33rd Annual General Conference of the Canadian Society for Civil Engineering
33e Congrès général annuel de la Société canadienne de génie civil

Toronto, Ontario, Canada
June 2-4, 2005 / *2-4 juin 2005*

ON THE FACE STABILITY OF TUNNELS IN WEAK ROCKS

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ABSTRACT: This paper investigates the near-face stability of tunnels excavated in weak shaly rocks. The effects of in situ stresses on the excavation induced stresses and displacements around the tunnel opening are evaluated. Results are validated by comparing the calculated response with the field measurements taken during the construction of the Darlington Tunnel near Toronto, Ontario. The effect of rock mass strength reduction on the tunnel deformation, face stability, and distribution of stresses at the tunnel circumference is examined for different in situ stress conditions.

1. INTRODUCTION

Several researchers (Lo *et al.* 1992, Hoek and Marinos 2000, Hoek 2001) investigated the stability of tunnels is 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 response of large tunnels excavated in rock mass subjected to anisotropic in situ stress conditions is examined. The case of Darlington intake tunnel is analyzed using three-dimensional elasto-plastic finite element analysis. The resulted 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 provide an example for the actual response of the rock mass under given in situ stresses. A detailed three-dimensional 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 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.

2. DARLINGTON TUNNEL

The tunnel investigated is a nuclear-powered generating station located on the shore of Lake Ontario near Toronto and was constructed in 1982. 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 drill and blast method using full face advance. The actual excavated tunnel opening was found to be 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 surficial silty clay layer overlying glacial till over bedrock. The first 8 m of the rock is a dark brown shaly limestone followed by grey limestone of the Lindsay Formation. The grey limestone has 100% recovery with RQD ranging from 93% to 100%. The results of uniaxial compression tests showed 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.

In order 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. Based on this system, the uniaxial compressive strength of the rock mass is related to the Mohr-Coulomb parameters by the relationship (Hoek *et al.* 1995)

$$\sigma_{cm} = \frac{2c\cos\phi}{1-\sin\phi}$$

where, c and ϕ are the cohesion and angle of internal friction respectively. The rock parameters used for the analysis of Darlington Intake Tunnel is shown in Table 1.

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

Table 1. Rock Mass Parameters (Lo and Lukajic 1984)

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 progresses. These field measurements were taken at locations away from the tunnel face where plane strain conditions prevail.

3. ANALYSIS DETAILS AND RESULTS

The analyses were performed using Plaxis 3D-Tunnel finite element program employing 15-noded wedge elements. The onset of plastic failure of the rock mass is defined by the Mohr-Coulomb criterion. Material was removed in several stages so that, for the non-linear analyses, 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 ensure 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 19991 nodes. 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.

In order to investigate the effects of rock mass strength reduction on the stability of tunnels constructed in anisotropic rock mass conditions, the *phi-c reduction* method was used where the cohesion and tangent of the friction angle are reduced in the same proportion:

$$\frac{c}{c_r} = \frac{\tan\phi}{\tan\phi_r} = strength\,reduction\ ratio$$

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

3.1 Displacements

The distribution of radial displacement at the tunnel circumference with distance from the tunnel wall is shown in Figure 1. The recorded displacements at six measuring points around the tunnel circumference are also plotted for comparison purpose. The distribution indicates an inward displacement at the springline and upward displacement at the crown that decrease 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 well agreed with the field measurements.

