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Technical Note

Design of Supports for Excavation in Tunnels with Limited Stand-up Time

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Introduction

Ranadive and Parikh (2003) have discussed the problem of tunnel excavation in rock mass with infinite stand up time. More often than not such ideal system is not encountered and therefore enroute construction and installation of the supports are essential. The current practice is to adopt New Austrian Tunnelling Method (NATM) for such situations. The NATM was first introduced by Rabcewicz (1964), which is too well known to need a detailed description. However, in context with the proposed investigations two important features of the method need to be highlighted.

NATM recognizes the importance of timely introduction of the supports. This is an aspect, which is not easily amenable to analytical treatment. As the construction progresses the site conditions would dictate, the instance of time during construction at which the support needs to be installed. Some aspect of this was discussed by Parikh and Ranadive (2000 and 2001) for highlighting the importance and application of NATM. It is felt that the theoretical calculation of the timely requirement may in future be covered by analytical process demanding three dimensional considerations rather than the two dimensional approach which is in vogue for the tunnel design.

Above represented the first of the two aspects. The second aspect deals with provision of an appropriate support system. With available information on geotechnical constitutions of the strata through which the tunnel is proposed to be driven, a plane strain finite element analysis should be conducted in advance to facilitate the rational design of supports. Many

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types of supports are employed in practice such as shotcrete, rock bolts, anchor rods, steel ribs, concrete lining, prefabricated lining, etc., of these shotcreting and installation of rock bolts appear to be of primary importance, hence, it is proposed in this paper to deal with their design through the two dimensional finite element analysis. Further, an attempt is made to validate Hoek and Brown (1980) failure criteria by application of finite element method of analysis for NATM.

Practical Aspect

The NATM is an observational method, which requires application of a thin layer of shotcrete with or without rock bolts, wire mesh fabric, lattice girder and monitoring and observing the convergence of the opening. If the observed convergence exceeds the acceptable limits, then subsequent applications of next layers of shotcrete are required until the convergence has stopped or it is within the acceptable range. The shotcrete thickness is thereby optimized according to the admissible deformations. The guidelines for tunnel lining design are prepared by technical committee on tunnel lining design of the underground technology of American Society of Civil Engineers technical council. These are published by O'Rourke (1984), which explains, the details regarding observational programmes and provide a basis for organizing observations and instrumentation for lining design purpose.

According to Megaw and Bartlett (1983) the basic principle of NATM is to ascertain and control the development of stresses and deflections and their interaction with supports and lining, and thereby to establish within the surrounding rock a load-bearing ring. Bieniawski (1984) in his very clear explanation of NATM, also lists 7 principles related to mobilization of the strength of the rock mass, shotcrete protection, measurements, flexible support, closing of invert, contractual arrangements and determination of support measures by rock mass classification. A case has been presented by Mahatab et al. (1992) regarding application of NATM; and predicted and measured tunnel convergence prior to routine reinforcement installed behind the face from the full heading. The technical papers and case studies reported by Barton and Grimstad (1994), Fugeman (1991), Hari Prasad (1991), Irshad (1988), Huges (1987), Martin (1991), Murphy (1994), Purrer (1990) and Sinha (1988) give details regarding application of NATM and the construction with reference to the important aspects such as analysis, designs, safety, excavation procedures, initial and final support systems, waterproofing etc. Whereas, the papers and cases published by Batra (1994), Caputo (1987), Grondziel (1991), and Holmgren (1987) provide the materials used for shotcrete and support systems such as steel fibrous concrete, polymeric membranes, steel fibre-shotcrete, bolt-anchors etc. Fugeman (1991) illustrated the effective use of instrumentation for NATM. Case studies of collapse have been presented by Leichnitz (1990), Wallis (1987 and 1991) and the remedies with the help of

NATM have been discussed. Penny (1991) proposed that with respect to normal and shield tunnelling there is saving of about 15% of total cost by using NATM.

