Support Requirement for Underground Excavation Using Numerical Technique

A.K. Ghosh*
B.K. Saha
&
J.M.Shirke

Rock Mechanics Division
Central Water & Power Research Station
P.O. Khadakwasla , Pune -411 024 ,
Maharashtra,India
Phones: 091-020-24380511Ext. 3425/3297/3245
Fax : 091-020-24381004
**E-mail: emarald249@yahoo.co.in*

ABSTRACT

Rock Mass Classification systems such as Geomechanics Classification (RMR) proposed by Bieniawski(1973) and NGI Tunnelling Quality Index (Q) by Barton et al.(1974) act as an useful tool for initial estimation of support requirement immediately after tunnel excavation. Suitable adoption of numerical technique, based on site specific conditions is helpful to study alternative support systems required for stabilization of structure including visualization of failure zones / zones of overstress surrounding the excavation. A review of support requirement for a typical cross section of an excavation of water conductor tunnel in Maharashtra in poor quality rock mass where some form of initial failure has already taken place is presented. Study of support requirements from rock mass classifications as well as 2D Stress Analysis by FEM are undertaken and compared. It has been found that numerical technique in conjunction with RMR and Q can provide important clues about the behaviour of structure prior to excavation, provided the measured geological parameters and properties of rock mass and rock materials including rock mass profile at the section under study is proper with acceptable degree of accuracy.

Keywords: Deccan trap basalt; Volcanic breccia; RMR; Q; Elastic; Plastic; Support; In situ stress; Rock bolt; Shotcrete.

1. INTRODUCTION

A 22km long water conductor tunnel in Maharashtra is under construction to convey water by gravity flow from reservoir at higher elevation to facilitate irrigation at lower drought prone areas. The tunnel alignment cuts across the divide between two valleys, nine minor nallas (gullies) also run almost parallel to the alignment. During

construction, most sections of the tunnel, passing through weak or jointed rock strata experienced failures in the form of rock falls of various magnitudes from crown portion including moderate to heavy seepage at certain stretches. In this paper, a typical section of the tunnel passing through poor quality rock strata where damages due to rock fall has taken place is considered for studying support requirement (from geological considerations as well from numerical analysis using finite element technique).

2. GEOLOGY

2.1 Rock Types Encountered

The entire tunnel passes through the Deccan Trap formation made up of thick pile of one compact Basalt flows having their top surfaces almost horizontal and with difference in the field characteristics in the flow from its top to bottom. Due to gradient, there is elevation difference of about 6.5m between the upstream and downstream portal and the tunnel takes almost oblique sections of the combined basalt flow. Geological cross section of the tunnel alignment through compact basalt flow is shown in Fig. 1. The tunnel from upstream portal to a distance of about 4.0 km traverses through top portion of compact Basalt flow represents hydro-thermally altered vesicular Amygdaloidal Basalt whereas the rest of the tunnel up to downstream portal passes through middle or jointed portion of Compact Basalt. In order to categorize the rock mass, geological logging of drilled cores of about 47 drill holes has been undertaken by the project geologists. Brief descriptions of varieties of rock mass encountered are as follows.

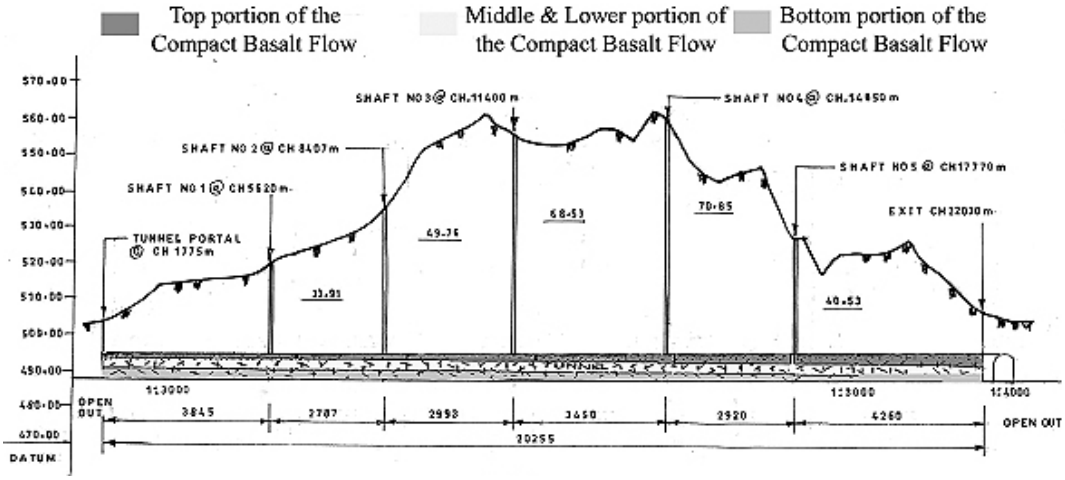

Fig.1 - Geological cross-section of the tunnel alignment through the compact basalt flow

Vesicular Amygdaloidal Basalt: At the top portion of Basalt flow, the rock is mostly purple or reddish in color due to hydrothermal alteration and is vesicular and amygdaloidal .In this zone, veins and injections of red tachylitic basalt also occur and due to network of these tachylitic veins and injections, rock breaks up into angular fragments of different shapes and sizes and appears like volcanic breccia in which rock fragments are held together in red tachylitic lava matrix. This variety of rock,

commonly termed as volcanic breccia / red bole, is a common feature in Deccan Trap rock in and around Maharashtra also gets deteriorated very fast on exposure to atmospheric conditions and gets converted into powder like material. Such soft variety of rock with almost no joint, though desirable from drilling point of view, poses problems like over-breaks and rock falls from crown and is met with in the tunnel for a distance of about 5km from the upstream portal. A greenish variety of this category is also present which is of better strength quality compared to its purple variety.