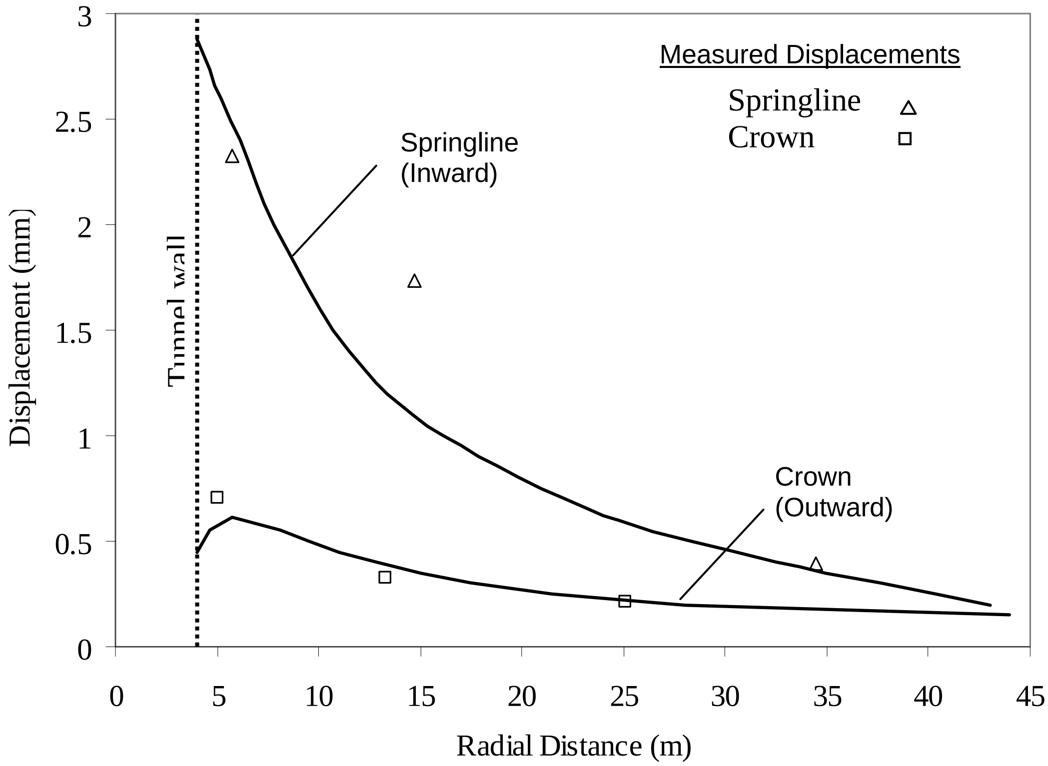

Figure 1. Radial Displacements at the Circumference of Darlington Intake Tunnel

3.2 Stresses

The distributions of radial and tangential stresses at the tunnel crown with distance from the tunnel wall are shown in Figure 2. Compressive stress concentration of 28 MPa in the tangential (horizontal) direction was calculated near the tunnel wall. This maximum compressive stress is well below the compressive strength of the rock (80 MPa). The stresses in the radial direction are zero at the tunnel wall and increased to 3 MPa at a distance of 2 m and decreased to the vertical in situ stress (1 MPa) at a distance of 40 m from the tunnel wall. The calculated stresses are in agreement with the measured stresses at the six locations surrounding the tunnel circumference as shown in Figure 2.

