



Analysis of support system for a conveyance tunnel in the higher himalayan zone: A Case Study on Upper Tamakoshi HEP, Nepal

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Abstract

After excavation, insitu stress conditions are changed which lead deformation due to the stress concentration. For the stability in the excavated tunnel profile, appropriate support system is essential. For the analysis, site specific data are used from Higher Himalayan Region of Nepal. Study is focused on 3 m and 6 m size inverted D Shaped tunnel with three different overburden thickness. For the analysis of support system: Empirical method, Analytical method and Numerical Modeling are performed. Result obtained from the different approaches for three different overburden heights as well as for both size tunnels are compared and finally required support system is recommended. It was found that with increase in size of tunnel there will be significant increase in deformation. Overburden thickness is also playing the vital role in this parameter but size effect is more prominent.

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1. Introduction

Affordable clean energy is the seventh goal of sustainable development goals envisaged by the United Nations [1]. The clean energy comprises the renewable energy i.e. hydropower, wind energy and solar power. In context of Nepal Hydropower is the most economical source of clean energy [2]. However, the hydropower demands many large structures and huge construction works like dams and tunnels. Since the geology of the Nepal is young and fragile [3] construction of tunnel faces many challenges. The foremost challenge is to provide proper support for excavated surface to prevent or limits the deformation and the dilation of rock mass that may lead to collapse. Support system in tunnel is essential to enhance the ability of the rock to be self-supporting. Rock masses are quite strong if progressive failure along planes of low strength is prevented [4].

Basically the support system provides tensile, shear, and/or frictional strength across discontinuities. The shear strength of the discontinuities will always be less

after slippage or separation takes place. For this reason, the support system should be installed as soon as possible after the excavation is made. The discontinuous nature of rock masses permits many possible modes of deformation [4]. Therefore, during analysis of rock support, the primary emphasis should be to guard against the most probable modes of deformations.

The study is focused in the analysis of support system for underground excavation by using the combination of shotcrete reinforced with wire mesh and rock bolts in the HRT of Upper Tamakoshi HEP. To fulfill the objective, analysis from empirical, analytical and finite element based numerical modeling with the help of *Phase²* (version 7.0), developed by rocscience group is applied.

Optimization of the support system is carried from longitudinal displacement profile drawn from the analytical method. Stresses and deformations induced after excavation in the tunnel are observed from the analytical method as well as model created on *Phase²* and finally required support system is proposed.

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2. Literature Review

In our common practice supports systems are: temporary support and permanent support. The temporary supports are installed just after excavation to stop the further deterioration of the rock mass and to provide immediate support to the rock mass for the tunneling works to continue. The Permanent support (like concrete lining) are installed in due course of time after the temporary support to take up the stress induced, including the long term stresses and to limit deformation within the tunnel[4]. The temporary supports such as shotcrete and steel sets sometimes can be considered permanent support if no further support like concrete/steel lining is to be added and the preliminary support take up the entire load and limit deformation. Furthermore, with the advent of technologies like TBM and Shield tunneling required support is installed one at a time functions as permanent and temporary support disappears.

For the analysis, three different methods are basically applicable. They are empirical, analytical and numerical methods.

2.1. Empirical approach

Empirical analysis method relates practical experience gained on previous projects to the conditions anticipated at a proposed site and requires experience as well as engineering judgment. Rock mass classification systems are an integral part of empirical tunnel design and have been successfully applied throughout the world as a unique method for design.

Barton et al. (1974) proposed the Q- system of rock mass classification on the basis of about 200 case histories of tunnels and caverns. The system was initially developed for tunneling cases in hard and jointed rock mass [5], and thus, this system is more often applied in jointed rock mass [6]. However, its application in tunnel support design is limited to an equivalent dimension (De) ranging from 2.5 to 30 and a Q value ranging from 0.1 to 40 [7]. They have defined the rock mass quality Q by the causative factors as in Eq. 1 [8].

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

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

SRF : Stress Reduction Factor

The numerical value of the index Q is very sensitive [9] and varies in logarithmic scale from 0.001 to a maximum of 1000. According to the Q-system, support can be estimated based on Q-values and Span/ESR (after Norwegian Geotechnical Institute, 2013) as shown in the Figure 1 [10].

