



## STABILITY ANALYSIS AND DESIGN OF ROCK SUPPORT FOR TUNNEL AND CAVERN OF KATHMANDU UNIVERSITY GEO-LAB

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### ABSTRACT

This study focuses on the design and stability analysis of underground structures in the Lesser Himalayan Region of Nepal. The rock support design for such opening depends upon the rock mass strength. In this study a proposed Kathmandu University Geo-Lab tunnel and cavern, which passes through weak rock mass conditions, was used as a case study. Existing empirical and analytical methods for the stability of the proposed tunnel and cavern are used for the estimation of support pressure and design support. A detailed numerical study was carried out in 2D finite element analysis to design the tunnel in such region. The results of analysis showed that the support pressure and deformation can be predicted very well from the numerical analysis.

**Keywords:** Stability analysis, Underground structures, Support system, Numerical modeling

### INTRODUCTION

Nepal has the longest division of the Himalaya occupying the central sector of the Himalayan arc with about 880 kilometers from east to west and has a width ranging from 150 to 250 kilometers. It lies in a highly seismically vulnerable region by virtue of its proximity to the young Himalayan range and the ongoing neo-tectonic activities in the region. The seismicity of the country is attributed to the location in the subduction zone of Indian and Asian tectonic plate [1]. Due to this active tectonic movement, the rock masses in Nepal are fragile and different in their engineering behavior. The major tectonic thrust faults such as main central thrust (MCT) and main boundary thrust (MBT) have significant influence on the high degree of shearing and fracturing to the rock mass. Considerable discrepancies have been found between predicted and actual rock mass conditions, resulting to insignificant cost and time overrun for most of the tunneling projects [2]. High overburden and poor quality of rock masses are the major factors for the stability problems in underground structures in the lesser Himalayan region of Nepal. Tunnel through such weak rock mass may undergo severe squeezing problems and is one of the major areas of concern regarding its stability. Most of hydropower tunnels in this region like tunnels of Kaligandaki-A, Middle Marsyangdi, ModiKhola, Khimti and Chameliya Hydropower Projects underwent severe squeezing problems during their excavation.

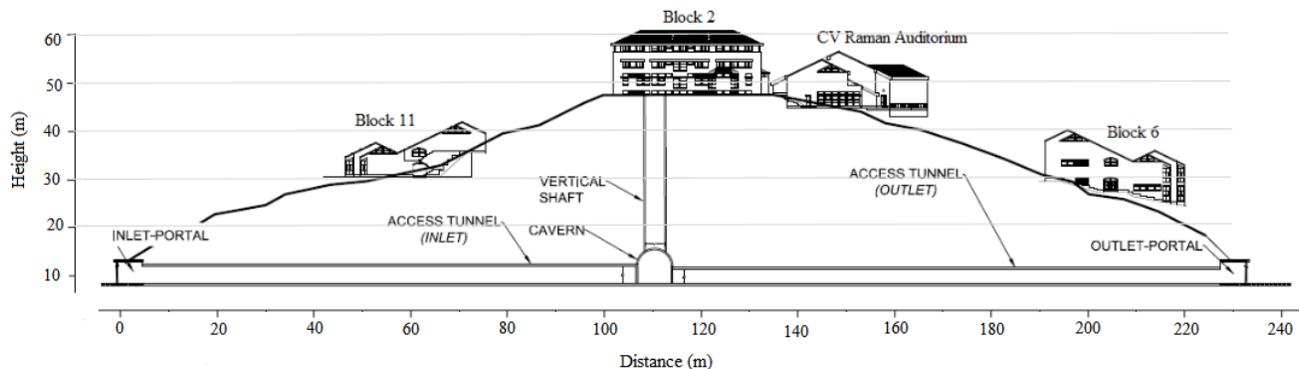
Predicting rock mass quality and analyzing stress induced problems have been a challenging task in this region. Thus extensive researches are to be carried out to incorporate necessary measures to rectify all geological problems. This paper is particularly focused on the design and analysis of Kathmandu University (KU) Geo-Lab tunnel and cavern.



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

### KATHMANDU UNIVERSITY GEO TECHNICAL LAB

A proposed Kathmandu University tunnel and cavern is passing under the hillock of Kathmandu University, Dhulikhel Nepal. The underground cavern lies 40 m below under and in between the Library Block and Block 2. The Cavern is connected by the two inverted D-shaped access tunnels and also a 3.0 m diameter vertical shaft from the top of the hill. Total length of the tunnel including the cavern is 240 m. The width, height and length of the proposed cavern are 7.0 m, 7.0 m and 20.0 m respectively. The dimension of inlet access tunnel is 3.5 m × 3.5 m and that of outlet access tunnel is 2.0 m × 2.0 m respectively as shown in Figure 1.



