DESIGN OF ROCKBOLT AND SHOTCRETE FOR TUNNELS IN JOINTED ROCK

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ABSTRACT
In this paper, the use of rockbolts and shotcrete for tunnels excavated through jointed rock is modelled using the distinct element method. The geometry and mechanical properties of joints have a considerable effect on the stresses and deformations developed in the rockbolts and shotcrete. The effect of various parameters pertaining to joints including joint spacing and orientation, friction angle, cohesion and dilation angle of the joints on the design of fully grouted rockbolts and shotcrete are evaluated.

RÉSUMÉ
Dans ce papier, l'usage de rockbolts et shotcrete pour les tunnels creusé par le rocher de jointed est modelé l'utilisation de la méthode d'élément distincte. La géométrie et les propriétés mécaniques de joints ont un effet considérable sur les tensions et les déformations ont développé dans le rockbolts et shotcrete. L'effet de divers paramètres se rapportant aux joints y compris l'espacement de joint et à l'orientation, l'angle de friction, l'angle de cohésion et dilatation des joints sur la conception d'a mastiqué entièrement rockbolts et shotcrete est évalué.

1. INTRODUCTION

Rockbolts and shotcrete have been used for the support of underground excavations for many years. With the development of fibre reinforced shotcrete and the advent of new types of rockbolts, these supporting systems have become even more attractive. There are several approaches for the design of a support system consisting of rockbolts and shotcrete. In this paper a numerical approach based on the distinct element method is used and the effect of various parameters pertaining to the geometry of the joints and the mechanical properties of the discontinuities on the design of grouted rockbolts and shotcrete is investigated.

2. DESIGN APPROACHES

For the stability analysis of underground excavations and the design of support systems three methods comprising empirical, analytical and numerical methods can be used. The accuracy and reliability of these methods respectively increase from empirical methods towards numerical methods. Each of these methods has limitations and advantages. To obtain more reliable results, all of these three methods can be used simultaneously in the various phases of analysis and design.

2.1 Empirical Approaches

Empirical methods are based on the experience obtained from past projects. Rock mass classification procedures constitute the base for the empirical approaches. These methods are usually used in the preliminary stages of a tunnel design. In these methods, the values of stresses and displacements developed in the support system and the surrounding ground can not be evaluated.

2.2 Analytical Approaches

In the analytical approaches, the design of support system is based on theoretical models and some formulations developed for certain conditions. Analytical approaches can take into account the interaction between the rock mass and the dividing layer. In most of these formulations it is assumed that a circular cross section is excavated in a homogenous and isotropic medium with hydrostatic in-situ stresses. In a jointed rock, an equivalent continuum medium should be used in the analysis. In the case of grouted rockbolts, this approach only observes the analysis from a qualitative point of view and takes into account the effect of rockbolts by improving the characteristics of the existing rock mass.

2.3 Numerical Approaches

Numerical methods for the stress analysis can be classified into two main categories: (1) Domain or differential methods, represented by the finite element, finite difference and distinct element methods. These methods solve the field equations by dividing the rock mass into elements or zones within which the governing equations are formally satisfied. The finite difference and the distinct element methods have the same basis in the solid mechanics. In the finite difference method, attention is focused on the continuum, although several discontinuity surfaces (slip lines) can also be modelled. Alternatively, in the block-jointed medium, the interaction between blocks is of primary concern, and the state of stress in the interior of the blocks is conveniently determined using the distinct elements. The finite element method is closely related to the finite difference method in that the interior of the problem domain must be discretized completely into separate elements. (2) Integral or boundary methods, represented by the several versions of the boundary element methods, construct solutions to the field equations using fundamental solutions to these equations and by applying some formal solutions from solid mechanics. In these methods only the surface of an excavation is used in the solution, and the interior of the problem domain is not represented explicitly (Brady 1992). The appropriate numerical methods for the various rock mass and joint-scale conditions are presented in Table 1.

Table 1. Appropriate computational method for various rock-mass and joint scale conditions (Brady 1992)

Rock-mass conditions	Rock material	
	Elastic	Elasto-Plastic
Massive rock	Boundary element	Finite element, Finite difference
Sparsely-jointed rock	Boundary element	Finite difference, Finite element
Closely jointed rock	Distinct element	Distinct element
Heavily jointed rock		Finite element, Finite difference

In this paper, closely jointed rock masses have been modelled using the distinct element method as an appropriate numerical approach. UDEC program has been used for conducting the analysis. The Universal Distinct Element Code (UDEC) is a two-dimensional numerical program based on the distinct element for discontinuum modelling. UDEC simulates the response of discontinuous media (such as a jointed rock mass) subjected to either static or dynamic loading.