Some researchers in the field of tunnelling opposed the use of NATM. Fawcett (1994) strongly opposes the NATM in London clay giving its disadvantages, whereas, in the same paper (Fawcett, 1994) Sauer recommends the method giving its advantages. Kovari (1994) presents evidence from the literature to prove that NATM theory is based on faulty logic and ambiguous terminology. Victor (1996) has given shortcomings in NATM and suggested another method known as rapid shotcrete supported tunnelling method (RSST).

Thus, it can be concluded from the literature review that, constant monitoring of the ground behaviour and its interaction with immediate support elements will be resulting in the highly safe and economical construction of tunnels by NATM. Thus, excavation by NATM, constant observation of ground movement and related data will allow the designer to ensure controlled excavation and installation of adequate immediate support. The provision of optimum support can be ensured by application of finite element technique to NATM, which is discussed in the following sections.

Application of Plane Strain Finite Element Technique

From the various details presented above it is clear that the construction by NATM needs understanding of the behavior of the rock mass as the excavation progresses. During excavation the gauge readings of displacements at key locations would indicate to an experienced engineer the time at which the supports need to be introduced. But the details such as the kind of supports and the structural constitution should be established so that the execution team will be equipped with the structural component (shotcrete, rock bolts, etc.) that would be introduced, to provide the required stability. It means that there is now scope for preparing analysis and design of the supports in advance.

For geotechnical materials with indefinite stand-up time (the time to failure) this aspect is not of much importance, and a linear analysis was justified for arriving at the state of affairs with regard to the formations at the opening. If the support is required to be introduced prior to failure of geotechnical material, the failure module for the material will have to be taken into account, whereby, a stress dependent non-linear analysis will have to be performed. In fact, some aspects of this kind of analyses have been covered by Ranadive and Parikh (2001) while dealing with the case of disintegrated geotechnical material. The other three types of surrounding geotechnical material (i.e. intact rock, widely spaced rock joints and closely spaced rock joints) considered by Parikh and Ranadive (2001) in the technical paper are basically rock masses, therefore, a failure criterion for the rock mass needs to be evolved, so that the supports could be made to interact in such a manner that the failure of the rock mass is at best avoided or at least minimized.

Failure Criterion

By now the subject of failure criterion, irrespective of various material have been established and the same are available in standard literature such as given by Hoek and Brown (1980) and Mahatab et al. (1992) etc. In majority of the cases the failure criteria are functions of the principal stress (σ_1, σ_2 and σ_3) wherein, σ_1 is the major principal stress, σ_2 is the intermediate principal stress, and σ_3 is the minor principal stress. Application of the failure model which are functions of σ_1, σ_2 and σ_3 would however be practically inconvenient, because in any case the decision regarding failure and its avoidance are usually considered to be function of σ_1 and σ_3 arising from plane strain deformations. In fact, in a long tunnel the actual failure would be a definite function of such plane strain behaviour. Thus majority of the works theoretical as well as practical depends only on the principal stress parameters (σ_1, σ_3). Based on extensive laboratory and practical observations Hoek and Brown (1980) have established a failure criterion dependent on the behaviour of triaxial test specimen in laboratory, wherein σ_3 represents the confining pressure and σ_1 the applied load in the axial direction of the sample.

The theory proposed by Hoek and Brown (1980) is presented below in short, which is based on theoretical and experimental work. The empirical relationship between the principal stresses associated with the failure of rock is

$$\sigma_1 = \sigma_3 + \left(m\sigma_c\sigma_3 + s\sigma_c^2\right)^{1/2} \tag{1}$$

where σ_c = the uniaxial compressive strength of the intact rock material,

m and s = the constants which depend upon the properties of the rock and upon the extent to which it has broken before being subjected to the stresses σ_1 and σ_3.