Compact Basalt: The middle and lower portion of the compact basalt flows are free from vesicles and amygdales and occur in true sense as compact rock. Joints, which are the contraction cracks developed during cooling and solidification of lava, are found to be present in this portion. Rock mass is jointed with closely to broadly spaced joints with number of discontinuities varying from well developed 3 sets to poorly developed 1 set of joints including some random joints. Condition of joints varies from tight and fresh to open and weathered. Due to presence of joints, excavated surface of rock is in dissected condition and uneven having deep angular depressions and wedge shaped protrusions with sharp edges. Due to the joint patterns, shapes, spacing etc., rock falls from crown portion are visible in most of the remaining sections either (a) in the form of slabbing forming deep grooves and notches, or (b) causing deep depressions in the form of chimney due to presence of wedge shaped joint blocks and (c) in the form of roof collapses due to presence of columnar jointed compact basalt. The remaining section of the tunnel after 5.0 km from upstream portal passes through this variety of rock mass.

Amygdaloidal Basalt: This relatively soft but less jointed variety of rock mass lies in the bottom portion of the compact basalt flow.

2.2 Overall Geotechnical Features

Geological reports (unpublished) on tunnel condition provided by Maharashtra Krishna Valley Development Corporation (MKVDC), Maharashtra, gives a fairly good understanding about the geotechnical features of the entire tunnel. The conditions of joints vary from tight and fresh to open and weathered. Since the top flow of Basalt is exposed to surface while the lower two flows are exposed to a sloping ground on either sides of hill, pronounced weathering including seepage of various magnitudes has been encountered in tunnel above the crown portion. Reason of seepage can be attributed to irrigation and presence of percolation tanks in the close vicinity of the tunnel and has been found to be high in rainy seasons. Rock fall of various magnitudes from crown portion has been observed at number of sections along the entire length of the tunnel and in extreme cases it has caused inverted V-cut at crown, damage to chain link and dislodging of rock blocks around rock bolts. Typical cross sections of the tunnel with damages at crown portion due to rock fall are shown in Fig. 2.

2.3 Rock Mass Quality

Rock characterization by geomechanics classification (RMR) proposed by Bieniawski et al. (1973) and NGI rock mass quality index (Q) by Barton et al. (1974) has been carried out by the project geologist for the rock category appearing at the crown portion and a copy of the report has been provided by the MKVDC. A brief summary of the

percentage distribution of RMR and Q values for representative rock mass of entire tunnel based on 271 data points is presented in Fig. 3. From the figures it can be seen that about 25% and 22% of the rock mass of entire length fall under 'poor' category as per RMR and Q systems respectively.

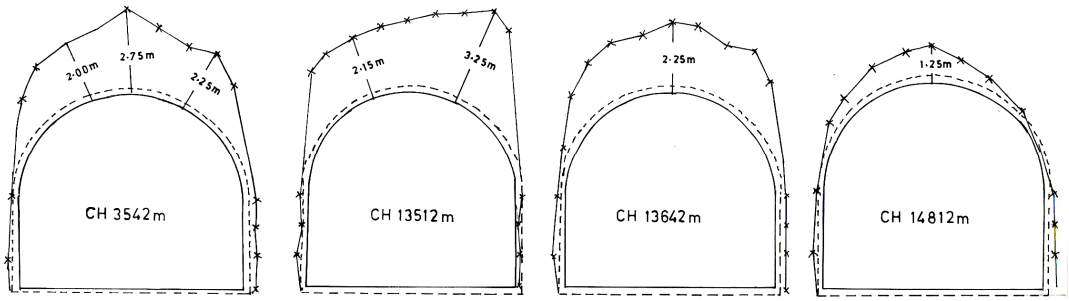

Fig.2 - Typical cross-sections showing magnitudes of rock fall from crown portion

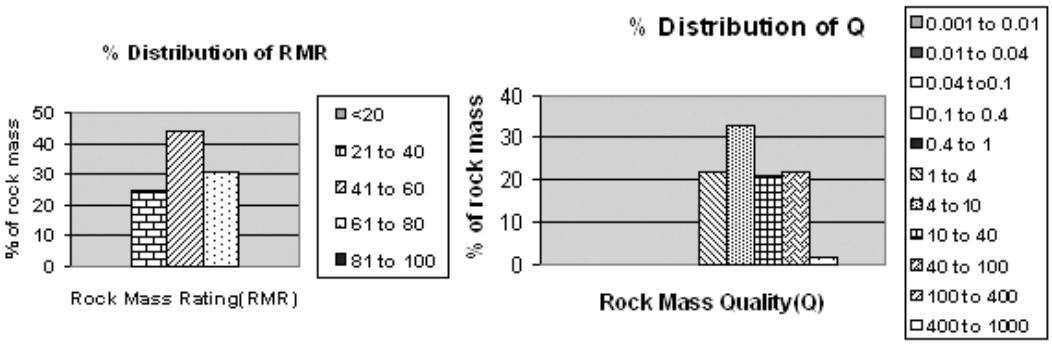

Fig.3 - Percentage distribution of RMR and Q of rock mass for entire tunnel

3. SELECTION OF TYPICAL CROSS-SECTION

Since rock mass of 'poor' category needs particular attention regarding stability and safety for any construction activity, one of such cross sections (Fig. 4) is selected for study of support requirement from geological considerations as well as numerical analysis. The entire section passes through varieties of hypo-thermally altered vesicular amygdaloidal basalt termed as volcanic breccia. The rock mass falls under poor category with RMR and Q values of 25 and 1.67 respectively. Heavy water seepage is also encountered at this section. As a temporary measure of support, both chain link mesh as well as rock bolts have been provided immediately after excavation. Although un-jointed nature of the rock mass provides suitable medium for tunneling, exposure to alternate humid and dry atmosphere has caused disintegration of the rock and as a result, the tunnel section has suffered roof collapse along with rock bolts.