3. ROCKBOLT AND SHOTCRETE DESIGN

3.1 Problem Statement

In this research, it is assumed that a 10m diameter circular tunnel is excavated 200m deep in sandstone containing two continuous joint sets deviated $\pm$45 from the vertical, with an average spacing of 1m and subjected to a hydrostatic in-situ pressure (k=1). A combination of grouted rockbolts and shotcrete is used for the support of the tunnel.

3.2 Constitutive Model of Intact Rock and Joints

To represent the behaviour of intact material, deformable blocks have been used with constitutive model of Mohr-Coulomb plasticity, which is considered suitable for underground excavations.

Joint area constant-Coulomb slip constitutive model has been used for joint modelling, which is considered appropriate for the joints and faults analysis in the general rock mechanics (e.g., underground excavations).

3.3 Rock and Joints Mechanical Properties

Mechanical properties of intact rock and joints are summarized in Table 2 (Goodman 1989, Rahn 1986, Stillborg 1986).

Table 2. Mechanical properties of intact rock and joints.

Properties	Values
Elastic modulus of intact rock (GPa)	19
Poisson's ratio of intact rock	0.2
Density of intact rock (kg/m³)	2600
Compressive strength of intact rock (MPa)	72
Tensile strength of intact rock (MPa)	5
Friction angle of intact rock (degree)	50
Cohesion of intact rock (MPa)	12.5
Friction angle of joints (degree)	30
Cohesion of joints (MPa)	0.2
Dilation angle of joints (degree)	0

The Constant Normal Stiffness (CNS) technique is more appropriate than the Constant Normal Load (CNL) technique for the stability analysis of the roof of an excavation. The CNS technique could also be extended to model the behaviour of bolted joints, in view of the stabilisation of unstable roofs in underground excavations. In the analysis, the contribution of the surrounding rock mass stiffness is considered to be constant, while the normal stress continues to vary during deformation (Indraratna and Haque, 2000).

Therefore, appropriate values should be assigned to the normal and shear stiffnesses of rock joints. Values for normal and shear stiffnesses of rock joints typically range from roughly 10 to 100 MPa/m for joints with soft clay in-filling, to over 100Gpa/m, for tight joints in granite and basalt (Itasca Consulting Group, Inc. 2000).

Given the joint spacing and elastic modulus of intact rock, a normal stiffness of 38 GPa/m and a shear stiffness of 7.6 GPa/m have been used in the analysis.

3.4 Properties of Rockbolts and Shotcrete

For short term applications, the bolts are generally left ungrouted. For more permanent applications, the space between the bolt and the rock can be filled with cement or resin grout (Hoek et al., 2000).

Given the permanent application of the support system, fully grouted rockbolts are used. The properties of rockbolt and steel fibre reinforced shotcrete are summarized in Table 3.

Table 3. Rockbolts and shotcrete properties.

Properties	Values
Rockbolt diameter (mm)	25
Hole diameter (mm)	38
Elastic modulus of rockbolt (GPa)	200
Ultimate tensile capacity of rockbolt (kN)	200
Grout shear strength (MPa)	4
Grout shear modulus (GPa)	9
Grout shear stiffness (GN/m/m)	12
Grout cohesive strength (kN/m)	300
Density of shotcrete material (kg/m^3)	2500
Elastic modulus of shotcrete (GPa)	21
Poisson's ratio of shotcrete	0.15
Compressive yield strength of shotcrete (MPa)	35
Tensile yield strength of shotcrete (MPa)	20
Residual yield strength of shotcrete (MPa)	10

3.4.1 2D/3D Equivalence

Reducing 3D problems with regularly spaced structural elements to 2D problems involves averaging the effect in 3D over the distance between elements. According to Donovan et al. (1984) suggestion, linear scaling of material properties is a simple and convenient way of distributing the discrete effect of elements over the distance between elements in a regularly spaced pattern. The element spacing, S, can be used to scale the structural element properties. The scaled property is determined by dividing the actual property by S. For rockbolt elements, the following properties should be scaled: (1) elastic modulus of the rockbolt, (2) tensile yield strength of the rockbolt, (3) stiffness of the grout, and (4) cohesive strength of the grout. (Itasca Consulting Group, Inc., 2000)

3.4.2 Plane-Strain Conditions

The structural element formulation is a plane-stress formulation. If the element is representing a structure that is continuous in the direction perpendicular to the analysis plane (e.g., a shotcrete lining), the value specified for the elastic modulus should be divided by $(1-\upsilon^2)$ to account for the plane-strain conditions. (Itasca Consulting Group, Inc., 2000)

3.5 Preliminary Design Based on Q-system

The Q system has been used for the preliminary selection of the support system. The values of parameters used for the calculation of Q index are presented in Table 4. It has been assumed that three joint sets exist; two joint sets striking parallel to the tunnel axis and one perpendicular the tunnel axis. Based on the values presented in Table 4, the Q index is computed as 6.6. The tunnel is assumed to be a major road or railway tunnel with ESR=1. According to Barton and Girmstad (1993) recommendation, 3m long rockbolts regularly spaced at 2.2m plus fibre reinforced shotcrete with a thickness of 40 to 50mm is required to stabilize the tunnel.