The uniaxial compressive strength of the rock material is given by substituting $\sigma_3 = 0$, in Eqn.1 giving,

$$\sigma_{cs} = \left(s\sigma_c^2\right)^{1/2} \tag{2}$$

For intact rock, $\sigma_{cs} = \sigma_c$ and $s = 1$, where, σ_{cs} is the uniaxial compressive strength of the intact rock specimen.

For previously broken rock, $s < 1$ and the strength at zero confining pressure is given by substitution of $\sigma_1 = 0$ in Eqn.1 and by solving the resulting quadratic equation for σ_3:

$$\sigma_t = \frac{1}{2}\sigma_3\left\{m-\left(m^2+4s\right)\right\}^{1/2} \tag{3}$$

where σ_t = uniaxial tensile strength of rock.

In addition to the relationship between the major and minor principal stresses at failure, it is sometimes convenient to express the failure criteria in terms of the shear and normal stresses acting on a plane inclined at an angle β to the major principal stress direction. When the inclination β of the failure surface is known the shear and normal stresses τ and σ can be determined directly from the following equations:

$$\left.\begin{aligned} \tau &= \frac{1}{2}(\sigma_1-\sigma_3)\sin 2\beta \\ \sigma &= \frac{1}{2}(\sigma_1+\sigma_3)-\frac{1}{2}(\sigma_1-\sigma_3)\cos 2\beta \end{aligned}\right\} \tag{4}$$

This failure criterion is used to obtain the deformations at key locations of tunnel, which is later on compared with the results obtained by FEM software developed and used for the purpose.

Illustrative Problem

It is proposed to illustrate the application of above criteria for deciding the nature of support that would arrest disintegration of rock mass or failure of rock mass during excavation. A typical case study from Hoek and Brown (1980) solved by closed form solution is taken up for the purpose of illustration. Hoek and Brown (1980) obtained the displacements at key locations of tunnel by above mentioned failure criteria. The same is tried herein with the help of stress dependent non-linear finite element analysis, and the results are compared.

Material Properties

The material properties used as data for the analysis are as given below.

Uniaxial compressive strength of rock (σ_c)	= 69 MPa
Material constants for original rock mass, m	= 0.5
s	= 0.001
Modulus of elasticity of rock mass (E)	= 1380 MPa
Poisson's ratio of rock mass (μ)	= 0.2
Unit weight of broken rock mass (γ_r)	= 0.02 MN/m^3
In situ stress magnitude (P_0)	= 3.31 MPa
Modulus of elasticity of shotcrete (E_c)	= 20 GPa
Poisson's ratio of shotcrete (μ_c)	= 0.25
Diameter of rock bolt	= 25 mm
Length of rock bolt	= 5.3 m
Modulus of rock bolts (E_s)	= 200 GPa

A circular tunnel with internal radius (r_i) = 5.35 m is considered, whereas, length of tunnel assumed for the analysis is 1 m, meaning thereby length of tunnel with shotcrete is 1 m and rock bolts are assumed to be placed at 1 m center to center.

Finite Element Programme

The computer programme developed in Fortran is used for non-linear finite element analysis of geotechnical materials. The details of the programme have been already discussed in a technical paper by Ranadive and Parikh (2001), which are valid for the present purpose also. The minor difference lies in the fact that, in the technical paper by Ranadive and Parikh (2001), the action of overburden pressure was considered through a stepwise incremental analysis, wherein, in this case a similar stepwise analysis is introduced to consider the effect of in-situ stresses on the excavation process. Ranadive and Parikh (2003) have described the effect of residual stresses and sequence of excavation on displacement of tunnel by finite element method for various shapes of opening. It means that the concept of the technical papers by Ranadive and Parikh (2001 and 2003) have been combined here for solving the problem. The excavation of the circular tunnel opening is assumed to be from center to outward direction. Total eight support systems cases have been studied in this connection as enlisted in Table 1. For the purpose of analysis isoparametric 2-noded line elements representing rock