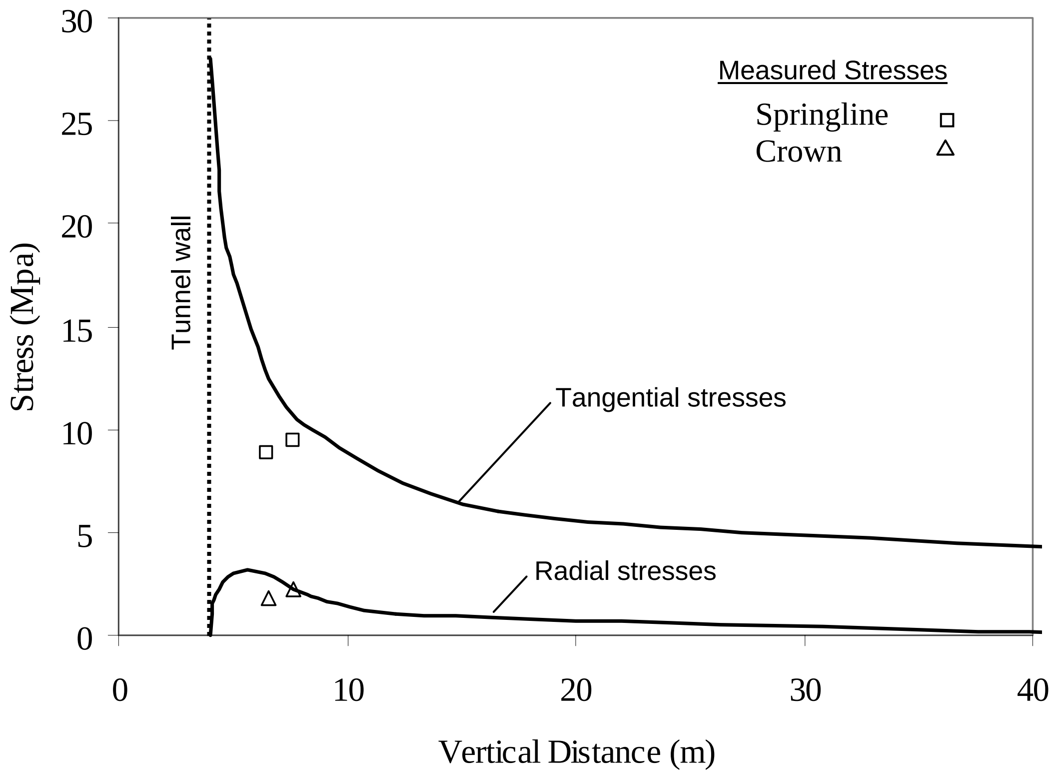

Figure 2. Induced stresses at the tunnel crown due to tunnel excavation

4. STABILITY ANALYSIS

The influence of rock mass strength reduction on the stability of tunnels is investigated in this section by examining the induced displacements at three different locations, namely, the face centre, springline, crown, and invert. The results are presented for K_o values of 1, 3, and 6 covering a wide range of anisotropic in situ stress and rock mass strength conditions.

4.1 The springline

The induced strains at the tunnel springline for different rock mass strength and K_o values are shown in Figure 3. In this context, strain is defined as the percentage ratio of tunnel displacement to tunnel radius. For rock mass strength to in situ stress 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 the vertical in situ stress falls below 0.5. At this ratio severe instability and possibly failure is expected at the springline unless immediate tunnel support is provided.

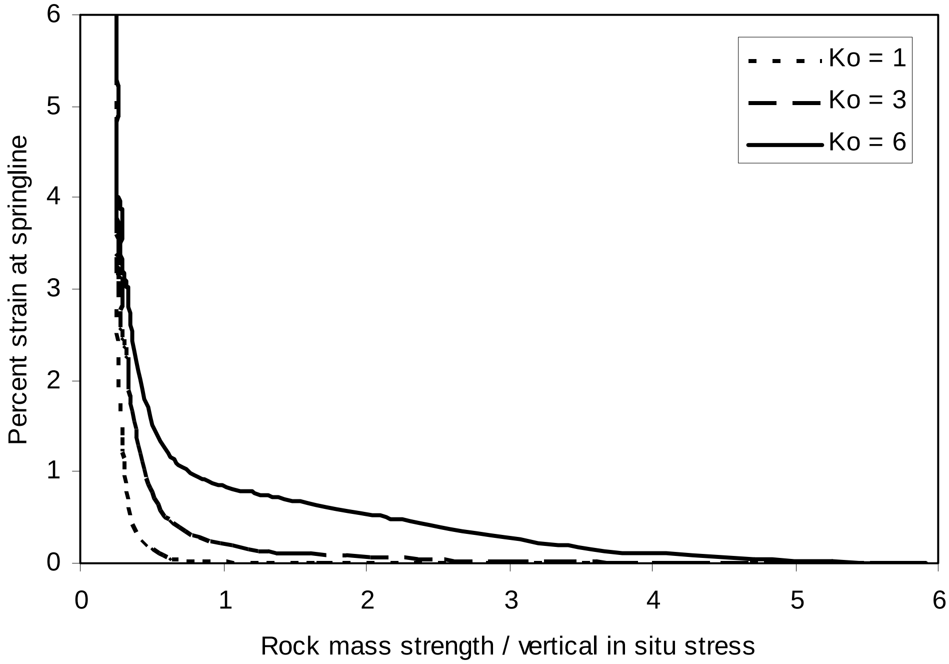

Figure 3. The Effect of shear strength reduction on the developed displacements at the springline

4.2 The face centre

A plot of tunnel strain at the face centre against the ratio of rock mass strength to the vertical in situ stress is shown in Figure 4. Strains increase asymptotically when the ratio of rock mass strength to the vertical in situ stress falls below 0.5. This indicates the onset of severe face instability, and without adequate support the tunnel face would collapse.

4.3 The crown and invert

The effect of rock mass strength reduction on the induced displacements at the tunnel crown is shown in Figure 5. Displacements changed from outward to inward as the ratio of rock mass strength to vertical in situ stress ranged between 1 and 3. Strains increased asymptotically when the ratio of rock mass strength to the vertical in situ stress falls below 0.5 and increased significantly as the value of K_o increased from 1 to 6. For the investigated range of K_o, instability was observed to correspond to inward radial displacements. The measured strain at the crown of the Darlington tunnel is 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.

