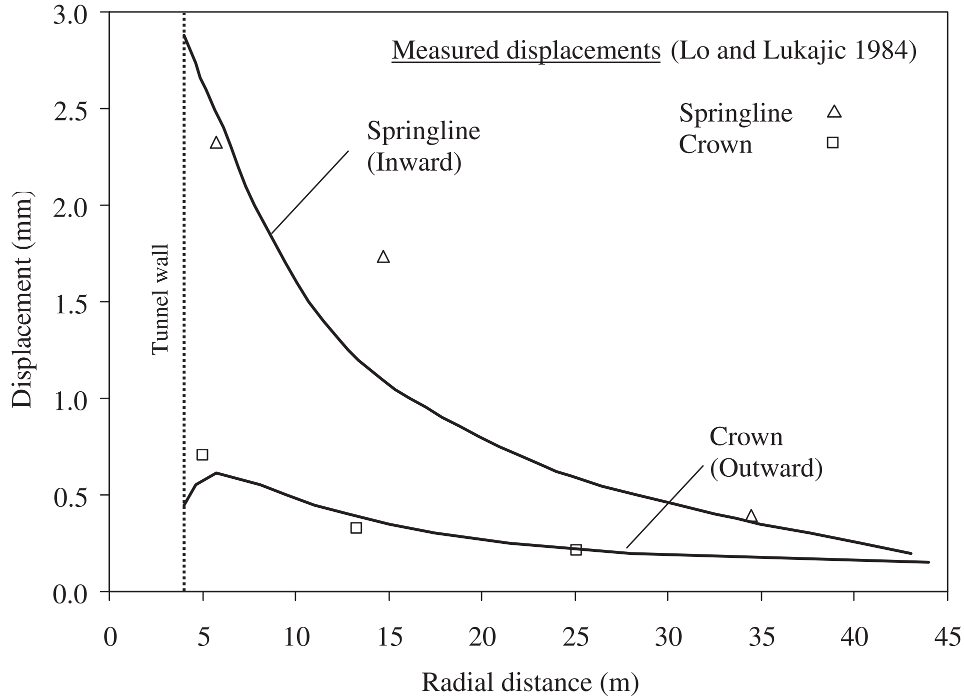
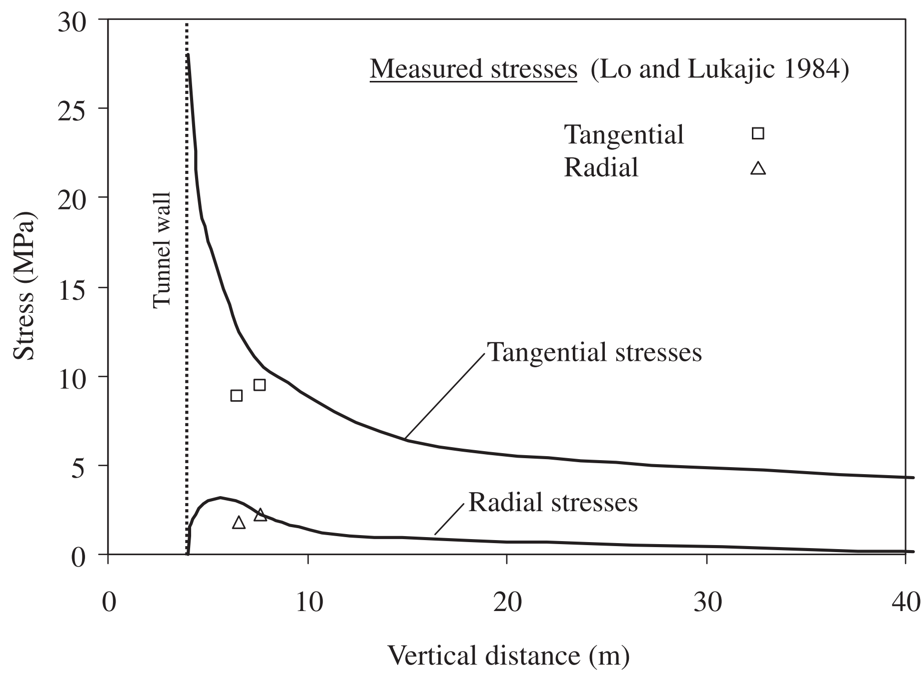


Fig. 4. Induced stresses at the tunnel crown due to tunnel excavation.



gested design lines were established for a K_o value of 10 as shown in Fig. 8. The strain was found to range between 0.07% and –0.04% at the tunnel crown. The measured strain at the crown of the Darlington tunnel was 0.02%, which fits between the elastic and the suggested maximum strain range, indicating that no stability problems are expected at the tunnel crown under the existing in situ stress conditions.