TABLE 1 : Details of Displacements under Various Support Systems

Case No.	Investigation	v_{invert} (mm)	v_{crown} (mm)	$u_{sidewall}$ (mm)	Incre-ments
1.	Behaviour of rockmass with an opening without any support	12.198	−12.198	−12.198	84
2	Introduction of shotcrete layer of 5 cm thickness	12.203	−12.198	−12.20	97
3	Introduction of shotcrete layer of 7.5 cm thickness	11.793	−11.793	−11.793	100
4	Introduction of shotcrete layer of 10 cm thickness	11.0997	−11.0997	−11.0997	100
5	Introduction of rock bolts of length 5.3 m and dia. 25 mm (without shotcrete)	12.215	−12.201	−12.441	88
6	Introduction of shotcrete of 5 cm thickness and rock bolts of length 5.3 m and dia. 25 mm	12.061	−11.945	−12.206	100
7	Introduction of shotcrete of 7.5 cm thickness and rock bolts of length 5.3 m and dia. 25 mm	11.316	−11.317	−11.526	100
8	Introduction of shotcrete of 10 cm thickness and rock bolts of length 5.3 m and dia. 25 mm	10.6609	−10.6609	−10.8579	100

v_{invert} indicates vertical displacement of invert of tunnel in mm.
v_{crown} indicates vertical displacement of crown of tunnel in mm.
$u_{sidewall}$ indicates horizontal displacement of sidewall of tunnel in mm.
−ve sign indicates displacement in downward direction or towards −ve x-direction.

bolts, 4-noded quadrilateral elements representing shotcrete as well as surrounding geotechnical material and 3-noded triangular elements representing continuum at the center of the tunnel opening are used. (Fig.1b). Key diagram (Fig.1a) represents the salient features of tunnel opening, overburden and surrounding geotechnical material properties, whereas, Fig.3b illustrates the typical idealization for case 1 (i.e. rockmass without any support) used for the purpose of analysis. Half symmetry about vertical axis of tunnel is considered. The nodes on central vertical axis are restrained from horizontal displacement, whereas, nodes along the boundary of the continuum are restrained from horizontal as well as vertical displacement.

Results and Discussion

The solution details have been through stepwise linear analysis comprising of 100 increments for the removal of residual stresses prevailing in the opening area. Failure of the tunnel opening was observed by huge displacements at the crown and sidewall. These huge displacements indicate