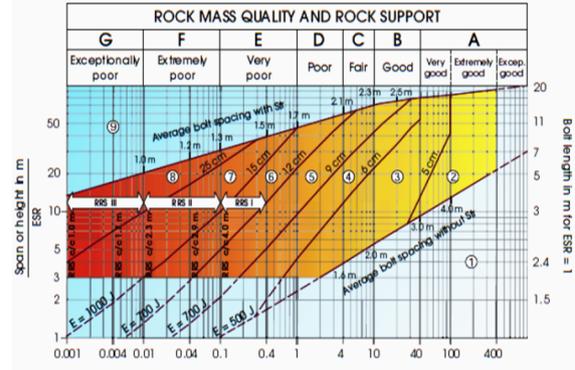


Figure 1: Rock Mass Quality and Rock Support

2.2. Reinforcement Categories

- i Unsupported or spot bolting
- ii Spot bolting
- iii Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm
- iv Fibre reinforced sprayed concrete and bolting 6-9 cm
- v Fibre reinforced sprayed concrete and bolting 9-12 cm
- vi Fibre reinforced sprayed concrete and bolting 12-15 cm + reinforced ribs of sprayed concrete and bolting
- vii Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting
- viii Cast concrete lining
- ix Special evaluation

2.3. Analytical method

The construction material “rock” is a natural, non-homogeneous material. In most cases, rock deformations are of plastic nature, or at least partly elastic or partly plastic [4]. The concept developed for interaction of load-deformation characteristics of rock mass and support system, results in the convergence confinement method (CCM), which is often used in design of support based on idealized uniform stress field and circular opening.

To find out appropriate timing for installation of support against displacement for analysis, it is important to plot longitudinal closure or displacement profile for the tunnel [11]

2.4. Numerical Modeling

Any geotechnical problem associated with tunneling can be analyzed by using different models which represents the in-situ conditions and the problems associated. The methods or models that can be adopted in this context are Mathematical models, Physical model and Numerical models.

All of the above models have their own advantages and disadvantages. The mathematical model, which follows the closed form method of analysis, although the simplest of the three is limited to linear elastic constitutive behavior and thus cannot be used in cases of rock masses as the rocks exhibit plastic behavior and time dependent deformation. The Physical models although in many senses give the best representation of a field problem is mostly used for research purposes because of the time and cost related and moreover the complexity of the process itself.

When the ground exhibits large deformation and/or when the support is very stiff and installed close to the tunnel face, the results obtained with the convergence confinement method may significantly differ from those obtained using 3D numerical modeling. As well as CCM method is limitedly applicable for circular tunnel only. In such conditions, Numerical modeling has been gaining popularity in the analysis of complex problems related to tunneling due to the simplicity of use, the accuracy of representation and the much reduced time and cost. The use of these techniques is now firmly embedded in rock mechanics [4]. Numerical modeling is considered to be the optimum tool for the analysis of support system in tunnels. For the analysis from Numerical Modeling, *Phase*² 7.0 is a powerful 2D elasto-plastic finite element stress analysis program for underground or surface excavations in rock or soil. It can be used for a wide range of engineering projects and includes support design, finite element slope stability, groundwater seepage and probabilistic analysis. Complex, multi-stage models can be easily created and quickly analyzed. For example: tunnels in weak or jointed rock, underground powerhouse caverns, open pit mines and slopes, embankments, and much more. Progressive failure, support interaction and a variety of other problems can be addressed [12]

3. Research Methodology

The methodology can be determined depending upon the parameters under consideration and analysis based

on their respective roles and the associated outputs anticipated. The major parameters under consideration are the stresses induced, strength of the rock mass under consideration, the associated deformation as well.

To fulfill the objectives of research, results are drawn by using following methods as follows:

3.1. Empirical method (Q-system)

The Q-system provides a much wider range of support systems. Hence this is compatible for frequently changing geological parameters like our Nepalese geology.

3.2. Analytical method (CCM)

This method enables calculation of average radial pressure applied to the support by the intersection of two curves representing the radial stresses as a function of radial strain. As well as displacement and stresses in rock masses surrounding an excavation are controlled. Moreover, CCM is applicable for the analysis of longitudinal displacement profile along the tunnel face.