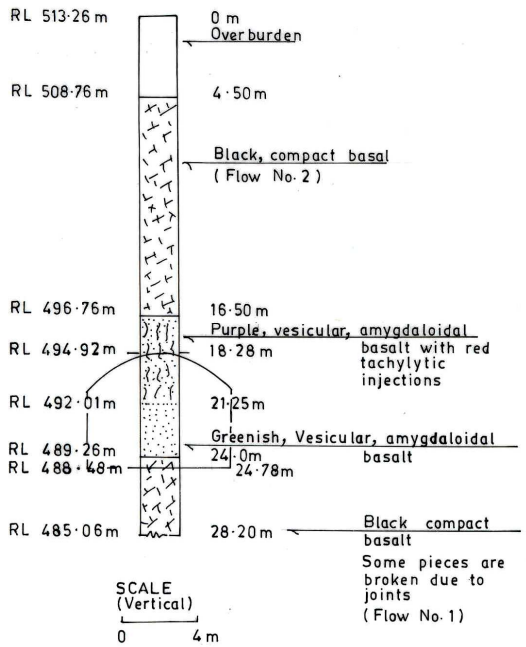

Fig.4 - Geological cross section of tunnel under study for support requirement

4. SUPPORT SYSTEM

4.1 Based on Geomechanics Classification (RMR)

The geomechanics classification or the Rock Mass Rating (RMR) for the jointed rock masses was developed by Bieniawski in 1973 utilizing following six basic parameters, all of which are measurable in the field.

(i) Uniaxial compressive strength of intact rock material
(ii) Rock quality designation (RQD)
(iii) Spacing of discontinuities
(iv) Condition of discontinuities
(v) Ground water condition
(vi) Orientation of discontinuities

To apply the geomechanics classification, the rock mass along the tunnel route is divided into a number of structural regions, i.e. certain geological features are more or less uniform within each region. The above six classification parameters are determined for each structural region from measurements at field. Based on these measurements and guidelines in a tabular form by Bieniawski, each parameter is assigned a rating and the sum total of these ratings is the RMR value for that category of rock mass. For the section as shown in Fig. 4, RMR value of 25 has been assigned to the volcanic breccia at the crown portion and as per guidelines for excavation and support of 10 m span rock tunnels in accordance with RMR system (Bieniawski, 1989), the rock mass at crown can be designated as ‘Poor’, Category IV with RMR between 21 and 40, and the support requirement as per the guidelines are given below.

(i) *Rock bolt* - Systematic bolts 4 to 5 m long, spaced 1-1.5m in crown and walls with wire mesh
(ii) *Shotcrete* - 100 to150mm thick in crown and 100mm in side walls
(iii) *Steel sections* - Light to medium ribs spaced at 1.5m where required

4.2 Based on NGI Rock Mass Quality Index (Q)

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al. (1974) of the Norwegian Geotechnical Institute (NGI) proposed rock mass quality index (Q) to determine the rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by Eq. 1,

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \quad (1)$$

where
RQD = Rock quality designation,
J_n = Joint set number,
J_r = Joint roughness number,
J_a = Joint alteration number,
J_w = Joint water reduction factor, and
SRF = Stress reduction factor.

Based on the rock mass condition at the crown portion, rating of each of the above parameters has been obtained by project geologist and the Q value of 1.67 has been assigned for the volcanic breccia. In order to know the stability and support requirements of underground excavations using Q, Barton et al. (1974) defined additional parameters such as the equivalent dimension (De) of the excavation and the excavation support ratio (ESR.). The equivalent dimension is obtained by dividing the span, diameter or wall height of the excavation by ESR. The values of ESR for various excavation category suggested by Barton et al. (1974) are related to the intended use of the excavation and to the degree of safety. The equivalent dimension (D_e) is defined as follows.

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio (ESR)}} \quad (2)$$

The present section falls under category of water tunnels for hydropower (excluding penstocks) and is assigned an ESR value of 1.6 and for the excavation span of 8.0 m, the equivalent dimension D_e works out as 5. Now based on Q value of 1.67 and equivalent dimension of 5, estimation of support requirement as per Q system is done from the published chart by Grimstad and Barton (1993) and is shown in Fig. 5. From Fig. 5, a value of De =5.0 and Q = 1.67, places this section in category 5. Based on Barton et al. (1980), the length L of rock bolts for roof and sidewalls and maximum unsupported

span has been estimated as 2.75m, 2.66m and 3.9m respectively from the equations as detailed below.