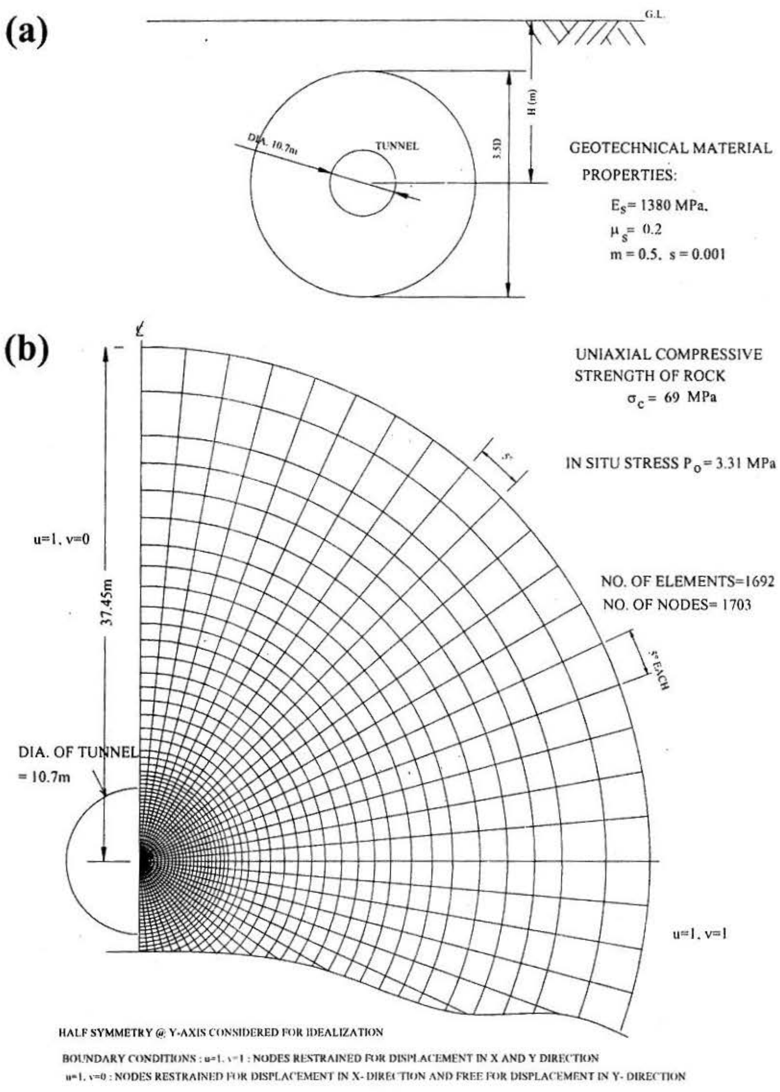

FIGURE 1 : (a) Key Diagram; (b) Idealization of Circular Tunnel for NATM

failure of the tunnel and in turn termination of the computer programme beyond that particular increment for the removal of residual stresses. The details of various cases and the corresponding observations of key point displacements are presented in Table 1. Maximum 100 increments are considered for the analysis, whereas, the maximum number of increments required (may be upto the failure) are given in Table 1. The results of displacement of invert, crown and sidewall of the tunnel for the eight cases

TABLE 2 : Presentation of Results for Cases 1 to 8

Iterative Load Increments	v_{invert} (mm)	v_{crown} (mm)	$u_{sidewall}$ (mm)	Material Properties
Case 1 : Rock mass without any support				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1692 Material properties as given in the text
25	3.630	−3.630	−3.630	
50	7.260	−7.260	−7.260	
75	10.890	−10.890	−10.890	
84	12.197	−12.197	−12.197	
100	10^{15} (Failure)	10^{15} (Failure)	10^{15} (Failure)	
Case 2 : Shotcrete layer of 5 cm thickness				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1692 Material properties as given in the text
25	3.145	−3.1451	−3.1451	
50	6.7935	−6.7935	−6.7935	
75	9.4354	−9.4354	−9.4354	
98	12.2031	−12.1979	−12.2030	
100	−615 (Failure)	−124000 (Failure)	−1598000 (Failure)	
Case 3 : Shotcrete layer of 7.5 cm thickness				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1692 Material properties as given in the text
25	2.9484	−2.9484	−2.9483	
50	5.8968	−5.8968	−5.8968	
75	8.8452	−8.8452	−8.8452	
100	11.7935	−11.7937	−11.7937	
Case 4 : Shotcrete layer of 10 cm thickness				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1692 Material properties as given in the text
25	2.7749	−2.7749	−2.7749	
50	5.5498	−5.5498	−5.5498	
75	8.3247	−8.3247	−8.3248	
100	11.0996	−11.09966	−11.0997	
Case 5 : Rock bolt of 25 mm dia. and length = 5.3 m				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1822 Material properties as given in the text
25	3.4677	−3.4677	−3.5337	
50	6.9354	−6.9354	−7.0674	
75	10.4031	−10.4031	−10.6011	
100	Programme terminates at 89th increment i.e. failure.			
Case 6 : Shotcrete 5 cm thick + Rock bolt of 25 mm dia. and L = 5.3 m				
0	0	0	0	No. of nodes = 1703 No. of Elements = 1822 Material properties as given in the text
25	3.0145	−3.0145	−3.0703	
50	6.0290	−6.0290	−6.1407	
75	9.0435	−9.0435	−9.2111	
100	12.0615	−11.9458	−12.2068	

TABLE 2 : Continued ...

