Comparison of tunnel support requirements assessed by DEM and the Q system

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Abstract: There are several approaches for the design of support systems for tunnels excavated in rock. These approaches can be classified as analytical, numerical and empirical methods. Empirical methods are based on the experience obtained from past tunnelling projects. The Q system is one of the empirical methods used for classification of the rock mass and evaluation of support requirements.

In this paper, the effect of rockbolt length, diameter and spacing on the stresses developed in the rockbolts and the deformations developed in the rock mass are evaluated using the distinct element method. Also, the effects of the distance of supporting system from the tunnel face, tunnel depth and diameter on the stresses and deformations are investigated. The results of this numerical analysis are compared with the results obtained using the Q system. It is shown that the design of supporting system using the Q system is more conservative than the numerical method.

Résumé: Il y a plusieurs approches pour la conception du système de soutien pour les tunnels excavés dans la roche. Ces approches peuvent être classifiées en tant que méthodes analytiques, numériques et empiriques. Des méthodes empiriques sont basées sur l'expérience obtenue à partir des projets passés. Le système de Q est l'une des méthodes empiriques fréquemment utilisées pour la classification de la masse de roche et de l'évaluation des besoins en soutènement.

En cet article, l'effet des rockbolts longueur, le diamètre et l'espacement sur les efforts développés dans les rockbolts et les déformations développés dans la masse de roche sont évalués en utilisant la méthode distincte d'élément. En outre, les effets de la distance du système de soutien du visage de tunnel, de la profondeur de tunnel et du diamètre sur les efforts et les déformations sont étudiés. Les résultats de ces analyses numériques sont comparés aux recommandations du système de Q. On lui montre que la conception du système de soutien employant le système de Q est plus conservatrice que la méthode numérique.

Keywords: excavations, tunnels, classification, numerical models, rock mechanics, discontinuities.

INTRODUCTION

For the stability analysis of underground excavations and design of support systems three approaches comprising empirical, analytical and numerical methods can be used. 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.

Empirical approaches, such as the Q system, are based on the experience obtained from the past projects. Rock mass classification procedures constitute the base for the empirical approaches. In these methods, the values of stresses and displacements developed in the support system and the surrounding ground can not be evaluated.

In the analytical approaches, the design of support system is based on the theoretical models and relationships developed for certain conditions. In most of these formulations it is assumed that a circular cross section is excavated in a homogenous and isotropic media with hydrostatic in-situ stresses. In a jointed rock, equivalent continuum media is used in the analysis. Also, 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.

Numerical methods for the stress analysis can be classified into two main categories (Brady 1992):

- 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 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.
- 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.

In this paper, closely jointed rock masses are simulated using the Universal Distinct Element Code (UDEC) program and results of numerical simulations are compared with the recommendations of the Q system. UDEC is a two-dimensional numerical program based on the distinct element for discontinuum modelling. It simulates the response of discontinuous media (such as a jointed rock mass) subjected to either static or dynamic loading.

TUNNEL SUPPORT REQUIREMENTS

Problem Statement

It is assumed that a 10m diameter circular tunnel is excavated at a depth of 200m in sandstone containing two continuous joint sets deviated at $\pm 45^{\circ}$ from the vertical, with an average spacing of 1m and subjected to a hydrostatic in-situ pressure (k=1). The mechanical properties of the intact rock and joints are summarized in Table 1 (Goodman 1989, Rahn 1986, Stillborg 1986).

Table 1. 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 (degrees)	30
Cohesion of joints (MPa)	0.2
Dilation angle of joints (degrees)	0

Recommendations of the Q-system

It has been assumed that three joint sets exist; two joint sets striking parallel to the tunnel axis and one perpendicular the tunnel axis. The Q index calculation is summarized in Table 2.

Table 2. The Q index calculation

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	2
Joint water (J_w)	Dry excavation	1
Stress reduction (SRF)	Medium stress	1
Q index		6.6

According to Barton and Girmstad (1993) recommendations, 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 with ESR=1.

Numerical Simulation

Appropriate Numerical Approach

In the finite difference method, attention is focused on the continuum, whereas in the finite element method the interior of the problem domain must be discretized completely into separate elements, however in the block-jointed medium the interaction between blocks is of primary concern so in the case of closely jointed rock masses, the distinct element method is the most appropriate computational approach for either elastic or elasto-plastic analysis (Brady 1992).