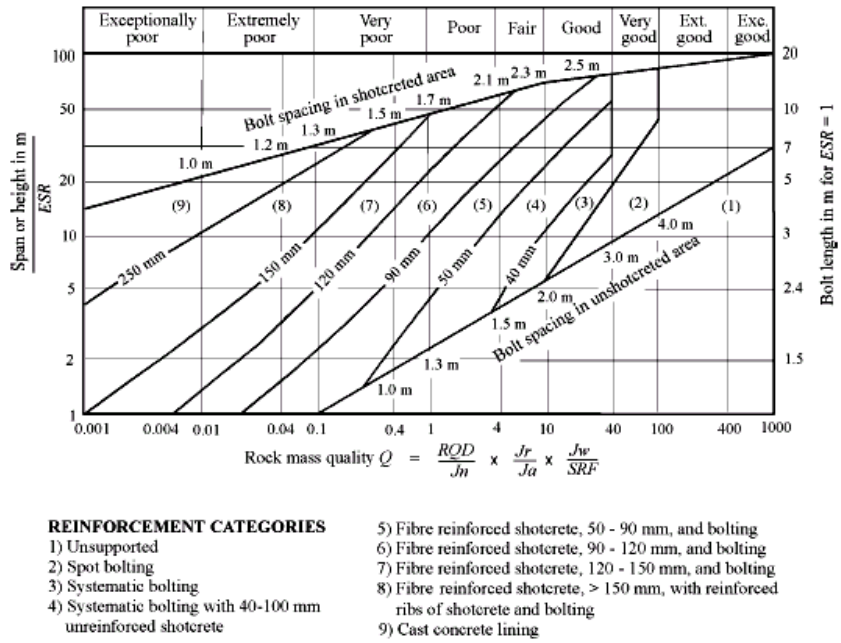

Fig. 5 - Estimation of support category based on Q system

Length of rock bolt (L) = 2 + (0.15B/ESR) for roof
= 2 + (0.15H/ESR) for wall
Maximum unsupported span = $2 \times ESR \times Q^{0.4}$

where L is length of rock bolt in m; B is excavation span in m; and H is excavation height in m.

So for the present section, the support requirement for the volcanic breccia rock mass at the crown portion as per Q system is,

(i) *Rock bolt* - Systematic bolts 2.75 m long, spaced about 1.3 to 1.5 m in the un-shotcreted area and 1.7m to 2.1m in the shotcreted area.
(ii) *Shotcrete* - 50 to 90mm thick steel fibre reinforced shotcrete.

5. NUMERICAL ANALYSIS

The support requirement for the crown portion of the section under study by the two different classification systems differs considerably from each other and the initial support to be provided towards stabilization of structure will have to be entirely based on actual site conditions and experiences of site geologist and practicing engineers. So in order to acquire more refined approach in this regard, it is decided to carry out numerical analysis of the structure, based on parameters such as opening geometry, geology of the section under consideration, material properties of rock mass and intact rock material, proper loading conditions and suitable criteria to describe the failure

mechanism. Such exercise can help to visualize the extent of probable failure zone around opening and several alternative support systems to choose from to decide the extent of support requirement necessary for stabilization. The model is prepared and analyzed based on the actual geological cross section of the tunnel (Fig. 4) and 2D FEM software Phase^2 of Rocscience Group, Canada has been used for the stress analysis. Although various rock types of the site have not been tested, test data of similar rock categories from nearby region of nearly identical geological features and tectonic history which has been tested both in the in situ as well as in the laboratory by the authors, are used for this present analysis.

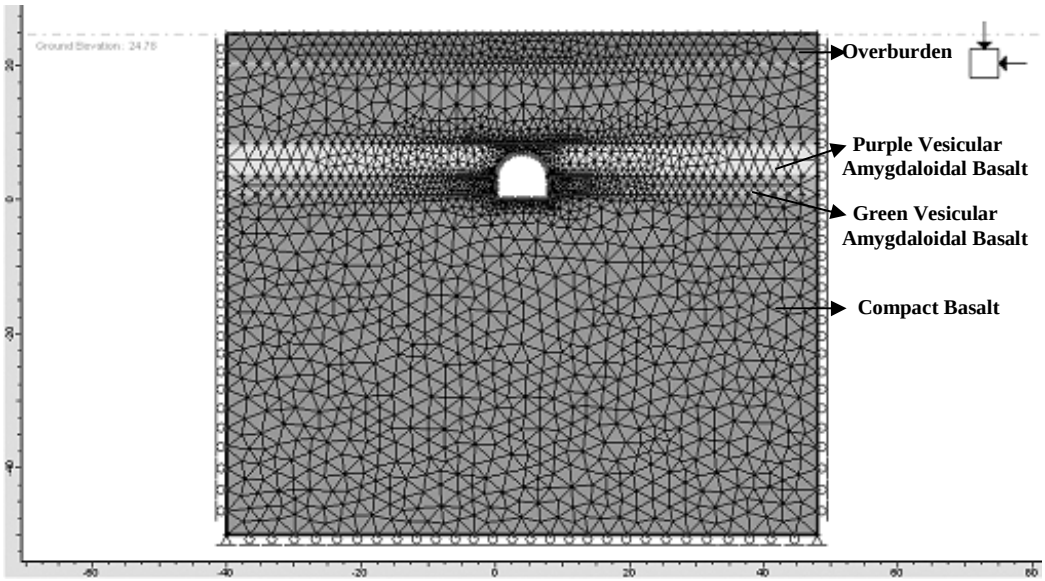

Fig.6 - 2D Finite element model showing mesh geometry and boundary conditions

The 2D finite element model for initial plane strain elastic analysis for the cross section of the tunnel under study for support requirement has been prepared using software Phase^2. The discretized and meshed finite element model using 3 nodded triangular elements with proper boundary condition, loading and materials in different color codes with respective properties including material boundaries at elevations as per actual cross section is shown in Fig. 6. The assumed properties for different materials as assigned in the present model are based on test data from nearby region including guidelines of Hoek and Brown (1980) on approximate relationship between rock mass quality and constants, to choose value of constants m and s for pre as well as post failure stages. Rock properties as assigned to the model including the value of constants are presented in the following Table 1. Since no in situ stress measurement data is available, based on measured stress data of nearby site and considering low rock cover of only 24.78m, gravitational mode of loading is used for the analysis with horizontal to vertical in situ stress ratio as 1.