At the tunnel invert, the relationship between percent strain and rock mass strength to overburden pressure ratio is shown in Fig. 9. Higher strains were generally observed compared with other examined locations around the tunnel opening. The elastic and suggested design lines were established, and the acceptable strain at the tunnel crown was found to range from 0.35% to 0.53%.

Discussion

Based on the previous results, the design of D-shaped tunnels constructed in anisotropic stress conditions should satisfy the criterion of limiting the induced displacements to

Fig. 5. Induced stresses at the tunnel springline due to tunnel excavation.

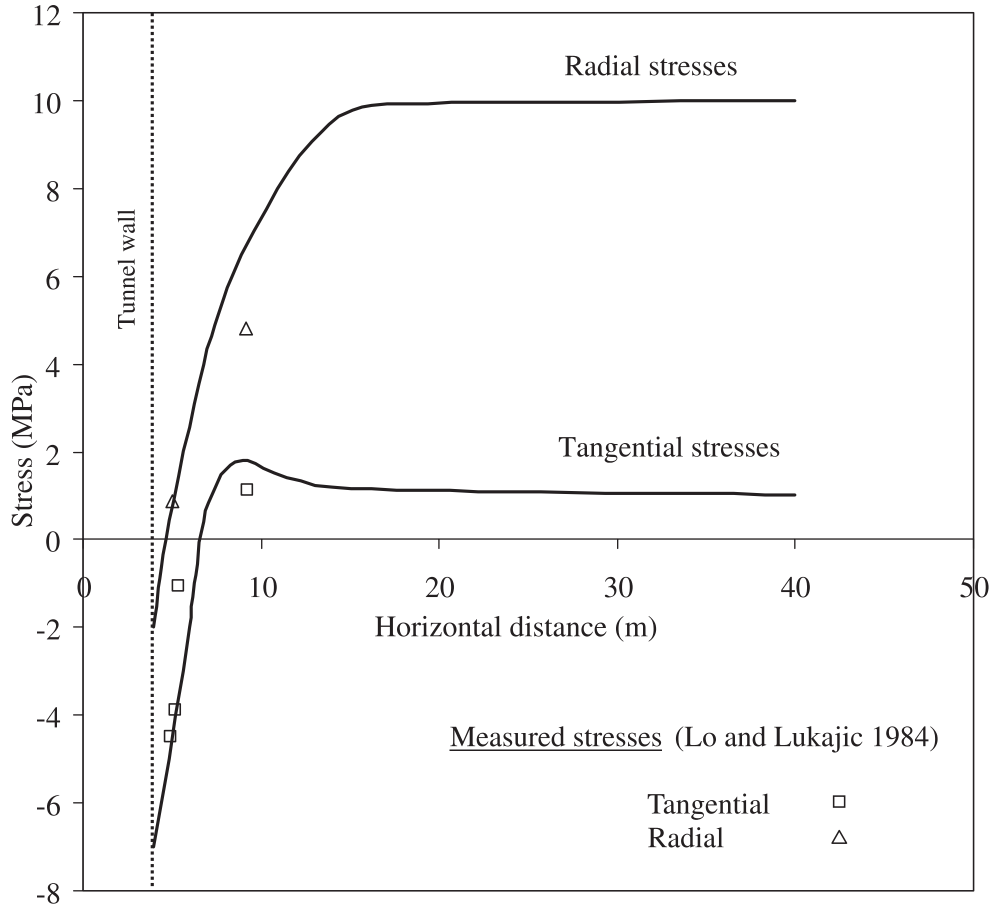


Fig. 6. The effect of rock mass strength reduction on the displacement at the face centre.