In this paper, closely jointed rock masses have been simulated employing the UDEC program, mentioned earlier.

Constitutive Model of Intact Rock and Joints

To represent the behaviour of intact material, deformable blocks are used with constitutive model of Mohr-Coulomb plasticity, which is considered suitable for underground excavations. Also, Joint area contact-Coulomb slip constitutive model is employed for joint modelling, which is considered appropriate for the joints and faults analysis in the general rock mechanics.

The Constant Normal Stiffness (CNS) technique is more appropriate than the Constant Normal Load (CNL) technique for the stability analysis of an excavation and for simulation of the behaviour of bolted joints. 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). Values for the normal and shear stiffness of rock joints typically range from roughly 10 to 100MPa/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 38GPa/m and a shear stiffness of 7.6GPa/m are used in analysis.

Rockbolt Simulation

For permanent applications, the space between the bolt and the rock can be filled with cement or resin grout. The fully grouted rockbolt properties, which are used in the numerical simulations, are summarized in Table 3.

Table 3. Rockbolt 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

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), 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 scaled property is found by dividing the actual property by the element spacing. For rockbolt elements, the following properties should be scaled: elastic modulus of the rockbolt, tensile yield strength of the rockbolt, stiffness of the grout, and cohesive strength of the grout.

Shotcrete Simulation

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. The fibre reinforced shotcrete properties are summarized in Table 4.

Table 4. Shotcrete properties

Properties	Values
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

2D Model Boundary Conditions

Artificial boundaries do not exist in reality, but they should be introduced in order to constrain the number of elements. The model 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. In addition, the boundary conditions in a numerical model consist of the values of field variables (e.g., stress, displacement) that are applied at the boundary of the model.

Given the 10m diameter tunnel, a 60mx60m square has been chosen for 2D modelling of the rock mass. Compressive stresses are applied in the boundaries in accordance with the depth of the tunnel and the hydrostatic stress conditions.

Excavation and Support System Simulation

Before modeling the excavation, the model is solved to obtain the initial equilibrium and the displacements are reset, so only the changes in displacements due to the excavation can be monitored. Then the excavation is performed.

An initial support system, steel fibre reinforced shotcrete with a thickness of 50mm is modeled in combination with 3m long fully grouted rockbolts, regularly spaced at 2m. The rockbolts and shotcrete are extended in a 300 degree arc of the tunnel as shown in Figure 1.

In this excavation, it is assumed that the 2D plane which is modeled for simulating the support system around the tunnel is located 2.5m from the face of excavation. Therefore, the support system is activated after some convergence and relief of some part of the in-situ stress (Panet and Guenot 1982).

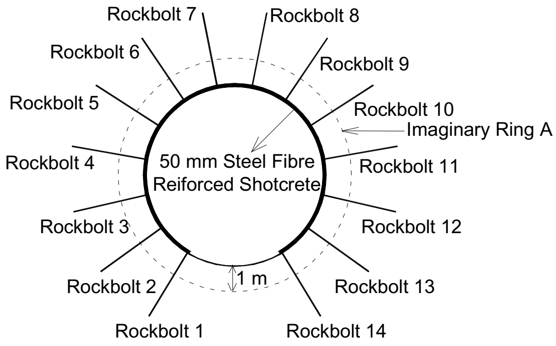


Figure 1. Support system around the tunnel

Analysis Results

The axial force in the rockbolts and the shear force in 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 is 9.35mm and the average radial displacement of ring A is 7.22mm.

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

Design Optimization

Appropriate safety factor for axial force of rockbolt and shear force of grout is between 2 and 3. In this section, the location of rockbolts around the tunnel periphery is optimized and the effect of rockbolt length, diameter and spacing on the stresses and deformations are evaluated.

Optimization of the Location of Rockbolts in 2D Plane

In optimized design, unnecessary rockbolts are omitted and in critical points of the 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 2. The results obtained from the revised analysis are presented in Table 6.