For Elastic analysis, the materials have been assumed as linear, isotropic and elastic and from the strength criteria point of view, the materials have been designated as Hoek and Brown materials. The displacement pattern from elastic analysis (Fig. 7) shows a maximum displacement of 0.42mm at the crown section and lesser amount of

displacement mainly at the sidewalls. The contours of strength factor in Fig. 8, representing ratio of available rock mass strength to induced stress, show significant zone of overstress mainly at the crown portion including sidewalls. Although the amount of maximum displacement is very small, the extent of yield zone at crown portion suggests collapse of the section. Moreover since the rock mass here is Red Bole, differential displacements in conjunction with deterioration of rock mass strength over period of time due to chemical alteration process may lead to progressive ravelling of rock blocks from crown, which can further lead to a collapse situation, if the crown section is left unsupported. Based on elastic analysis results, since the zone of overstress is significant, it has been decided to carry out plastic analysis.

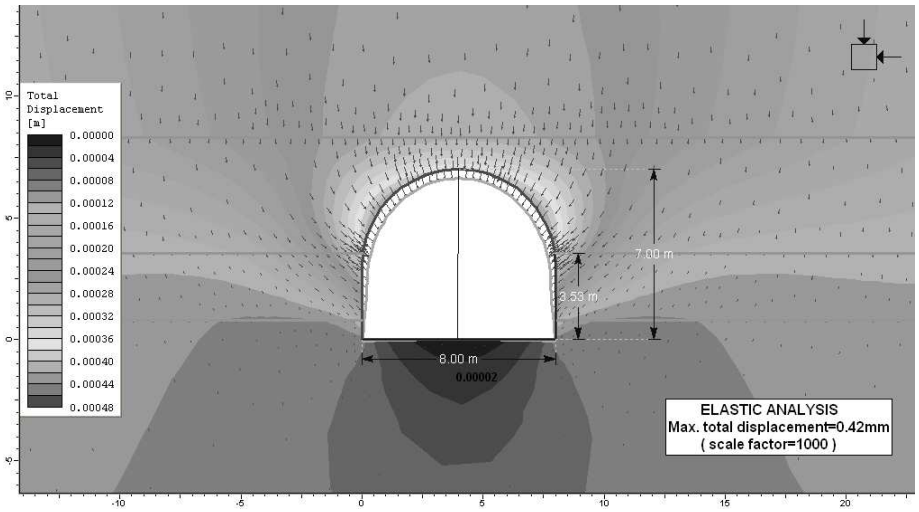

Fig.7 - Displacement profile with displacement vectors from elastic analysis

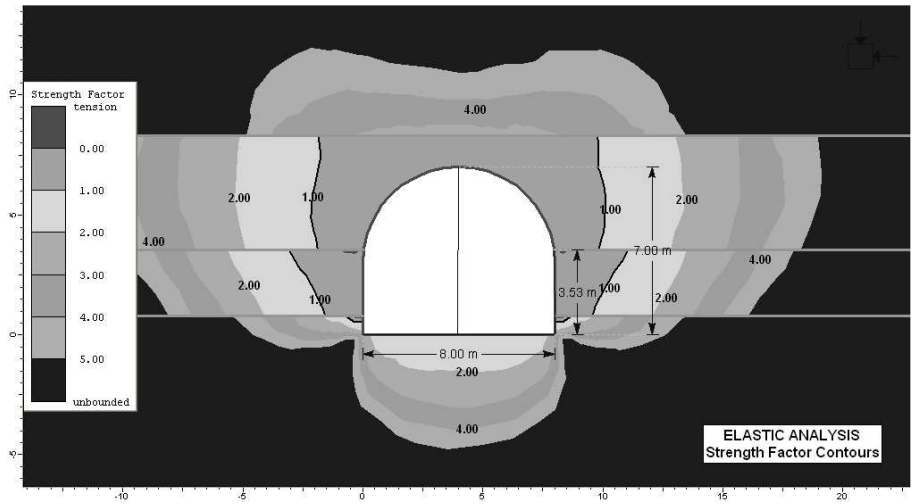

Fig.8 - Strength factor contours from elastic analysis showing significant zone of overstress mainly at the crown section

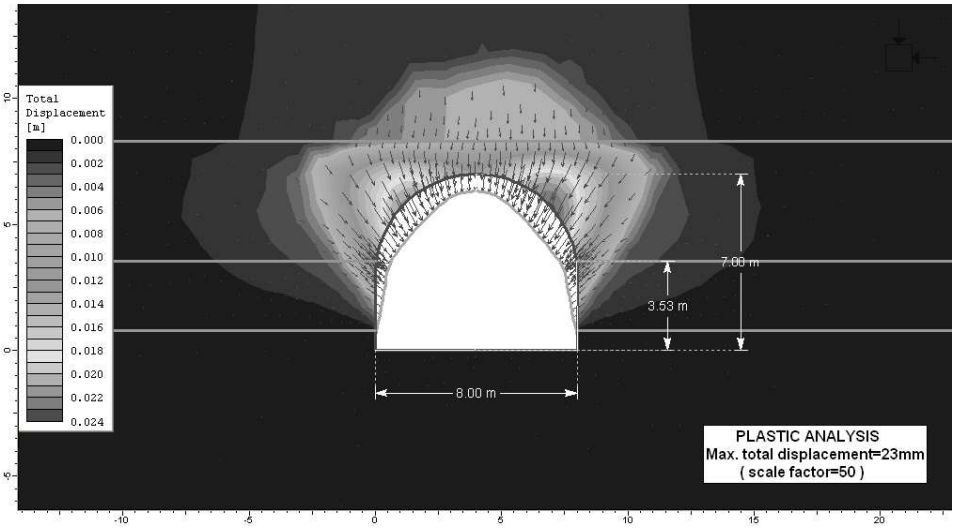

Fig.9 - Displacement profile from plastic analysis with displacement vectors and deformed view