3.3. Numerical Modeling

Finite element analysis based *Phase*², version 7.0, is used for analysis from numerical modeling. Parameters related with rock mass behavior are acquired from Upper Tamakoshi HEP area to simulate the tunnel behavior. Procedure followed for modeling have been summarized as follows:

3.3.1. Preparation of Geometrical model

The geometrical model is prepared with the determination for size of model and opening. Boundary conditions are applied. The model assumes a plastic zone around the excavation due to the failure of rockmass. Plastic zone is represented by the yielded elements of the control volume surrounding the excavation. Discretization of the control volume will be done by the creation of a mesh which is generated automatically by the software *Phase*². For the preparation of the mesh, the six noded triangular elements are used, which is the inbuilt element in *Phase*².

3.3.2. Assigning of Materials properties

In the analysis two distinct regions are considered, one just around the excavation and another far from it, each representing the perfectly elasto-plastic and elastic zones. Hence, Generalized Hoek Brown parameters and constants are taken. After classifying the materials characteristics as Elasto plastic in the *Phase*² model, material properties are assigned

3.3.3. Application of Boundary Condition to the Model

A fixed boundary condition in all direction (i.e. restrained in U_x and U_y) is considered representative of the rockmass condition. Analysis is carried out for varying input parameters such as stress ratio (k), overburden thickness and size of tunnel. The formation of plastic zone, stresses induced and deformation formed are observed from numerical modeling according to the Generalized Hoek Brown failure criteria. Support system is optimized by changing thickness and grade of shotcrete along with the specification of mesh. As well as length and diameter of rockbolts are also changing parameters for the safe and economical support. Tensile strength of steel for rockbolt is generally proposed same for the convenience.

Table 1: Properties of material (Site specific of HRT from Upper Tamakoshi, HEP)

S.No.	Description	Value / Category
1	Rock type	Banded Gneiss
2	Unit weight of rock	2.7g/cm ³
3	Overburden thickness	66.3m - 851.1m
4	Stress Ratio (k)	1.93
5	UCS of rock	101 – 553 kg/cm ²
6	Poisson's ratio for rock	0.2
7	Modulus of elasticity (E) for rock	36600 Mpa
8	RMR value of rock	50
9	Q value of rock	5
10	GSI value of rock	45
11	Material properties of rock, m_b	2
12	s	0.012
13	a	0.5
14	Disturbance factor (D)	0.7
15	Tunnel size	3m and 6m
16	Tunnel shape	Inverted D Shaped

4. Analysis, Result and Discussion

4.1. Empirical Methods

Using geological/geotechnical parameters from Upper Tamakoshi Hydroelectric Project and parameters involved, from empirical approach, support was assessed as per the requirements for the Q-system of rock mass classification provided in Figure 1. Table 3 presents required support for tunnel excavation.

For 3m diameter tunnel, Tunnel Span or Height (m)/ESR value is 1.88 and for 6m diameter tunnel, this value is 3.75. In these both conditions, shotcrete is not required for the tunnel support. In both conditions spacing of rock bolt is suggested between 1.0–2.0 m. Length of rock bolt for 3m diameter tunnel is required 1.5 m long

and for 6m diameter tunnel 2.4 m long rock bolt is required.

4.2. Limitations

Q-system of rock mass classification (after Hoek et al., (1995) only accounts for the Q value and the overburden of the tunnel is not considered. Hence, the above chart only gives the general assessment of the support system and the induced stresses are not truly considered. The chart only gives rough idea about the support system.

4.3. Analytical method

The convergence confinement method has been used for the analytical method as described earlier. Summary of the results is shown in Table 4 and the output plots for tunnel support interaction (as a sample plot) from this method are presented in Figure 2, 3, 4, 5 and 6