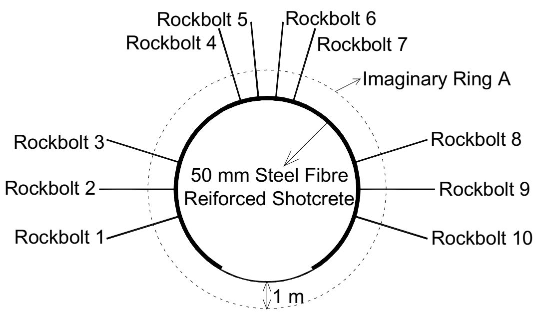


Figure 2. The location of rockbolts in the optimized design

The maximum axial force and moment in the shotcrete are 158.9kN/m and 3.2kN.m/m respectively. The maximum displacement of blocks is 9.5mm and 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.

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	86.2	2.3	117.8	2.5

Effect of Spacing of the Rockbolts along the Tunnel Axis

In order to investigate the effect of rockbolt spacing along the tunnel axis on the stresses developed in the supporting system, the spacing is varied between 1 and 4m. The results are illustrated in Figures 3 and 4.

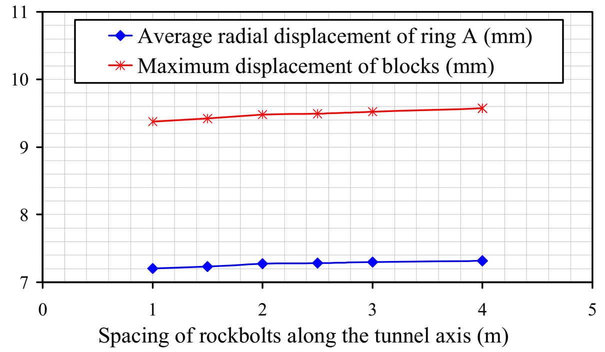


Figure 3. Effect of spacing of rockbolt along the tunnel axis on displacements

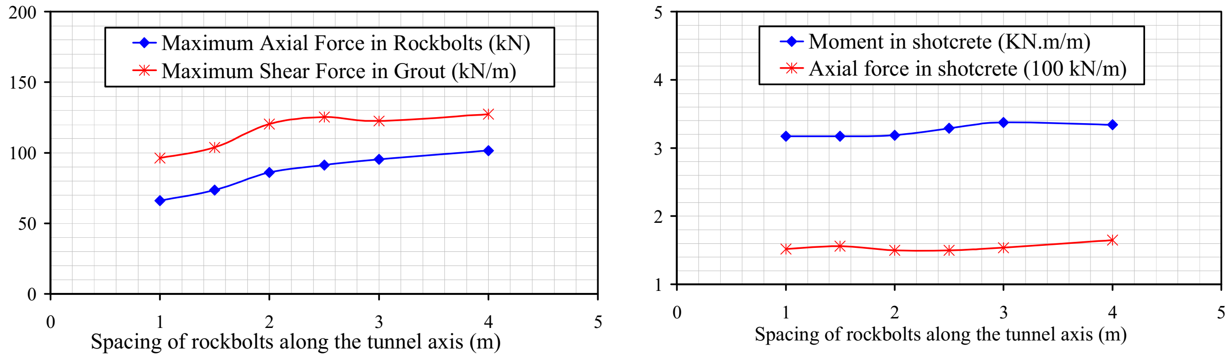


Figure 4. Effect of the spacing of rockbolts along the tunnel axis on the rockbolt and shotcrete forces

As shown in the figures, the safety factors of the rockbolts and the grout increase with a decrease of rockbolt spacing along the tunnel axis. Given the appropriate safety factors of rockbolts and grouts (2.3 & 2.5) a spacing of 2m is chosen for the rockbolts along the tunnel axis.

Effect of Rockbolt Length

In order to investigate the effect of rockbolt length on the design of support system and to evaluate the optimum length of rockbolts, its length is varied between 1.5 to 5m. The location of rockbolts around the tunnel periphery in all cases is the same as in Figure 2. The results are illustrated in Figures 5 and 6.

As shown in the figures, the stresses developed in the support system decrease with an increase of rockbolt length up to 3m and after this limit the results are not affected by rockbolt length. Therefore, the optimum length of rockbolt for this tunnel is 3m. It is clear that the changes in the rock mass properties or tunnel depth or diameter can change the optimum length of the rockbolt.