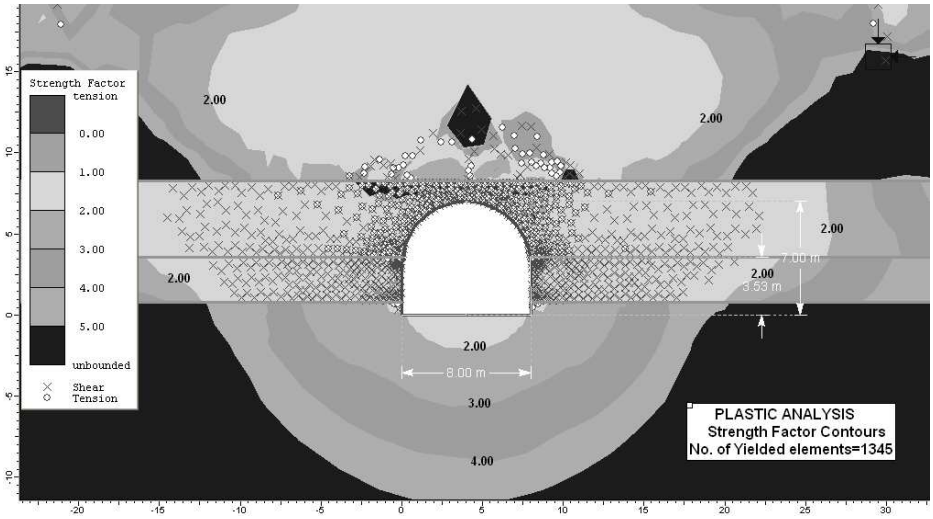

Fig.10 - Strength factor contours from plastic analysis with yielded finite elements

The material types of the model are now designated as plastic and the model is re-run. The amount of plastic displacements (Fig. 9) is now 23mm which is almost 55 times the maximum displacement from elastic analysis and occurring mostly at the crown portion though minor displacements at the side walls also can be observed. From strength factor contours (Fig. 10) it can be seen that much larger zone is encircled by the of strength factor = 2 compared to elastic analysis and due to yielding, there is no zone of strength factor < 1. Yielded zone show a total number of 1345 yielded finite elements out of a total of 4328 elements indicating both shear and tensile failure around the excavation and corresponds roughly with the zone of strength factor < 1 from elastic analysis. The extent of yield zone from both elastic and plastic analysis thus clearly explained the rock fall from crown portion experienced at this section (as shown in Fig. 2).

Table 1 – Rock properties and values of constants for the numerical analysis

Material	Rock Category/ Quality	E_M (GP_a)	σ_c (MPa)		m		s		γ (MN/m^3)	ν
			Peak	After Dilation	Peak	Residual	Peak	Residual		
Over burden	Lithified argillaceous rocks; RMR=3, Q=0.01; Very Poor	2	2	0	0.01	0.01	0	0	0.016	0.4
Black compact basalt	Fine grained igneous crystalline rocks; RMR=65, Q=10; Good	40	100	0	1.7	1.0	0.004	0	0.027	0.2
Purple vesicular amygdaloidal basalt with red tachylitic injection (Red bole)	Arenaceous rocks; RMR=23, Q=0.1; Poor	5	5	0	0.08	0.08	0.00001	0.00001	0.020	0.28
Green vesicular amygdaloidal basalt	Arenaceous rocks; RMR=44, Q=1; Fair	11	30	0	0.03	0.015	0.0001	0	0.022	0.22

Notations: E_M = Modulus of Deformation of rock mass; σ_c=unconfined compressive strength; m & s = Hoek constants, γ = rock material density and ν = Poisson's ratio.

6. DESIGN OF SUPPORT

From the support requirement based on RMR and Q, it is obvious that support suggested by Q system is less. So as a first trial, it is decided to see the effect of Q-based support system on the stabilization of the excavation geometry. Accordingly based on guidelines discussed earlier, a support of 2.8m long and 25mm diameter fully grouted un-tensioned steel dowels with maximum load capacity of 20ton is added normal to excavation boundary with in-plane spacing of 1.5m and out of plane spacing of 1m. Besides rock bolts, 90mm thick layer of Steel fibre reinforced shotcrete (for better strength, durability and reduced rebound) with Young's modulus of 30,000MPa, peak / residual compressive and tensile strengths of 35MPa /5MPa and 5MPa / 0 respectively has been used for the crown and sidewalls. Both rock bolt and shotcrete support are applied in crown and the sidewall (upto the zone of green vesicular amygdaloidal basalt) and the model is rerun. The enlarged view of excavation model after rock bolt and shotcrete support is shown in Fig. 11. Now the amount of plastic displacements (Fig.12) around excavation has become nil and maximum total displacement of the excavation boundary has also reduced from 23mm to 9.6mm i.e. almost a reduction of the order of 42 per cent has taken place from that of the unsupported excavation (Fig. 9). Strength factor contours (Fig.13) although shows sufficient reduction in the yield zone as well as number of yielded elements i.e. from 1345 yielded finite elements of the unsupported section to 855 for section with initial trial support, further reduction is desirable so that the extent of yield zone lies within the envelope of rock bolt support.