Table 4. Q index calculations.

Item	Description	Value
Rock Quality (RQD)	Good	80
Joint sets (J_n)	Three joint sets	9
Joint roughness (J_r)	Slickensided, undulating	1.5
Joint alteration (J_a)	Slightly altered joint walls.	2
Joint water (J_w)	Dry excavation	1
Stress reduction (SRF)	Medium stress	1

3.6 Modelling Process

3.6.1 2D Model Boundaries

Boundaries are of two categories; real and artificial. Real boundaries exist in the physical object being modelled (e.g., a tunnel surface or the ground surface). Artificial boundaries do not exist in reality, but they should be introduced in order to constrain the number of elements. The model artificial boundaries should be far enough away from the region of study so that the model response is not influenced adversely. In general, for the analysis of a single underground excavation, boundaries should be located roughly five excavation diameters from the excavation periphery. (Itasca Consulting Group, Inc., 2000) Given the 10m diameter tunnel, a 60mx60m square has been chosen for 2D modelling of the rock mass.

3.6.2 Boundary Conditions

The boundary conditions in a numerical model consist of the values of field variables (e.g., stress, displacement) that are prescribed at the boundary of model. Given the depth of tunnel and the hydrostatic pressure condition, compressive stresses are applied to boundaries as shown in Figure 1. Gravity is applied in the negative y-direction to help identify loose blocks around the opening.

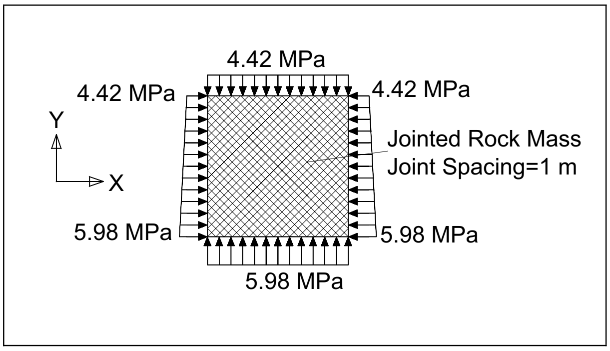

Figure 1. Boundary conditions on 2D model

3.6.3 Initial Equilibrium

The model should be at an initial force-equilibrium state before alterations can be performed. The boundary and initial conditions have been assigned so that the model is at equilibrium initially. However, it is necessary to calculate the initial equilibrium state under the given boundary and initial conditions. Therefore, after modeling the rock mass with the boundary conditions, the model has been solved to obtain the initial equilibrium.

3.6.4 Modeling Excavation Procedure and Support System

Before modeling the excavation, the displacements have been reset, so that only the change in displacements due to the excavation can be monitored. Then with fixing the bottom boundary from moving in y-direction, the excavation has been performed.

As initial support system, steel fibre reinforced shotcrete with a thickness of 50mm has been modeled in combination with 3m long fully grouted rockbolts, regularly spaced at 2m. The rockbolts and shotcrete have extended in a 300 degree arc and cover all but the invert of the tunnel as shown in Figure 2.

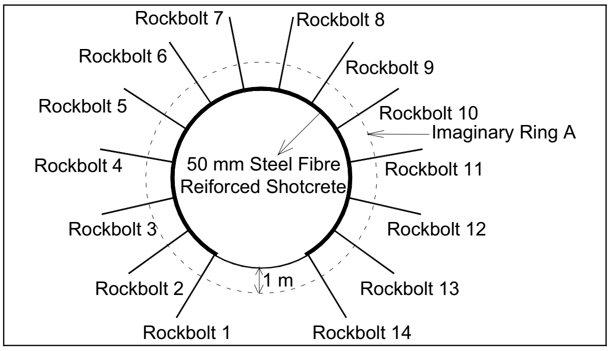

Figure 2. Support system around the tunnel

In this excavation, reinforced shotcrete with 50mm thickness is supposed as temporary support, which is activated after some convergence and relief of some part of the in-situ stress (Panet and Guenot 1982). Rockbolt is assumed as permanent support, which is installed after applying shotcrete. Then, the model is solved to obtain the final equilibrium condition with the above assumptions.