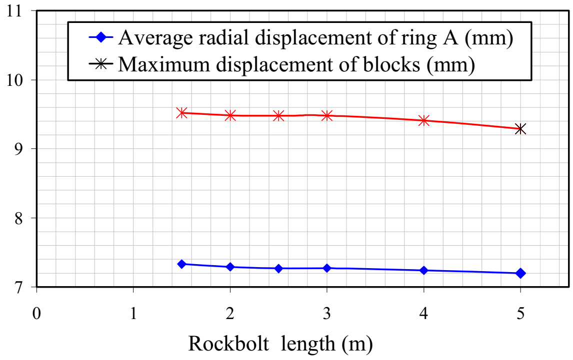
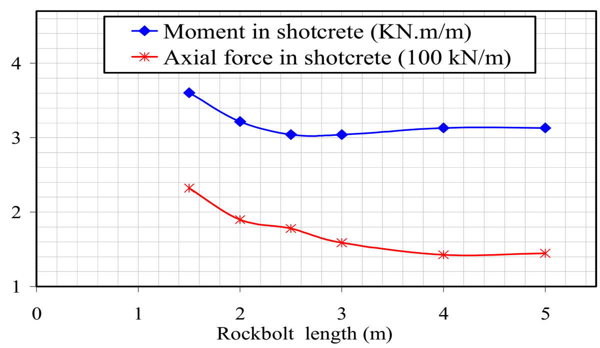


Figure 5. Effect of rockbolts length on displacements



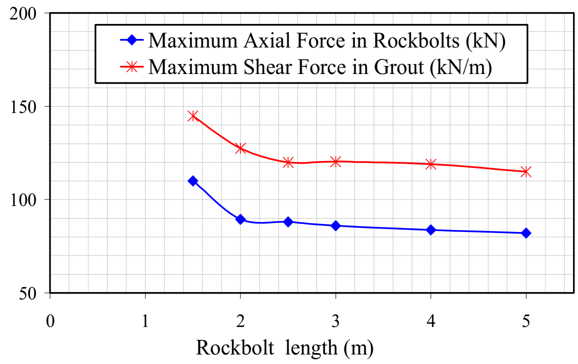


Figure 6. Effect of rockbolts length on rockbolt forces and shotcrete forces

Effect of Rockbolt Diameter

To investigate the effect of the rockbolts diameter on the design of the support system and to evaluate the optimum diameter of rockbolt, its diameter is changed between 20 to 40mm. The location of rockbolts around the tunnel periphery in all cases is the same as Figure 2. The results are illustrated in Figures 7 and 8.