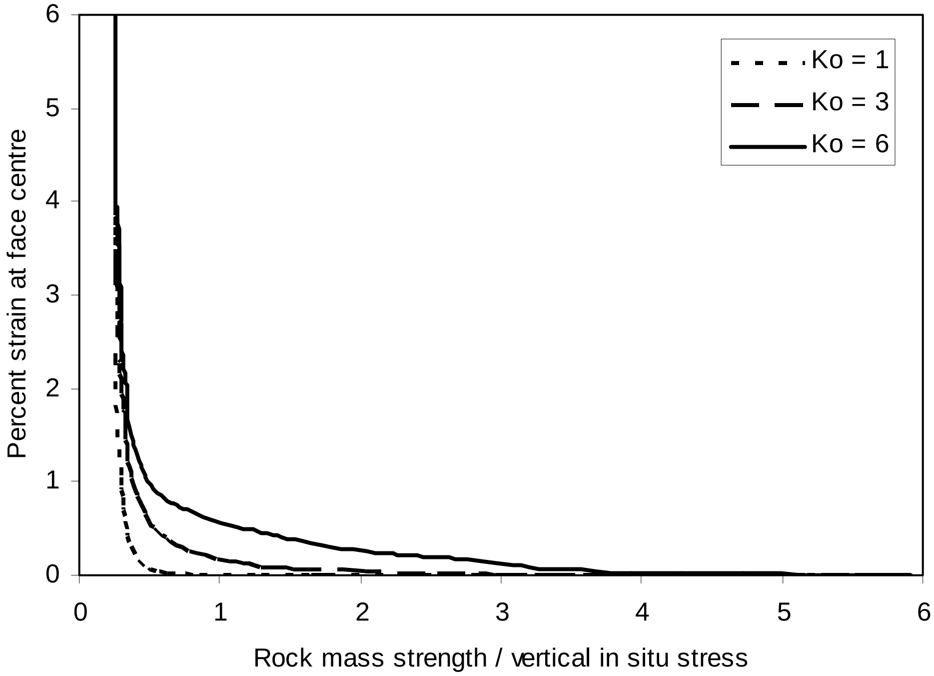

Figure 4. The Effect of shear strength reduction on the developed displacements at the face centre

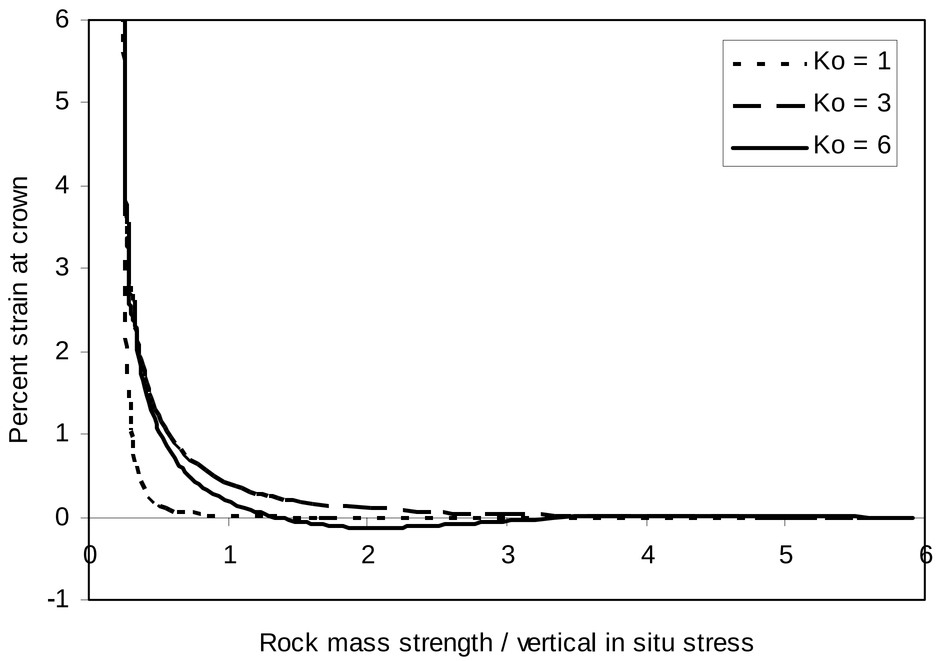

Figure 5. The Effect of shear strength reduction on the developed displacements at the crown

7. 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 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. Excessive strains and instability occurs when the ratio of rock mass strength to in situ stress falls below 0.5.

8. 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 McGill University Research Grant for the first author.

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