**Figure 1.** Longitudinal Section of Kathmandu University Tunnel

### *Geology*

The proposed Kathmandu University (KU) Tunnel and Cavern, lies within the Central Nepal Lesser Himalayan region. The main rock type of the area comprises of meta-sandstone with the intercalation of the phyllite of Tistung formation of the Kathmandu complex. Most of the area is covered with colluvial and residual soils whereas some parts have exposed rocks. The rocks that are exposed in different locations within the KU premises are mostly weathered and highly jointed. The bedrock of the area chiefly comprises fine grained grey colored micaceous meta-sandstone and is intercalated with grey colored phyllite.

### *Geotechnical Investigation*

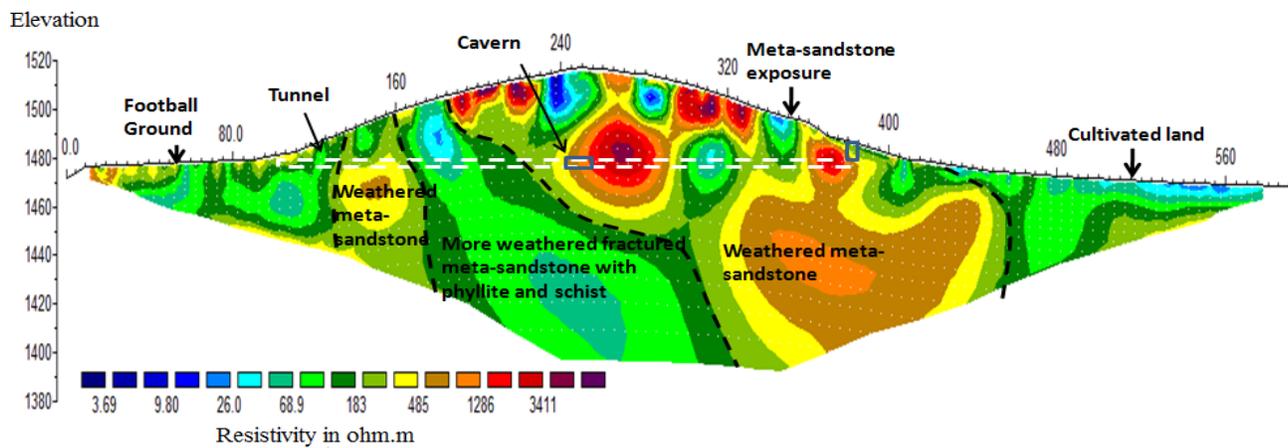
A geophysical investigation had been carried out along the proposed alignment of the tunnel. A 2D Electrical Resistivity Tomography (ERT) survey on the surface above the tunnel alignment was conducted. The survey was so designed that it gave results to a depth of 100 m below surface below the block 2 (Figure 1). The ERT result shows electrical resistivity contours with low and high amplitude with shorter wavelengths in the top part. There may be filling material or the weathered jointed phyllite upto near the inlet of tunnel with low resistivity values (Figure 2). There are medium resistivity contours of 485 Ohm-m up to 50 m from inlet tunnel and this is the part of less weathered meta-sandstone. The section of tunnel between 50 to 90 m from inlet tunnel consists of lower resistivity contours of 50 Ohm-m to 175 Ohm-m (Figure 2) and this is the part of weathered fractured and highly jointed meta-



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

sandstones with phyllite and schists. Normally, the resistivity contours of higher values of over 500 Ohm-m were recorded which suggested the presence of hard stable meta-sandstone. However, patchy low value resistivity contours existing in between cavern and outlet of tunnel may suggest the presence of more weathered fractured jointed rocks at these locations.

The cavern lies 110 m from the inlet tunnel (Figure 1), which is marked by higher resistivity contours of over 500 Ohm-m which suggests presence of hard stable meta-sandstone (Figure 2). The ERT result also showed presence of more uniform and contours of even longer wavelength at about 15m depth from the level of presently proposed tunnel. This suggests more stable and thicker beds with uniform geology.



**Figure 2.** 2D ERT Profile and Subsurface Geology of KU Hillock

### EMPRICAL APPROACH

Barton et al. (1974) proposed the Q- system of rock mass classification on the basis of about 200 case histories of tunnels and caverns. They have defined the rock mass quality Q by the following causative factors [4].

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

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 purpose of the Q-system is preliminary empirical design of support systems for tunnels and cavern. The rock mass quality Q is very sensitive index and its value varies from 0.001 to 1000. Use of the Q-system is specifically recommended for tunnels and caverns with arched roof [4]. Based on the Q-values,



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

the rock mass classified into nine classes are shown in Table 1. Meta-sandstone is the major rock present at the project site and the rock mass is highly fractured and weathered near the inlet of tunnel. At the proposed cavern, the rock mass is quite stable and hard meta-sandstone is found (Fig.2). The detail geotechnical study has not been conducted along the alignment of tunnel and cavern. Therefore, the average rock mass quality is considered for the design calculation. The minimum and maximum value of rock mass is obtained by considering the rock mass as poor and fair. The different parameters considered for estimation of average Q value is as shown in Table 2. It is found that the average Q value of the rock mass is 0.785. From Table 1, the rock mass lies in very poor class.