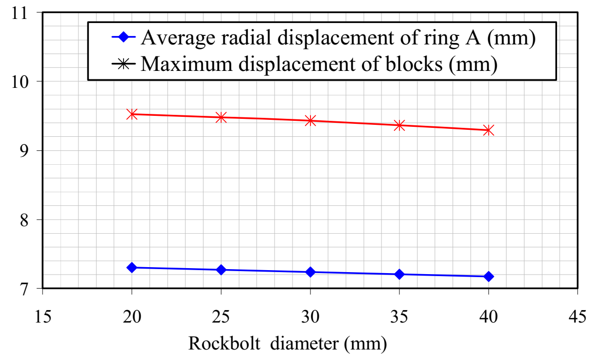


Figure 7. Effect of rockbolt diameter on displacements

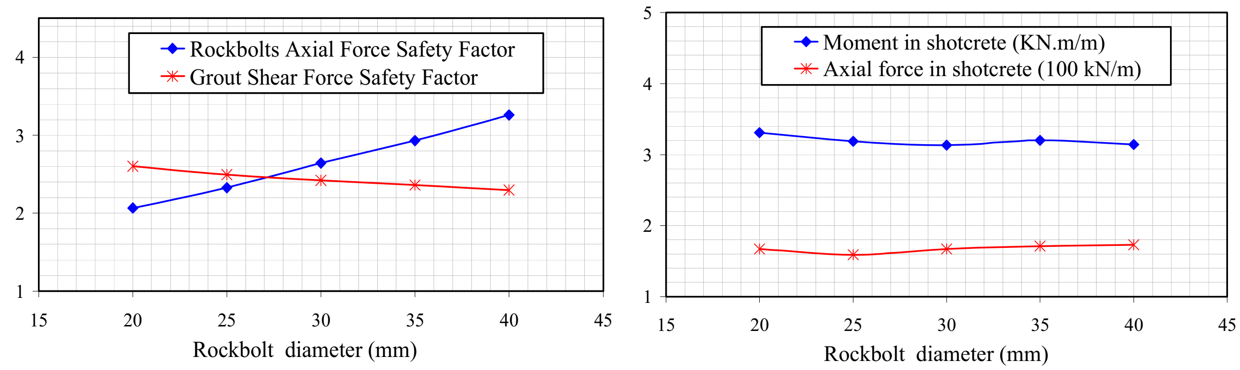


Figure 8. Effect of rockbolt diameter on rockbolt and shotcrete forces

According to the Figures, with an increase of rockbolt diameter not only does the safety factor for the rockbolts increase but it decreases for the grout. Therefore, the point where two curves cross each other is selected as the optimum diameter of rockbolts for supporting the tunnel. In this problem, two curves cross near the diameter of 25mm which is chosen as the appropriate diameter for the rockbolts. It is clear that the changes in the rock mass properties or tunnel depth or diameter can change the optimum diameter of the rockbolts.

Comparison of Numerical and Empirical Approaches

In both approaches, 50 mm steel fibre reinforced shotcrete with a thickness of 50mm is recommended for stabilizing the tunnel. In the Q system recommendations, 3m long rockbolts regularly spaced at 2.2m are suggested. Therefore, the rockbolt density is equal to 0.206. The numerical simulations suggest that 3m long rockbolts located around the tunnel periphery should be used, as shown in Figure 2 (rockbolt spacing along the tunnel axis=2m). The rockbolt density in this approach is equal to 0.19, with an average rockbolt spacing of 2.3m. Also, the simulations show that the evaluated optimum rockbolts length and diameter are similar to the Q system recommendations.

The investigations show that in evaluating the tunnel support requirements for this problem, the Q system recommendations are more conservative than the numerical approach.

Effect of Distance of Supporting System from the Tunnel Face

In order to investigate the effect of distance of support system from the tunnel face, its value is varied between 1 and 2.5m. This change simulated using Panet curve. The location of rockbolts around the tunnel periphery in all cases is the same as Figure 2. The results are illustrated in Figures 9 and 10.

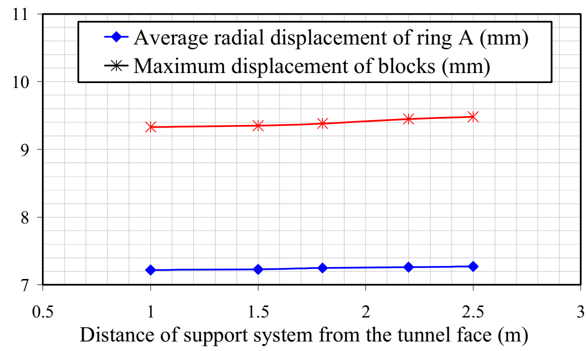


Figure 9. Effect of distance of support system from the tunnel face on displacements

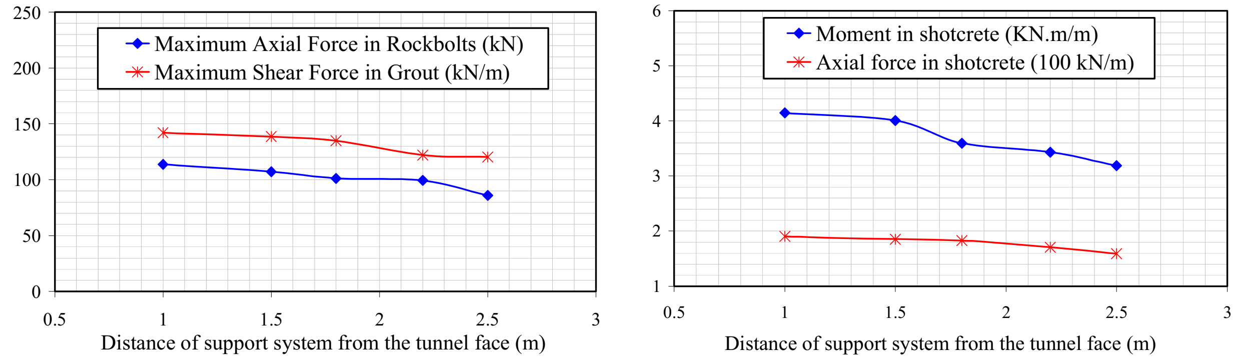


Figure 10. Effect of distance of support system from the tunnel face on rockbolt and shotcrete forces

As shown in the Figures, with an increase of the distance of support system from the tunnel face, the stresses developed in the rockbolts and shotcrete decrease and the deformations of surrounding rock mass increase. This is because increasing the support distance from the tunnel face results in increasing convergence of the tunnel periphery prior to providing support. Therefore, a small stress is carried by the support system. The results obtained for the analytical method based on the interaction between the rock mass and the dividing layer conform with the results for the simulations.