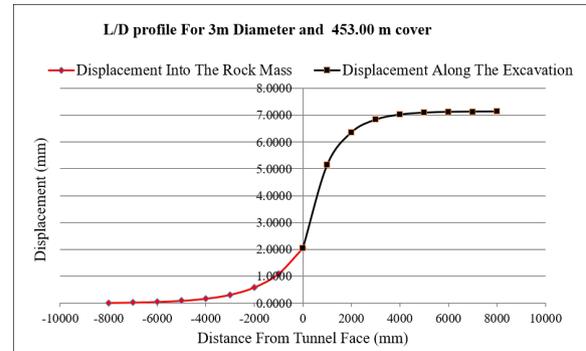


Figure 2: L/D profile For 3m Diameter and 453.00 m cover

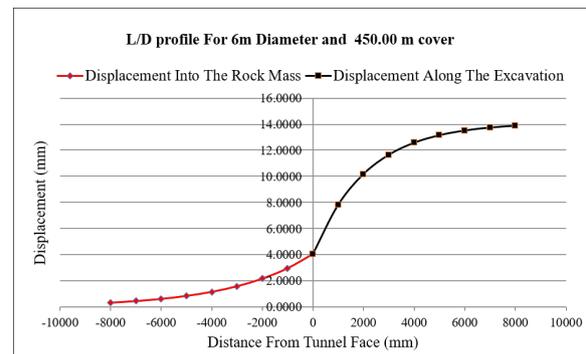


Figure 3: L/D profile For 6m Diameter and 450.00 m cover

4.4. Discussion with analysis on the results/output plots for Analytical Methods

The support interaction analysis for inverted D shaped tunnel of 3m and 6m diameter has been performed for

Table 2: Rock types and rock cover (Overburden) drawn from alignment of HRT, Upper Tamakoshi, HEP

Tunnel Segment	Chainage	Rock type	Average rock cover	Remarks
First	0.000 to 3390.50	Banded Gneiss	440.00 m	Segment first and third are designed considering 450.00 m cover.
Second	3390.50 to 5631.20	Banded Gneiss	851.00 m	To analyze the effect of minimum overburden
Third	5631.20 to 7861.48	Banded Gneiss	450.00 m	Overburden considered for analysis
Fourth	2735.30 to 2792.18	Banded Gneiss	66.00 m	To analyze the effect of minimum overburden

Table 3: Rock types and rock cover (Overburden) drawn from alignment of HRT, Upper Tamakoshi, HEP

Q Value	Excavation Support Ratio(ESR)	Tunnel Span or Height (m)	Tunnel Span or Height (m)/ESR	Spacing of Bolt (m)	Length of Bolt (m)	Rock Type	Remarks
5	1.6	3	1.88	1.0-2.0	1.5 m	C (Fair)	No shotcrete
5	1.6	6	3.75	1.0-2.0	2.4 m	C (Fair)	No shotcrete

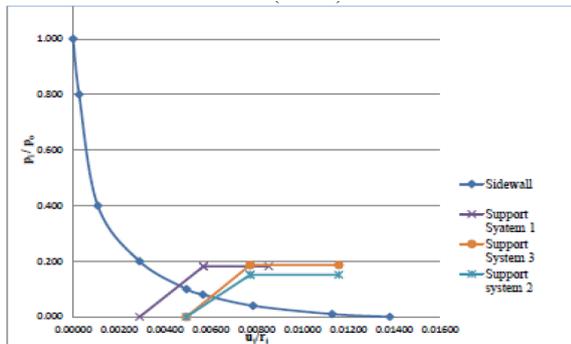


Figure 4: Tunnel Support Interaction Curve For 6m Diameter Tunnel and Overburden Thickness 851.00m (Side wall)

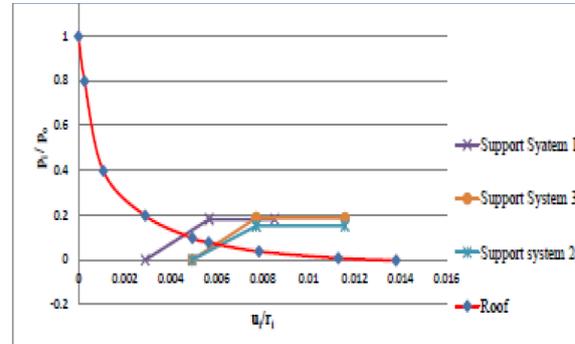


Figure 6: Tunnel Support Interaction Curve For 6m Diameter Tunnel and Overburden Thickness 851.00m (Roof)