3.7 Analysis Results and Design Optimization

Figure 2 illustrates the location of the shotcrete and rockbolts around the tunnel periphery. In the following paragraphs, ring A is define as an imaginary circle with 1m distance from the excavated surface.

The maximum axial force in the rockbolts and the maximum shear force per unit length of the grout with their corresponding safety factors are summarized in Table 5. The maximum axial force and moment in shotcrete are 226.5kN/m and 3.502kN.m/m respectively, which are well tolerated by shotcrete. The maximum displacement of the blocks (in the roof) is 9.35mm and the average radial displacement of ring A is 7.22 mm.

Table 5. Axial force in rockbolts and shear force in grout.

Rockbolts Number	Axial Force in Rockbolt		Shear Force in Grout	
	Force Value (kN)	Safety Factor	Force Value (kN/m)	Safety Factor
Rockbolts 1&14	3.3	60.6	4.1	73.2
Rockbolts 2&13	94.3	2.1	106.9	2.8
Rockbolts 3&12	63.3	3.2	101.7	2.9
Rockbolts 4&11	75.9	2.6	126.7	2.4
Rockbolts 5&10	7.6	26.1	11.1	27.0
Rockbolts 6&9	17	11.8	21.1	14.2
Rockbolts 7&8	54.4	3.7	101.7	2.9

Appropriate safety factor for axial force of rockbolt and shear force of grout is 2-3. As shown in Table 5 rockbolts number 1, 5, 10, 14 are almost unnecessary and rockbolts number 6 and 9 are bearing just a little force. Therefore, unnecessary rockbolts are omitted and in critical points of roof and springlines rockbolts are installed more closely. The new design includes 4 rockbolts in the roof with 1m spacing and 3 rockbolts in each springline with 1.5m spacing. The spacing of rockbolts along the tunnel axis is assumed to be 2m. The location of rockbolts around the tunnel periphery is shown in Figure 3. All rockbolts are 3m long and fully grouted. The maximum axial forces in the rockbolts and the shear forces in the grout along with their safety factors in the revised analysis are included in Table 6.

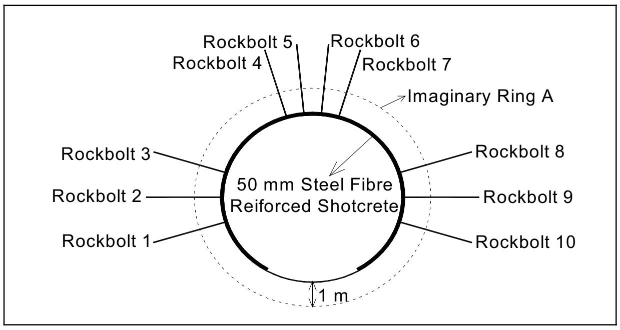

Figure 3. The location of rockbolts in the optimized design

Table 6. Axial force in rockbolts and shear force in grout in revised analysis.

Rockbolts Number	Axial Force in Rockbolt		Shear Force in Grout	
	Force Value (kN)	Safety Factor	Force Value (kN/m)	Safety Factor
Rockbolts 1&10	58.2	3.4	85.7	3.5
Rockbolts 2&9	73.4	2.7	96	3.1
Rockbolts 3&8	59.2	3.4	94.4	3.2
Rockbolts 4&7	63.4	3.2	105.8	2.8
Rockbolts 5&6	72.6	2.8	117.8	2.5

The maximum axial force and moment in shotcrete are 158.9kN/m and 3.2kN.m/m respectively. The maximum displacement of blocks (in the roof) is 9.5mm and the average radial displacement of ring A is 7.27mm. After decreasing the number of rockbolts, the safety factors are constrained between 2.3 and 3.7

4. EFFECT OF JOINT PROPERTIES

4.1 Joint Friction Angle

The friction angle of rock joints typically range from 10 to 20°, for joints with clay infilling, to 47°, for tight joints in basalt (Rahn, 1986). In order to investigate the effect of joint friction angle on the design of fully grouted rockbolts and shotcrete, a set of sensitivity analyses are performed for the joint friction angle varying between 10 and 50°. The values of other parameters are taken the same as those mentioned in Table 3. These analyses are divided into two groups:

(1) Pattern 1: Joint friction angle between 20 and 50°, 50mm thick shotcrete, 10 fully grouted rockbolts (Figure 3) of 3m length with 2m spacing along the tunnel axis.
(2) Pattern 2: Joint friction angle between 10 and 20°, 50mm thick shotcrete, 10 fully grouted rockbolts (Figure 3) of 4m length with 1m spacing along the tunnel axis.