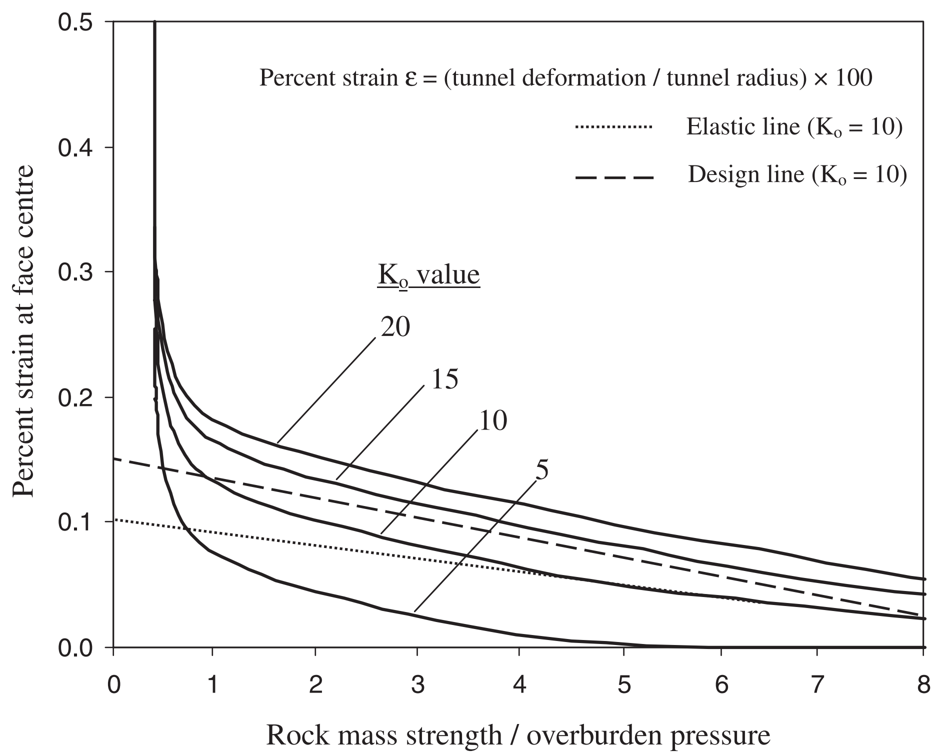


Fig. 7. The effect of rock mass strength reduction on the displacement at the tunnel springline.