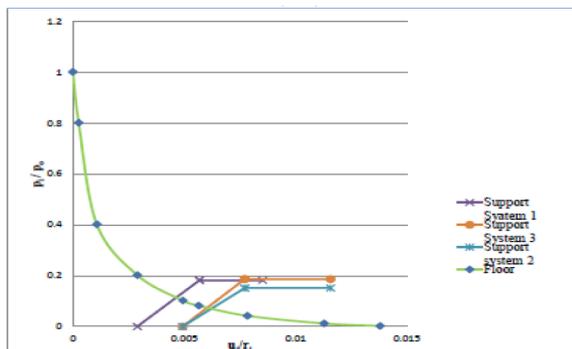


Figure 5: Tunnel Support Interaction Curve For 6m Diameter Tunnel and Overburden Thickness 851.00m (Floor)

three different overburden thickness by considering the equivalent diameters. For the shallow overburden thickness of 69 m for 3m diameter tunnel and 66 m for 6m

diameter tunnel: very little deformations are noticed and support systems is not suggested for this condition. For other conditions, deformations for different stage is calculated. Considering the deformations on the tunnel excavation, the required support system has been designed. For the six different models of analysis, grade of concrete for shotcreting was considered 25mpa, 30mpa and 35mpa with thickness variation. Rockbolts of 20 mm and 25 mm diameter are used. For the different overburden thickness and size of tunnels, three types of support systems, considering support 1, support 2 and support 3 has been used to optimize and only support 1, category has been suggested as a required support system.

Tunnel support interaction curves are plotted (as shown in figure above) from the analytical methods according to the ground conditions for both size of tunnel as well as for different overburden thicknesses. As a sample illustrations, of 6 m diameter tunnel with maximum

Table 4: Summary table of Analysis from Analytical Method

Tunnel Size (m)	Overburden Thickness (m)	Compressive Strength of Shotcrete (Mpa)	Thickness of Shotcrete (mm)	Diameter of Bolt (mm)	Length of Bolt (m)	Spacing of Bolt in meter (In plane and out plane)	Allowed deformation (mm)
3	69.00	25	Support system not recommended due to minimum deformation on fair rock.				
6	66.00	35	Support system not recommended due to minimum deformation on fair rock.				
3	453.00	35	50	25	2.4	1.2	0.248
6	450.00	35	200	25	2.8	1.4	3.760
3	854.00	35	100	25	2.4	1.2	4.661
6	851.00	35	250	25	2.8	1.4	9.266

Table 5: Summary table of support analysis from Numerical Modeling

Tunnel Size (m)	Overburden Thickness (m)	Deformation at Relaxation (m)	Deformation at Support Installed (m)	No of Yielded Elements	No of Yielded Bolts	No of Yielded Liner Elements	Compressive Strength of Shotcrete (Mpa)	Thickness of Shotcrete (mm)	Diameter of Bolt (mm)	Length of Bolt (m)	Spacing of Bolt in meter (In plane and out plane)	Diameter of wire mesh (mm)	Spacing of Wire Mesh in mm
3	69.00	0.0017	0.0018	No	No	No	25	30	20	2.4	1.2	4	15
6	66.00	0.0021	0.0035	No	No	No	25	30	20	2.8	1.4	4	15
3	453.00	0.0091	0.0092	116	No	No	25	50	20	2.4	1.2	4	15
6	450.00	0.0169	0.0176	116	3	No	30	100	25	2.8	1.4	4	15
3	854.00	0.0173	0.039	264	No	No	35	150	20	2.4	1.2	4	15
6	851.00	0.0276	0.0869	246	5	No	35	360	25	2.8	1.4	6	15

overburden thickness is presented separately for roof, side wall and floor.

4.5. Numerical modeling

Figure 7, 8, 9 and 10 shows support capacity plot (thrust vs. moment and thrust vs. shear force) of shotcrete for lining with 25 mpa. The plot is safe for the required factor of safety.

Figure 11 shows the triangular noded discretized model with installation of support (Rockbolts and shotcrete lining) and contours for maximum shear strain and total displacement are shown in Figure 12 and 13. Support system is optimized with trial process being concerned

with stress and deformations. Such that applied support must be stable within the factor of safety.