Iterative Load Increments	v_{invert} (mm)	v_{crown} (mm)	$u_{sidewall}$ (mm)	Material Properties
Case 7 : Shotcrete 7.5 cm thick + Rock bolt of 25 mm dia. and L = 5.3 m				
0	0	0	0	No. of nodes = 1703
25	2.8291	−2.8291	−2.8814	No. of Elements = 1822
50	5.6582	−5.6582	−5.7628	
75	8.4874	−8.4873	−8.644	Material properties as
100	11.3165	−11.3170	−11.5261	given in the text
Case 8 : Shotcrete 10 cm thick + Rock bolt of 25 mm dia. and L = 5.3 m				
0	0	0	0	No. of nodes = 1703
25	2.6652	−2.6652	−2.7144	No. of Elements = 1822
50	5.3304	−5.3304	−5.4289	
75	7.9957	−7.9957	−8.1434	Material properties as
100	10.6609	−10.6609	−10.8579	given in the text

v_{invert} indicates vertical displacement of invert of tunnel in mm.
v_{crown} indicates vertical displacement of crown of tunnel in mm.
$u_{sidewall}$ indicates horizontal displacement of sidewall of tunnel in mm.
−ve sign indicates displacement in downward direction or towards −ve x-direction.

studied are presented in Table 2. As mentioned above maximum 100 increments are considered for the analysis, whereas, some intermediate increments in percentage and the corresponding displacement values have been given in Table 2. The details regarding the comparison between various cases have been presented graphically in Fig.2.

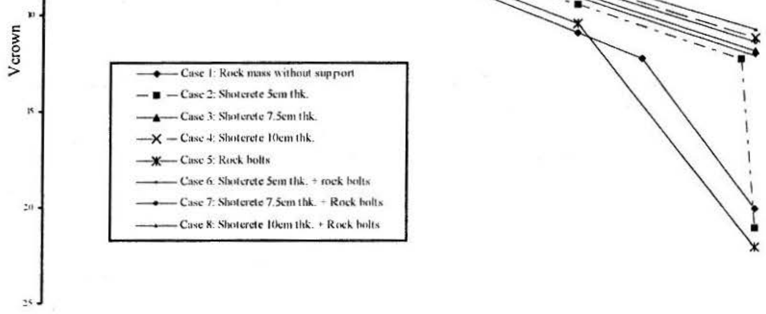

FIGURE 2 : Crown Displacement for Cases 1 to 8

Case 1 represents the rockmass without any support system. It withstands upto the iterative load increment of 84. After this increment the graph shows huge displacement, meaning thereby, sudden collapse and failure of the tunnel. Similarly Case 2 and Case 5 show failure at 97th and 88th increment. Comparison between all these cases (Case 1 to Case 8) shows that rock mass without any support (Case 1) is unsafe and early failure may take place. To avoid the failure, support system needs to be introduced. Case 2 is introduction of shotcrete layer of 5 cm thickness. The properties of which are given in above section. In this case also the maximum iterations performed are 97 and thereafter failure resulting in inadequate support. Case 3 is introduction of shotcrete layer of 7.5 cm thickness. In this case the maximum iterations performed are 100 and no failure was observed resulting in adequate support system. Case 4 deals with introduction of shotcrete layer of 10 cm thickness and the maximum iterations performed are 100, thus no failure was observed resulting in adequate support system. Thus, if Case 3 and 4 are compared it is economical to select Case 3 only.