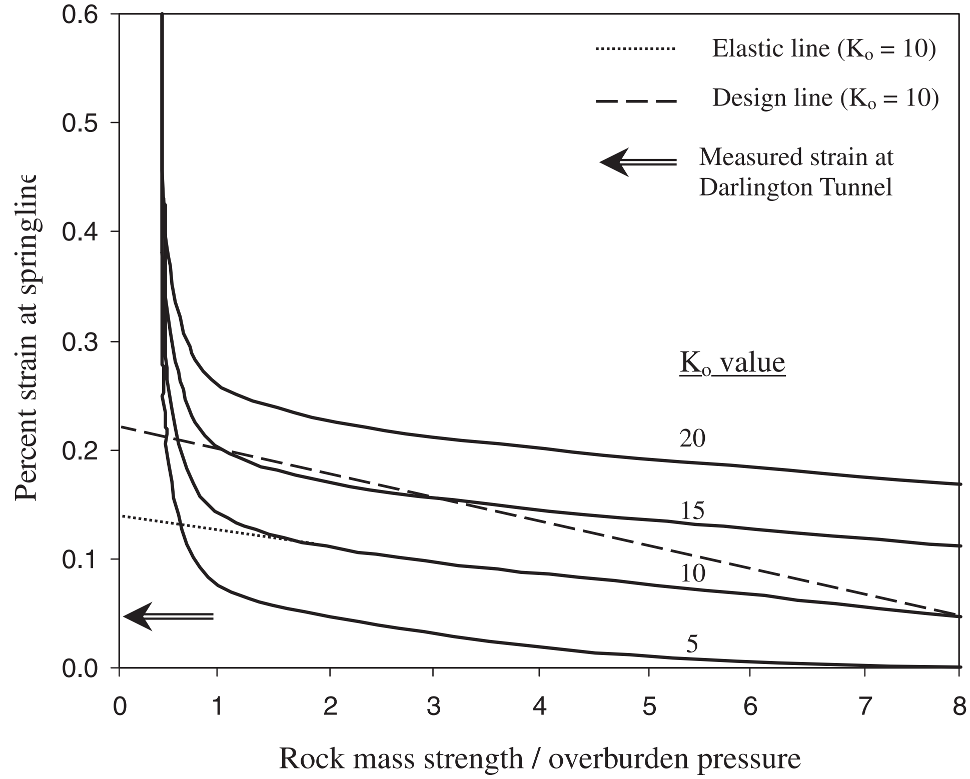
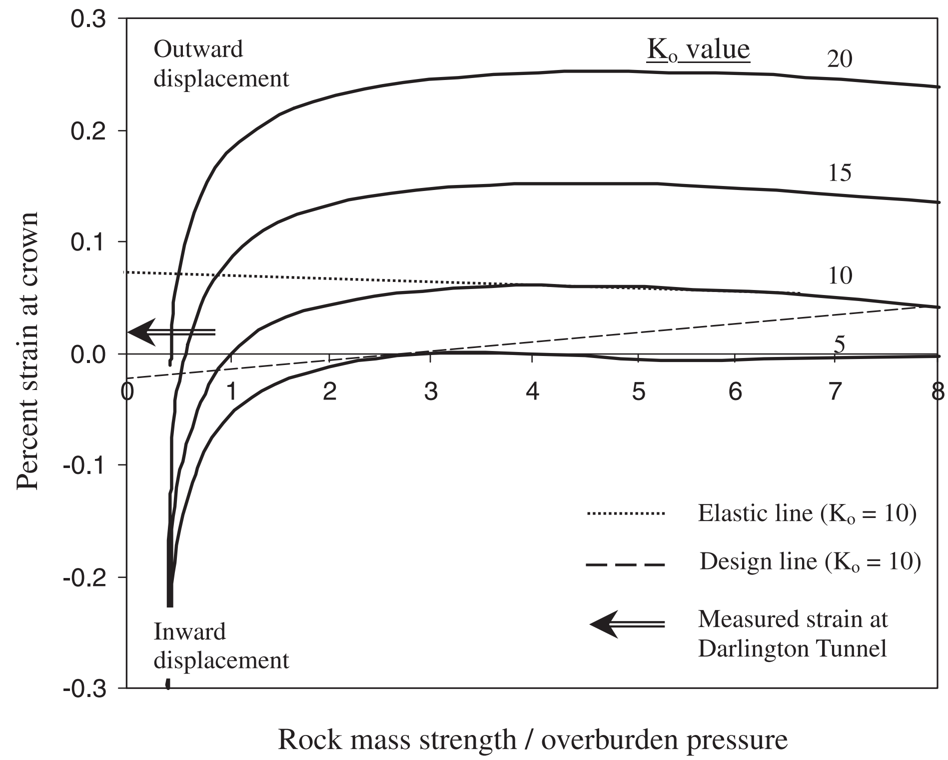


Fig. 8. The effect of rock mass strength reduction on the displacement at the tunnel crown.



within tolerable limits. These limits will vary depending on the location of the point of interest within the tunnel circumference. The elastic and suggested design strains are summarized in Table 2 for the case of K_o = 10. By inspecting the absolute values of strain at the investigated locations, it was observed that the allowable strain at the tunnel crown is ex-

Fig. 9. The effect of rock mass strength reduction on the displacement at the tunnel invert.