In low friction angles rockbolts are at their yield point (axial force = 200kN) and the results cannot be compared. That is why the sensitivity analyses are divided into the above mentioned groups. The results are presented in Figures 4 to 6.

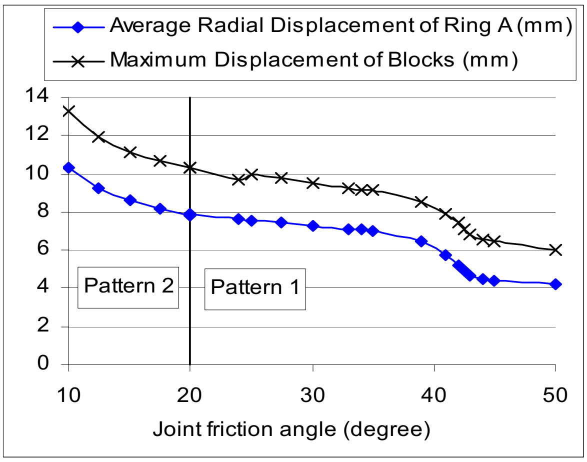

Figure 4. Effect of joint friction angle on displacements

In these figures three distinct regions can be identified. In the first region (10°<ϕ<20°) the behaviour of the rock mass is mostly governed by the strength parameters of the joints. In the second region (20°<ϕ<40°) the rock mass behaviour is a function of the strength and deformation parameters of the intact rock and the joints. In the third region (ϕ>40°) the slip between the joint does not occur. As a result, with a sudden change, the behaviour of rock mass is governed by the intact rock properties.

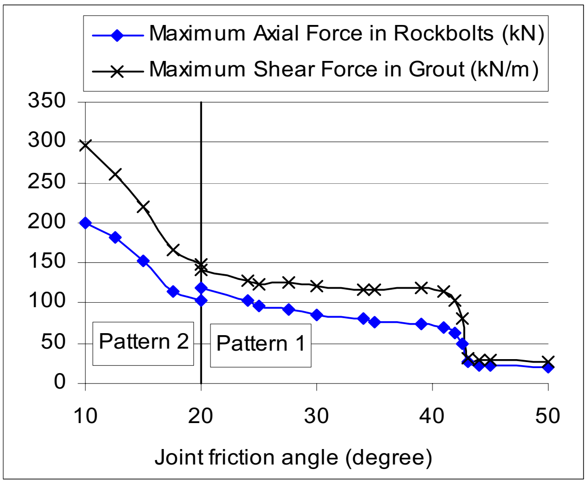

Figure 5. Effect of joint friction angle on rockbolts forces

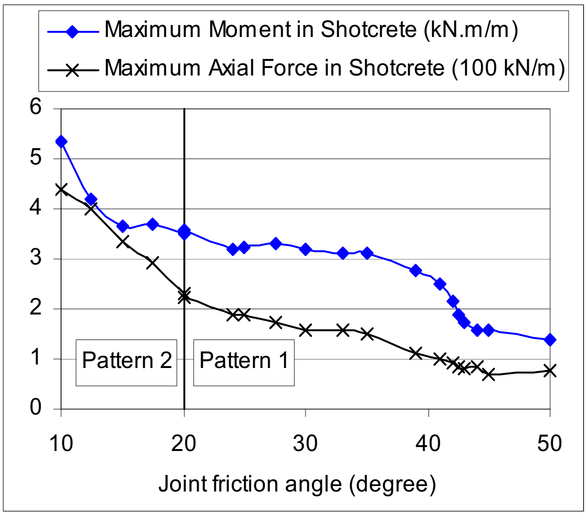

Figure 6. Effect of joint friction angle on shotcrete

4.2 Joint Cohesion

In order to investigate the effect of joint cohesion on the forces developed in the rockbolts and the shotcrete, the value of this parameter is changed between 0 and 5MPa while using ϕ = 17.5° . The values of other parameters are taken the same as those mentioned in Table 3. Rockbolts of 3m length with the pattern shown in Figure 3 are used. The spacing of the rockbolts along the tunnel axis is taken as 1.5m. The results are illustrated in Figures 7 to 9.