Now, another support system in the form of rock bolts is considered. Each rock bolt is of 2.5 cm dia. and 5.3 m length introduced at 1 m center to center along the periphery of the tunnel. Case 5 deals with introduction of rock bolt without any shotcrete layer. Only, after 88th iteration the programme is terminated resulting in inadequate support system. Hence, shotcrete layer of 5 cm thickness in addition to the rock bolts is introduced, which is considered in Case 6. Case 7 and 8 are introduction of shotcrete of 7.5 cm thick and 10cm thick in addition to the rock bolt of dimensions as mentioned above. All these cases were found to be safe and adequate as the failure is not observed even at the 100th increment. The comparison between Cases 6, 7 and 8 is made with respect to the thickness of shotcrete and the conclusion is drawn that, Case 6 is safe and economical. From the above discussion it is clear that, if Cases 3 and 6 are compared with respect to the displacements (Table 1), Case 3 is best one as it shows minimum displacement values.

The final decision could be taken after cost comparison between Case 3 and Case 6, i.e. the cost of shotcrete of 7.5 cm thickness and cost of shotcrete of 5.0 cm thickness with rock bolts of 5.3 m length and 2.5 cm dia. In this investigation tunnel support system in the form of shotcrete, rock bolt and their combination is considered for the analysis, whereas, there is provision in the software to facilitate all other types of tunnel support systems, e.g. steel liners, cast in-situ concrete linings, prefabricated lining segments, etc.

It is observed that the values of the parameters such as settlement at the crown, sidewall and invert of the tunnel, for a particular increment in each case are same. This is due to the consideration of the behaviour of rock in elastic limit. The deformation will produce differential settlement at the crown, sidewall and invert of the tunnel, only when failure conditions are simulated. For which the 3-D analysis is clearly warranted. The illustrative problem solved by Hoek and Brown (1980) by closed form solutions is tried in this paper by finite element method applying the principles of NATM, and it is observed that, the displacements within the elastic limit (i.e. displacements at crown = displacement at side wall = displacements at invert = 12.2 mm) perfectly tally. Hence the concept of NATM can be effectively used to solve the complex problems in tunnelling with reference to the important aspects such as analysis, designs, safety, excavation procedures, initial and final support systems in almost all difficult ground situations.

Conclusions

The method adopted has demonstrated its potential in connection with the decision making with respect to tunnel support system. The illustrative problem suggests that optimum combination of shotcrete and rock bolts happens to be a promising solution while undertaking the construction as per NATM principles. Given the data in advance of the construction the design chart could in fact be prepared regarding the capabilities of the supports in arresting the failure. While monitoring the observations during the construction this will help in deciding the time at which the support needs to be installed. Occasionally a problem may arise, wherein, the developed proposed system may be ideal in arresting the failure mechanism, but at the same time the geotechnical material may be weak enough to develop large displacements. In such cases the analysis can be extended further, by incorporating installation of steel sets and even secondary concrete linings. While monitoring the observations during the construction this will help in deciding the time at which the support needs to be installed.

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Notations

$\sigma_{1,}\ \sigma_{2,}\ \sigma_3$	=	major principal stress, intermediate principal stress, minor principal stress,
σ_c	=	uniaxial compressive strength of the intact rock material,
m and s	=	the constants which depend upon the properties of the rock and upon the extent to which it has broken before being subjected to the stresses σ_1 and σ_3,
σ_{cs}	=	uniaxial compressive strength of the intact rock specimen,
σ_t	=	uniaxial tensile strength of rock, the failure criteria in terms of the shear and
β	=	angle between shear and normal stresses acting on a plane
τ and σ	=	shear and normal stresses
E, μ	=	modulus of elasticity of rock mass, Poisson's ratio of rock mass,

γ_r	=	unit weight of broken rock mass,
P_0	=	In situ stress magnitude
E_c , μ_c	=	modulus of elasticity of shotcrete, Poisson's ratio of shotcrete,
E_s	=	modulus of rock bolts,
r_i	=	internal radius of tunnel,
v_{invert}	=	vertical displacement of invert of tunnel,
v_{crown}	=	vertical displacement of crown,
$v_{sidewall}$	=	horizontal displacement of sidewall.