4.6. Discussion with analysis on results/output plots for Numerical Modeling

According to the procedures mentioned in methodology, only optimized support requirements are compiled as a different option of support for the particular model. For analysis, grade of concrete for shotcrete was ranging from 25mpa to 35mpa with thickness variation. Rokbolts of 20 mm and 25 mm diameter has been used with tensile strength 500 mpa. Wire mesh of 4mm and 6mm diameter as a reinforced material for shotcrete has been used at 15cm spacing of square mesh. 10 cm to

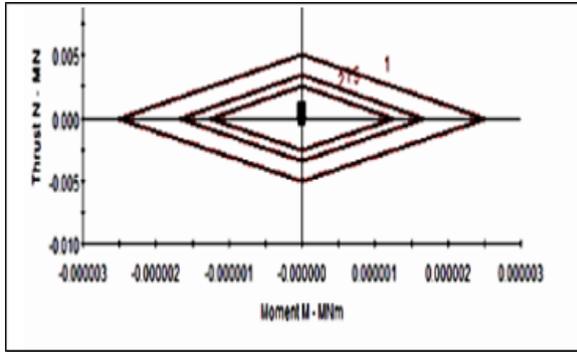


Figure 7: Illustrations of support capacity plots: Thrust Vs Moment

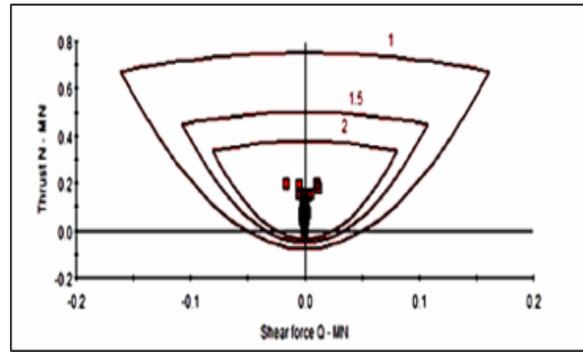


Figure 10: Illustrations of support capacity plots: Thrust Vs Shear force

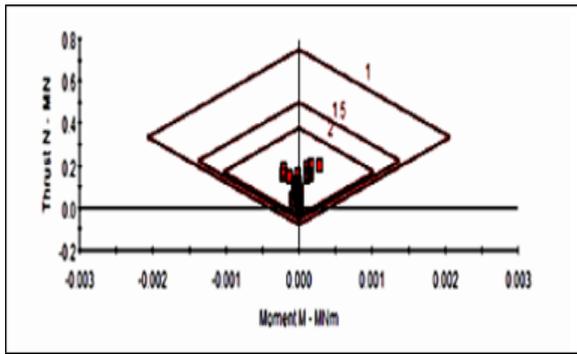


Figure 8: Illustrations of support capacity plots: Thrust Vs Moment

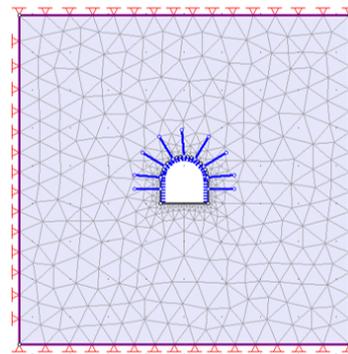


Figure 11: Triangular noded discretized model

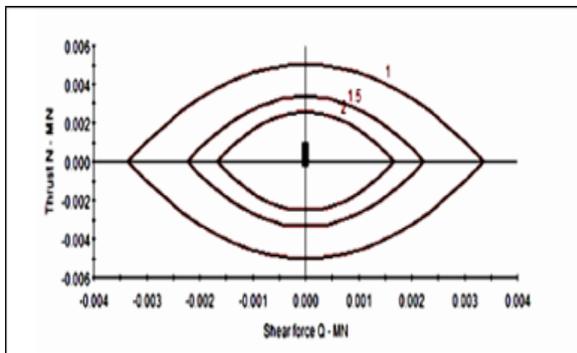


Figure 9: Illustrations of support capacity plots: Thrust Vs Shear force

12 cm thick shotcrete has been applied in a single layer and for the requirement of greater thickness, composite liner of shotcrete has been used. Factor of safety 1.5 is considered for analysis and regarding this value support systems are recommended. For the variation of overburden thickness, significant variation on deformations are observed. Similarly, significant effect on the variation of size are also has observed during analysis.