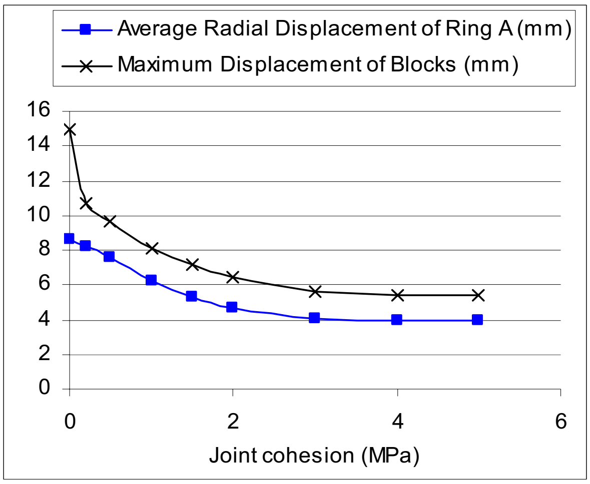

Figure 7. Effect of joint cohesion on displacements

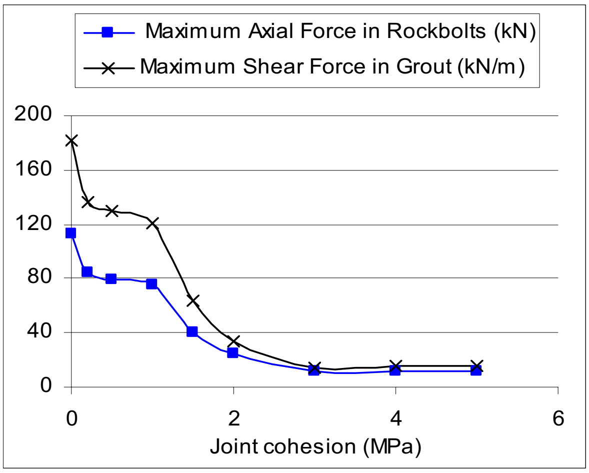

Figure 8. Effect of joint cohesion on rockbolts forces

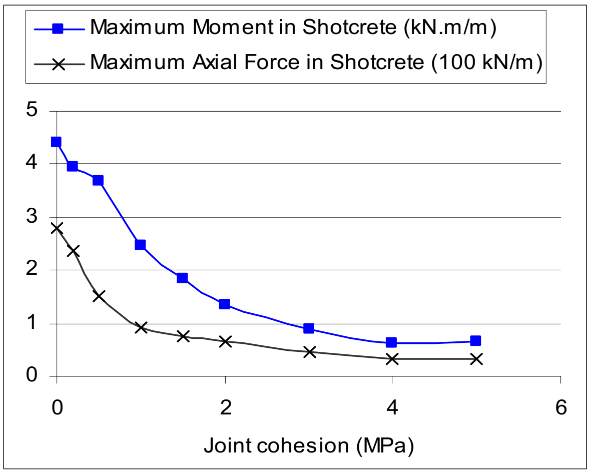

Figure 9. Effect of joint cohesion on shotcrete

Three regions can be identified in these figures. In the first region (C<0.2MPa), there is a sudden increase in the rock mass behaviour indexes with a small increase of the joint cohesion. Considering the fact that a small amount of confining pressure acts on the joints near the excavated surface, the cohesion of these joints is more significant. After a transition region, in the high values of cohesion (C>3MPa) the rock mass behaviour is governed by the intact rock properties (Third region).

4.3 Joint Dilation Angle

Joint dilation may occur at the onset of slip (non-elastic sliding) of the joint. Dilation is governed in the coulomb slip model by a specified dilation angle, ψ. A set of sensitivity analyses by using different values of joint dilation angle is run ($0\leq \psi \leq 20°$), where in all of them C=0MPa and ϕ =20° and other parameters are as mentioned in Table 3. The results are illustrated in Figures 10 and 12.