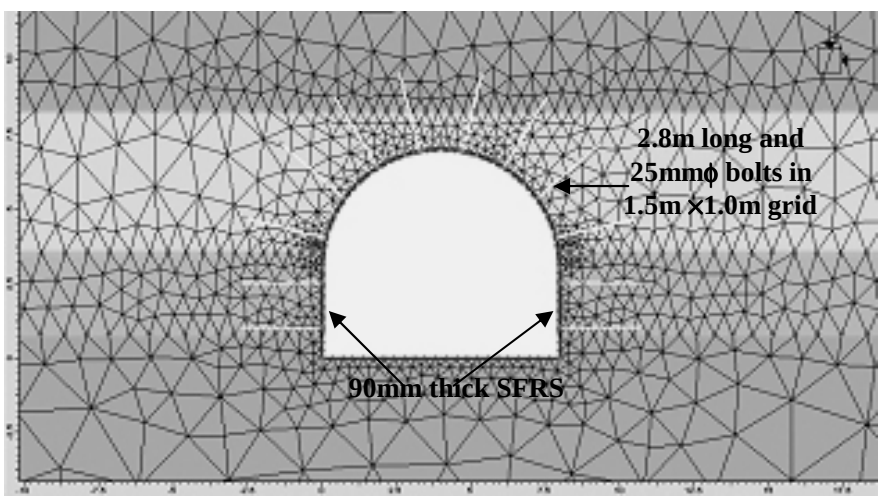

Fig.11 - 2D Finite element model with initial trial support from Q System (For support of 2.8m long 25mm dia rock bok bolts in 1.5mX1.0m grid and 90mm thick SFRS)

As a further attempt to reduce the extent of yield zone around opening so that it is contained within the rock bolt support envelope as well as to obtain an uniform deformed profile of the excavation, an attempt to improvise the support already designed based on Q-system has been done. As a step towards such improvisation, keeping the other parameters for rock bolt same as that used for initial trial support , 6m long bolt is used in a radial pattern at the crown portion only with a reduced in-plane spacing of 0.5m. The shotcrete is now applied to the complete section except base with

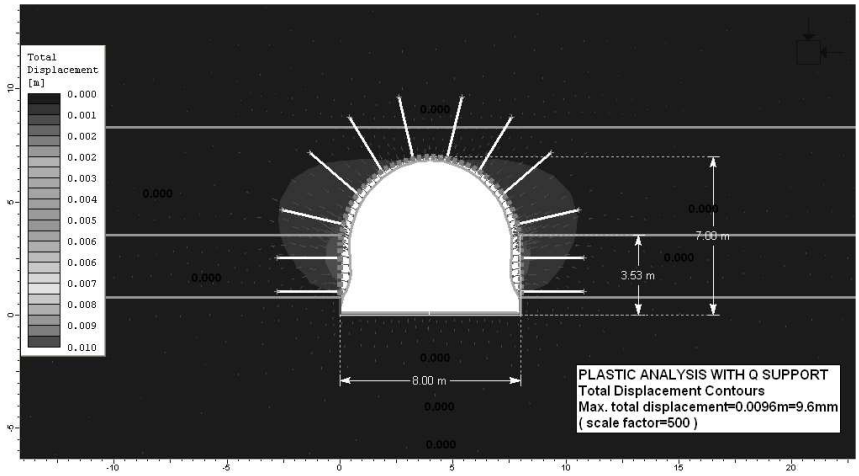

Fig.12 - Displacement profile with displacement vectors and deformed view from plastic analysis with initial trial support from Q System

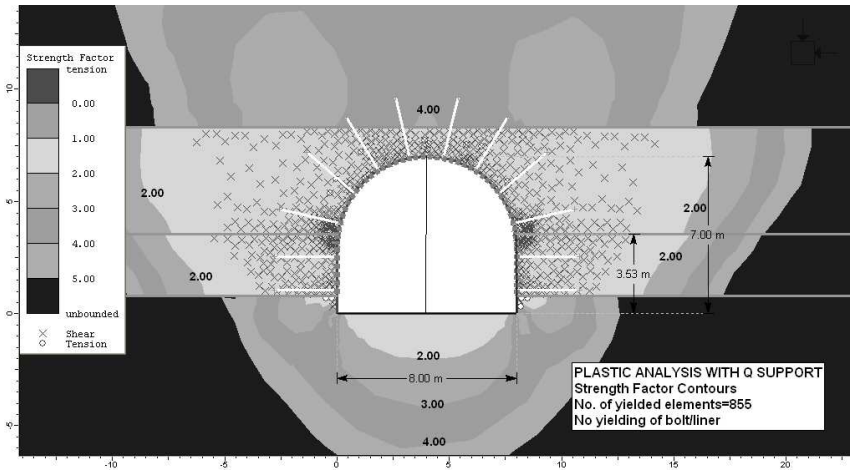

Fig.13 Strength factor contours and yielded elements after plastic analysis with initial trial support from Q System