Effect of Tunnel Depth

In order to evaluate the effect of tunnel depth on support requirements, the depth of the tunnel axis was changed between 100 and 1000m. A support system consisting of 50mm thick shotcrete is modeled in combination with 3m long fully grouted rockbolts. The location of the rockbolts around the tunnel periphery is shown in Figure 2. The results are illustrated in Figures 11 and 12.

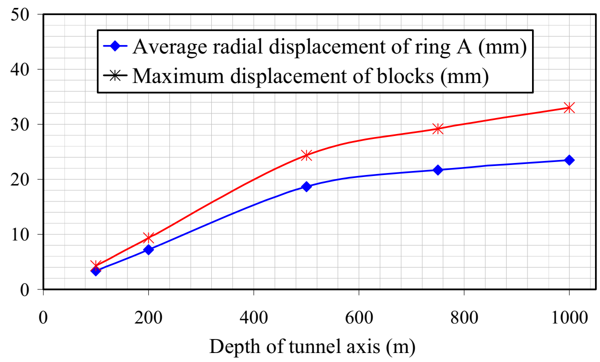


Figure 11. Effect of depth of tunnel axis on displacements

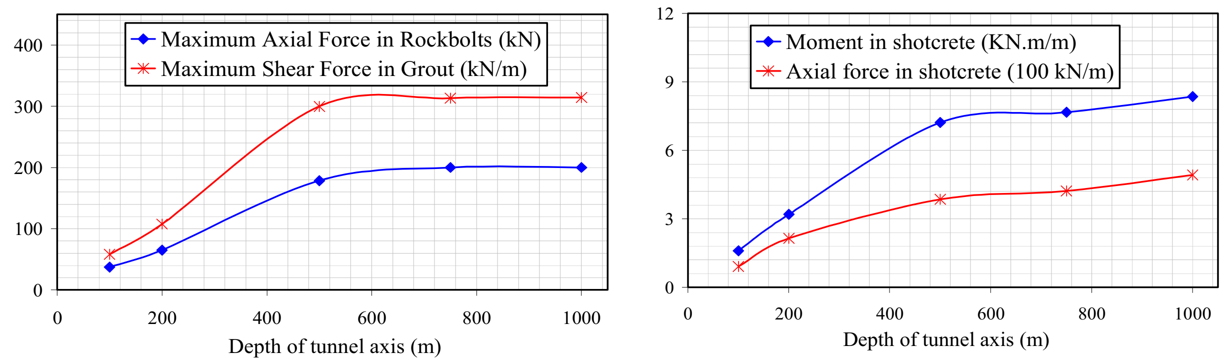


Figure 12. Effect of depth of tunnel on rockbolt and shotcrete forces

As shown in the figures, with an increase of the tunnel depth, the stresses developed in the support system and the deformations of the rock mass increase. Therefore, a heavier support system is required. For example in this problem, the support system used is not appropriate for tunnels deeper than 300m. It is clear from the figures that increasing of the tunnel depth up to a certain limit, in this case 500m, affects the stresses and deformations and after that limit an increase in depth does not affect the design.

Changing the depth of the tunnel affects the value of the Q index by changing the Stress Reduction Factor (SRF). In this problem, by increasing the tunnel depth from 100 to 1000m, the value of SRF changes from 1 to 10. Therefore, the Q index decreases from 6.6 to 0.66. The tunnel support requirement based on the Q system recommendations for various tunnel depths is summarized in table 7.

Table 7. The tunnel support requirement based on the Q system recommendations for various tunnel depths

Tunnel Depth	Q index	Rockbolt Length	Rockbolt Spacing	Thickness of shotcrete
100	6.6	3	2.2	40 - 50
200	6.6	3	2.2	40 - 50
500	1.32	3	1.7	50 - 90
750	0.88	3	1.6	90 - 120
1000	0.66	3	1.6	90 - 120

According to Table 7, the Q system recommends heavier support system for deeper tunnels. This is in accordance with the results of the numerical simulations. If the tunnel depth increases, the Q system recommends closer rockbolt spacing and a higher shotcrete thickness.