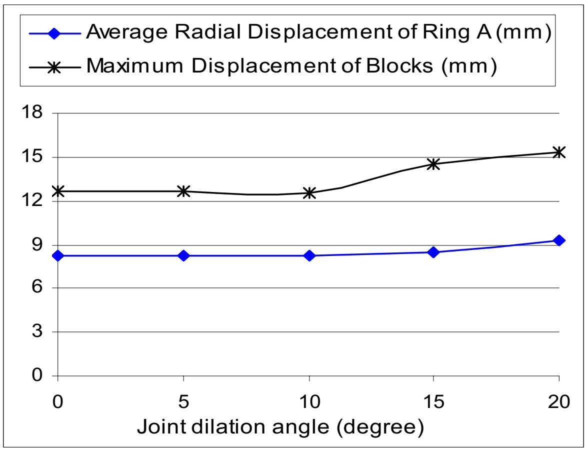

Figure 10. Effect of joint dilation angle on displacements

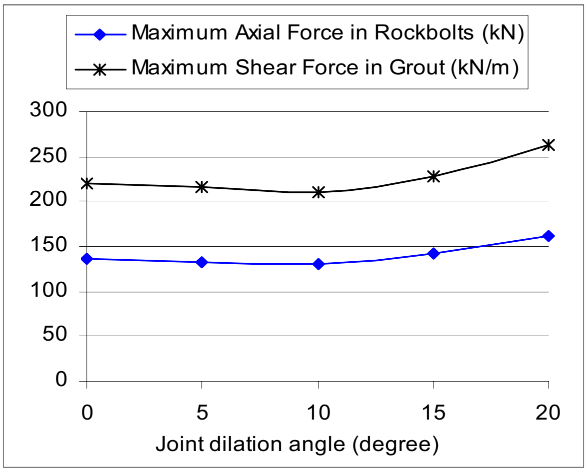

Figure 11.Effect of joint dilation angle on rockbolts forces

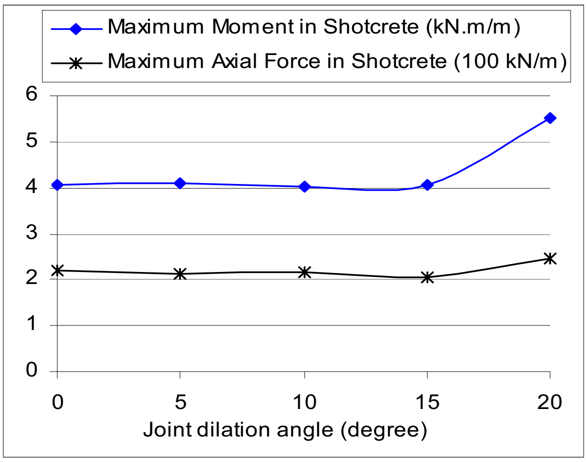

Figure 12.Effect of joint dilation angle on shotcrete

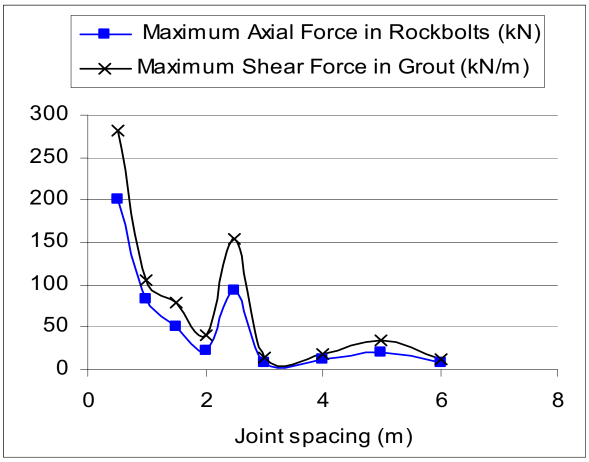

Figure 14. Effect of joint spacing on rockbolts forces

As shown in Figures 10 and 12, the rock mass displacements and the supporting system stresses are not sensitive to the increase of dilation angles up to 10°, while they increase with the increase of dilation angle from 10 to 20°. (In accordance with associated flow rule)

5. GEOMETRY OF JOINTS

5.1 Joint Spacing

In order to consider the effect of joint spacing on the design of rockbolts and shotcrete its value is changed between 0.5 to 6m. For the intact rock and joint mechanical properties the values mentioned in Table 3 are used. The results are illustrated in figures 13 to 15. It is shown that increasing the joint spacing causes less displacement in rock mass and less stresses in supporting system, expect in two cases, where joint spacing is 2.5 m and 5 m, in which the slipping wedge is of a considerable size.