4.7. Effect of overburden thickness and size of tunnel

For the analysis, overburden thickness of headrace tunnel are grouped in three categories namely: low, medium and high depending upon the thickness of overburden. Tunnel at low overburden thickness: nominal thickness of shotcrete for the support is sufficient for both 3 meter and 6 meter sized tunnel.

For tunnel at medium thickness of overburden: 6m sized tunnel is required higher thickness of shotcrete than 3m sized tunnel . Also rock bolts required for 6m sized tunnel are larger in diameter and length than 3m sized tunnel.

For tunnel at higher thickness of overburden: 6m sized tunnel is required higher thickness of shotcrete, larger size of rock bolts in diameter and length than 3m sized tunnel.

5. Conclusion

This research is focused from the available geotechnical date for Upper Tamakoshi Headrace Tunnel. Analysis for support system considering 3 m and 6 m diameter tunnel is performed. Geometry of the tunnel is inverted

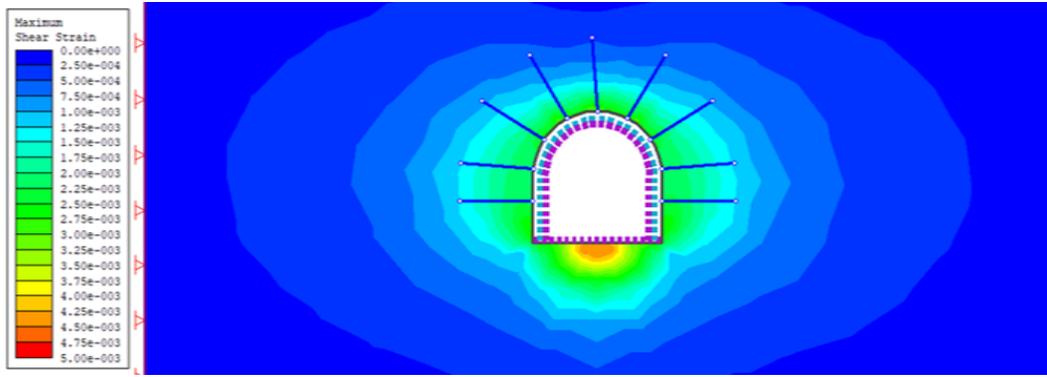


Figure 12: Contours for Maximum Shear Strain

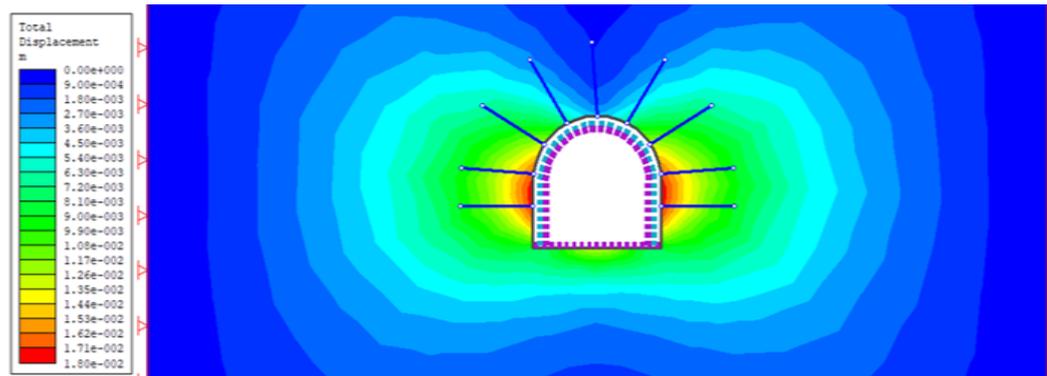


Figure 13: Contours for Total Displacement

D-shaped for both size. For the analysis, three different overburden thickness is considered as: Low, medium and high. Variation of overburden thickness on low to high is more than ten times, but the variation of overburden thickness on medium to high is nearly two times for both sized tunnel.

From the analysis: it seems the effect of increasing size of tunnel is more prominent than the increase in the overburden depth for the tunnel support. Hence while selecting an alignment and optimising the size of tunnel this effect should be considered. Further the use of longitudinal displacement profile seems to provide more safer and economical design for tunnel support.

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