Effect of Tunnel Diameter

In order to investigate the effect of tunnel diameter on support requirements, the tunnel diameter is changed between 5 and 20 m. Again a support systemconsisting of 50mm shotcreteis modeled in combination with 3m long fully grouted rockbolts. The location of rockbolts around the tunnel periphery is shown in Figure 2. The results are presented in Figures 13 and 14.

As shown in these figures, with an increase of the tunnel diameter, the stresses developed in the support system and deformations of the rock mass increase. Therefore, a heavier support system is required. For example in this problem, the recommended system is not appropriate for tunnels with a diameter more than 10m. Also, if the tunnel diameter is smaller than 10m, the system is conservative.

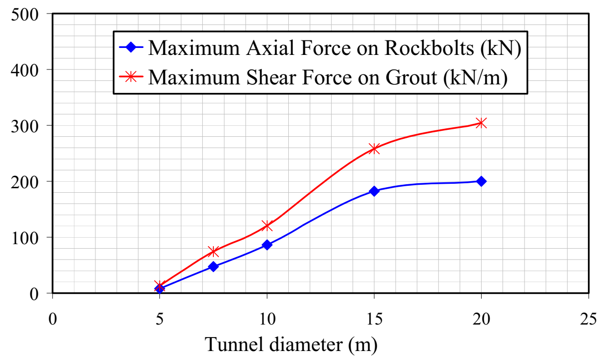
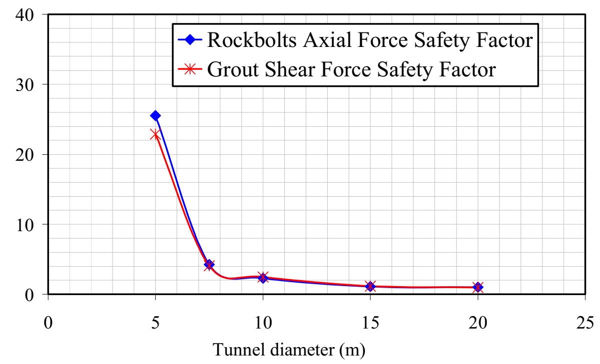


Figure 13. Effect of tunnel diameter on displacements



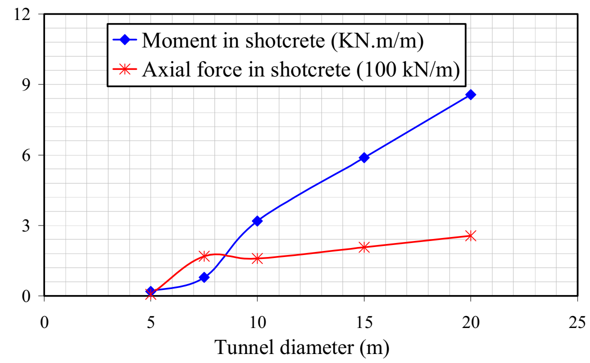


Figure 14. Effect of tunnel diameter on rockbolts and shotcrete

The support requirements based on the Q system recommendations for various tunnel diameters are summarized in Table 8. According to this table the Q system recommends heavier support system for tunnels with larger diameters. This conforms with the results of numerical simulations. If the tunnel diameter is increased, the Q system recommends that the length of rockbolt and the thickness of shotcrete tare also increased.

Table 8. The tunnel support requirements based on the Q system recommendations for various tunnel diameters

Tunnel Depth	Rockbolt Length	Rockbolt Spacing	Thickness of shotcrete
5	2.4	2.2	-
7.5	2.7	2.2	40 - 50
10	3	2.2	40 - 50
15	4	2.2	40 - 50
20	5	2.2	50 - 90

CONCLUSION

In this paper, the effect of rockbolts length, diameter and spacing on the stresses developed in the rockbolts and the deformations developed in the rock mass were evaluated using the distinct element method. Also, the effects of the distance of the support system from the tunnel face, tunnel depth and diameter on the stresses and deformations are investigated. The results of these numerical analyses were compared with the recommendations of the Q system. It was shown that the design of the support system using the Q system is more conservative than the results obtained by the distinct element method.

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