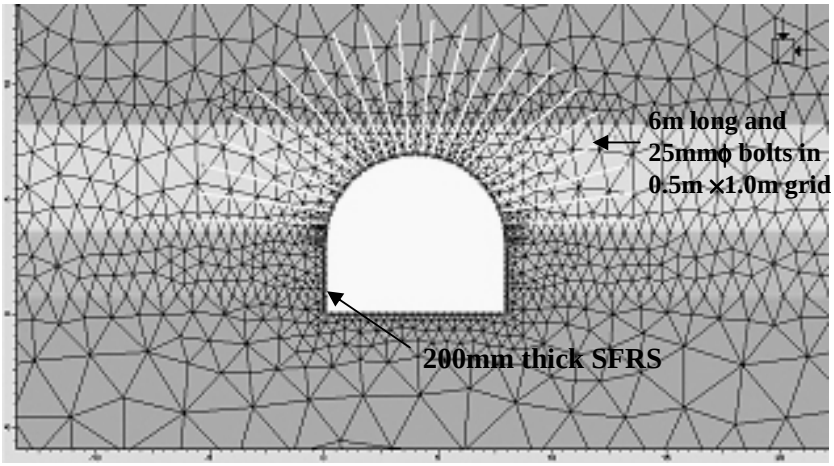

Fig.14 - 2D Finite element model with improvisation over initial trial support

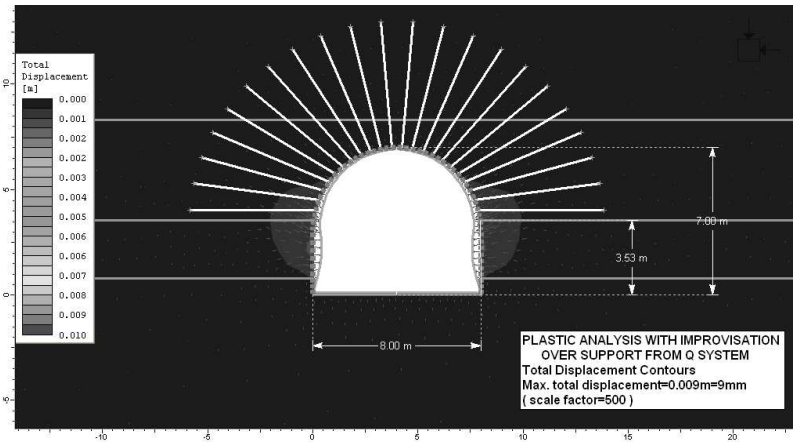

Fig.15 - Displacement profile with displacement vectors and deformed view after plastic analysis with improvised support

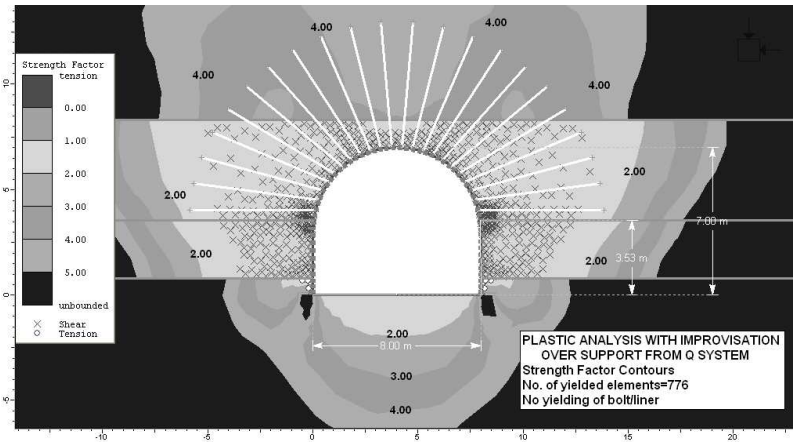

Fig.16 - Strength factor contours and yielded elements after plastic analysis with improvised support

an increase of thickness from 90mm to 200mm and the model is rerun. The enlarged view of present excavation model is shown in Fig. 14. Now from the displacement contours (Fig. 15) though minor improvement over that vide Fig. 12 has taken place, significant improvement has been observed from the strength factor contours (Fig. 16) where apart from reduction in the yield zone as well as number of yielded elements i.e. from 855 yielded elements from initial trial to 776, the yield zone is now well contained within the rock bolt support envelope.

7. RESULTS AND DISCUSSIONS

In the present analysis, the extent of yield zone at crown portion from elastic as well as plastic analysis clearly explains the rock fall already experienced at site from crown section. Since this software deals with small strain and cannot account for large strains associated with collapse of tunnel, in the elastic analysis, the overall displacement profile has been considered and not the magnitude. The deformed profile of the opening

from the plastic analysis with a maximum total displacement of 23mm is worth considering though it needs to be verified from the record of the instrumentation data from site specially in such situations where major portion of the excavation traverses through poor quality rock mass. The properties of amygdaloidal basalt rock mass with red tachylytic injections (red bole) at the crown section and that of green vesicular amygdaloidal basalt in the portion of the side walls have been selected based on RMR rating only. Since alteration of physical state of such type of poor quality rock mass is a continuous process based on weathering, it is advisable to go for staged excavation and apply a thin coat of shotcrete immediately after the weak rock mass is exposed which may help to prevent the progressive deterioration. From the first trial, since the support provided to the section as per guidelines from Q-system has not been sufficiently adequate, second trial with improvised support has been conducted which rendered the excavation stable though it may not be the ideal support from practical and economic point of view.

8. CONCLUSIONS

At the initial stages of excavation, when no experimental or instrumental data is available, support systems to be adopted are purely on recommendations of project geologist based on the site specific geological conditions, guidelines provided in various rock classification systems such as RMR,Q etc. and also from the past experiences gained by engineers and contractors from similar sites. The ratings given for various parameters such as RQD, joint condition, groundwater condition etc. by different geologists for classifying same rock mass can seldom be found to be in agreement which can affect the judgment process for selection of a site specific and cost effective support system . In addition, since rock mass can be defined as matrix of intact rock material with its system of discontinuities, the absence of rock material properties and a suitable criteria to define the failure mechanism of the rock material in any of the classification systems restricts the study of rock-structure interaction under site specific geology and in situ stress scenario. The absence of the record of any stress and displacement /deformation data further adds to the list of unknowns towards understanding the rock mass behaviour.

The authors want to emphasize the need for adoption of a suitable numerical technique at this stage which can help to visualize the probable scenario of rock – structure interaction and also the stabilization of yielding zone using trials with alternative supports. This in turn will enhance the judgment process for selecting support systems for underground excavations and reduce the difference between preliminary and final support to make the process a cost effective one. For major projects with either complex geological features or high in situ stress scenario, in addition to geological classification it is desirable to opt for various instrumentations and geo- technical studies to determine parameters describing behavioral aspects of rock mass and rock materials which are to be further used as input parameters for any numerical analysis. A small investment towards such studies compared to cost of the whole project can result in effective savings in the long run. As excavation progresses, the design needs to be updated from time to time based on surprises and challenges thrown to engineers by mother nature and also based on data from various instrumentations and geo-technical investigations.

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