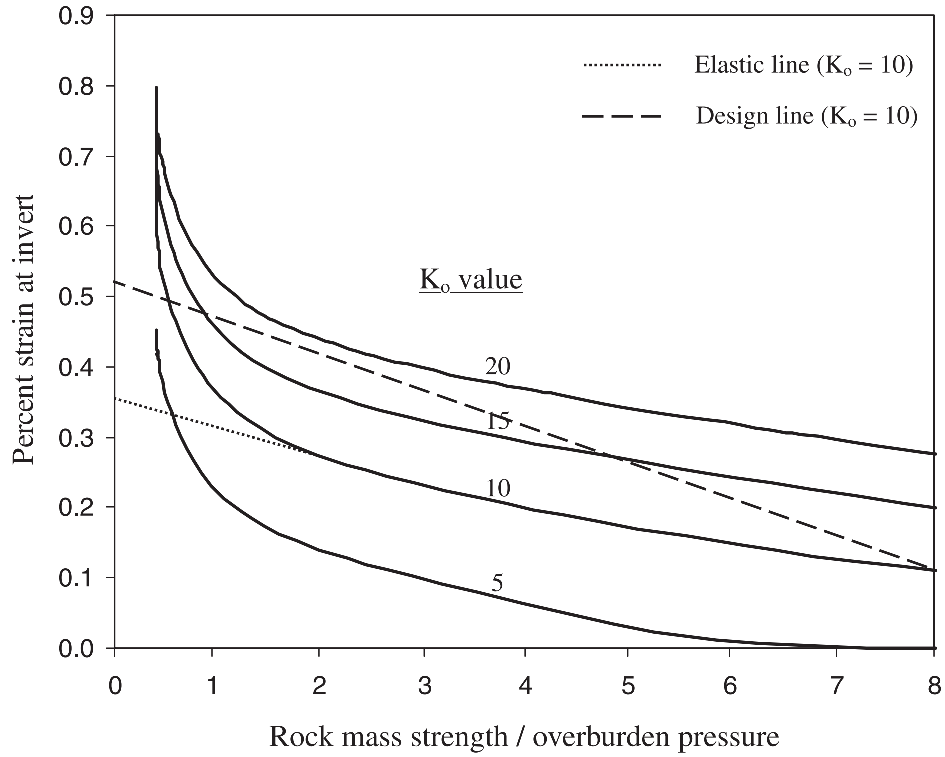


Table 2. Suggested strain range at different locations around the opening of D-shaped tunnels for K_o = 10.

	Face centre	Springline	Crown	Invert
Elastic strain (%)	0.10	0.14	0.07	0.35
1.5 Elastic strain (%)	0.15	0.21	–0.04	0.53

Note: Percent strain = (tunnel displacement / tunnel radius) × 100.

pected to be reached before the springline, face centre, and invert, which means that stability at the tunnel crown will govern the design of such D-shaped tunnels.

On the basis of the preceding discussion, for tunnels constructed in weak rock mass under anisotropic stress conditions, a supporting system has to be considered at the tunnel crown if the tunnel strain is expected to exceed the suggested range to avoid instability of the excavated sections. At the tunnel face, instability, in the form of excessive displacement and rock squeezing will occur, if the limiting value of strain is reached. Similar observations can be made for the tunnel springline and invert, however, since the allowable strain range is higher compared to the crown and face centre, the tolerable displacement is, therefore, greater. The analysis suggests that unless these ranges of suggested strain are exceeded, the circumference of the tunnel opening will not encounter problems during construction.

Summary and conclusions

The behaviour of the Darlington D-shaped tunnel constructed in rock mass subjected to high in situ horizontal stresses was investigated. The analysis allowed the computation of the complete stresses and deformation patterns around the tunnel and at the face. The calculated stresses and displacements were found to be in good agreement with the field measurements taken at different locations across the tunnel.

The effect of rock mass strength reduction on the stability of D-shaped tunnels constructed in anisotropic rock mass conditions was investigated. Rock mass strength reduction was achieved using the phi-c reduction method where the cohesion and tangent of the friction angle are reduced in the same proportion.

A design line was suggested to limit the induced strains to within tolerable limits. The suggested design line in this study is based on an additional 50% of the elastic strain. The analyses reported herein suggest that strains between the limits bounded by the elastic and design lines will not cause instability. Field verification of the proposed design limits would be appropriate.

Acknowledgements

This research is supported by the Natural Sciences and Engineering Research Council of Canada (NSERC). The finite element software used in this study was made available by a McGill University Research Grant for the first author.

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