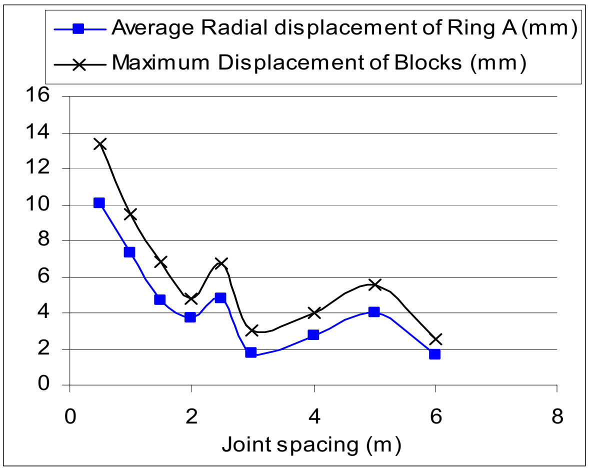

Figure 13. Effect of joint spacing on displacements

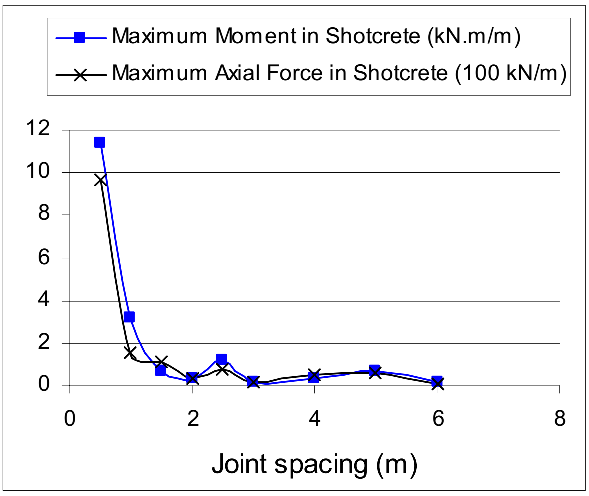

Figure 15. Effect of joint spacing on shotcrete

5.2 Joint Orientation

Using various combinations of two joint sets each dipping 0°, 30°, 45°, 60° or 90° (a total of 16 combinations), the effect of joint orientation on the support system and the surrounding rock mass behaviour is investigated. The results are summarized in Table 7 and Figure 16.

It is shown that the joint orientation has a significant effect on the design of rockbolts and shotcrete.

It is shown in Figure 16 that with constant orientation angle of one joint set, the average radial displacement of ring A increases with the increase of the angle between two joint sets up to 75° and then decreases with the bigger angles.

Table 7. Effect of joint orientation on rockbolts safety factor and shotcrete moment and axial force

Joint Orientations	Rocbolt Axial Force Safety Factor	Grout Shear Safety Factor	Maximum Axial Force in Shotcrete (kN/m)	Maximum moment in shotcrete (kN.m/m)
0° & 30°	1.80	1.70	2488.0	5.34
0° & 45°	1.75	1.78	4510.0**	8.70
0° & 60°	2.10	2.00	2509.0	5.70
0° & 90°	3.85	3.56	1861.0	1.97
30° & 45°	2.80	2.60	2449.0	6.06
30° & 60°	1.60	1.50	2598.0	3.08
30° & 90°	2.10	2.14	839.0	2.31
30° & 120°	1.50	1.40	3009.0	2.83
30° & 135°	Yield	0.98	4099.0	7.71
30° & 150°	3.50	3.33	3083.0	4.33
45° & 60°	3.00	3.60	1079.0	1.73
45° & 90°	2.40	2.20	1084.0	6.22
45° & 120°	Yield	1.01	2883.0	5.78
45° & 135°	2.40	2.87	1531.0	3.20
60° & 90°	Yield	1.04	3362.0	3.07
60° & 120°	3.70	4.50	1179.0	2.99

**Shotcrete will fail in some points.

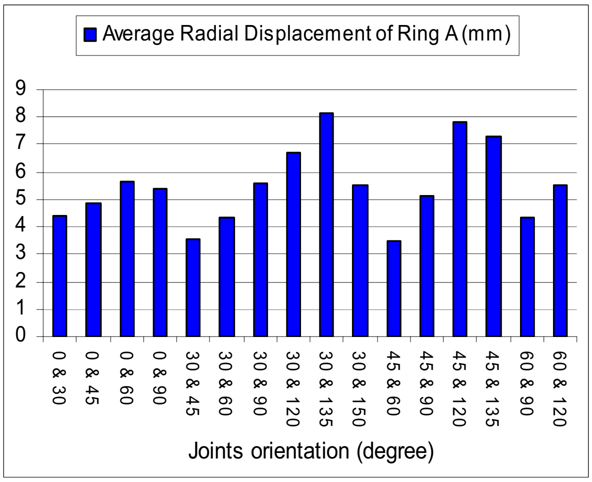

Figure 16. Effect of joints orientation on average radial displacement of ring A

6. CONCLUSIONS

In this paper, the use of rockbolts and shotcrete for tunnels excavated through jointed rock was modelled using the distinct element approach. The effect of various parameters pertaining to joints including joint spacing and orientation, friction angle, cohesion and dilation angle of the joints on the design of fully grouted rockbolts and fibre reinforced shotcrete were evaluated. It was shown that some of these parameters such as joint spacing and orientation can highly affect the surrounding rock displacements, the bending moments developed in the shotcere, the axial forces induced in the rockbolts and the shear forces generated in the grout. Joint friction angle and cohesion have especial effect in their low values on rock mass behavoiurs. Effect of joint dilation angle must be considered in its high values.

7. ACKNOWLEDGMENTS

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