**Table 1.** Classification of Rock Mass based on Q-values (after Singh et al. 2006)

Q	Classification
0.001-0.01	Exceptionally poor
0.01-0.1	Extremely poor
0.1-1	Very poor
1-4	Poor
4-10	Fair
10-40	Good
40-100	Very Good
100-400	Extremely good
400-1000	Exceptionally good

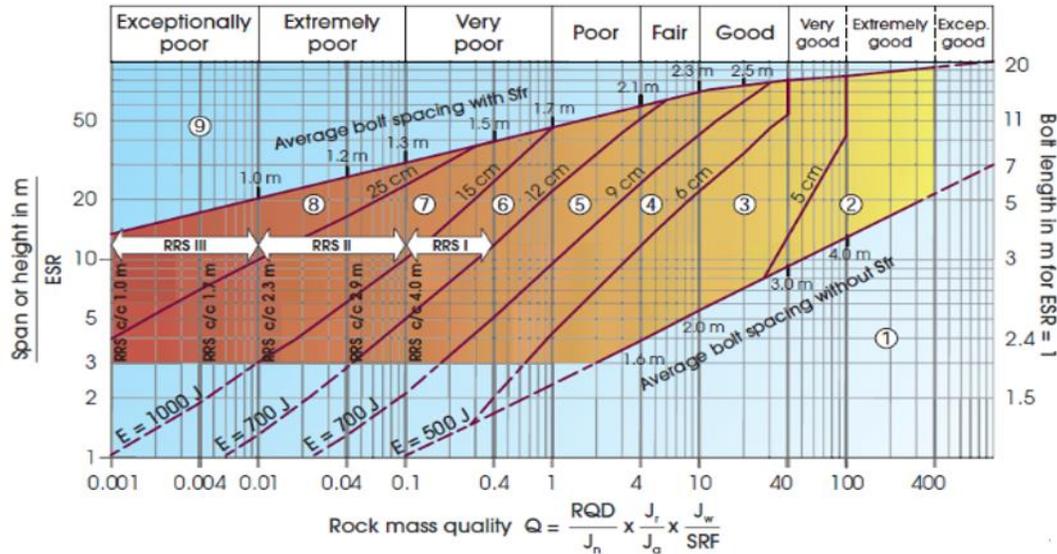
**Table 2.** Estimation of Average Q-value

Poor rock mass			Fair rock mass			average Q
Parameter of Q	Rating	Q <sub>min</sub>	Parameter of Q	Rating	Q <sub>max</sub>	(Q <sub>min</sub> ×Q <sub>max</sub> ) <sup>1/2</sup>
RQD	35	0.38	RQD	65	1.625	0.785
J <sub>n</sub> : three joint sets	9		J <sub>n</sub> : two joint sets	4		
J <sub>r</sub> : smooth planer	1		J <sub>r</sub> : smooth planer	1		
J <sub>a</sub> : slightly altered	2		J <sub>a</sub> : slightly altered	2		
J <sub>w</sub> : dry excavation	1		J <sub>w</sub> : dry excavation	1		
SRF	5		SRF	5		

According to the Q-system [5], the support categories, for 7.0 ×7.0 m cavern, lies in the fourth region (Fig. 3). The required supports are 6 to 9 cm thickness of fiber reinforced sprayed concrete with 3 m long bolts.



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.



**Figure 3.** Support Recommendation based on Q-values and Span/ESR (after Norwegian Geotechnical Institute, 2013)

### ESTIMATION OF ROCK MASS PROPERTIES

Generalized Hoek and Brown failure criteria (2002) has been used for estimation of rock mass properties. Rock mass strength is estimated by equation 2 as suggested by the Hoek et al. (2002) [6].

$$\sigma_{cm} = \sigma_{ci} \frac{(m_b + 4s - a(m_b - 8s)) \left(\frac{m_b}{4} + s\right)^{a-1}}{2(1+a)(2+a)} \quad (2)$$

where,  $\sigma_{cm}$  is the unconfined compressive strength of rock mass in MPa,  $\sigma_{ci}$  is the uniaxial compressive strength of intact rock in MPa,  $m_b$ ,  $s$  and  $a$  are the material constant defined in Hoek-Brown failure criteria (2002).

The rock mass modulus is given by equation 3:

$$E_m \text{ (GPa)} = \left(1 - \frac{D}{2}\right) \sqrt{\frac{\sigma_{ci}}{100}} * 10^{\left(\frac{GSI-10}{40}\right)} \quad (3)$$



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

Geological Strength Index (GSI) is necessary to estimate the rock mass strength. The GSI values are estimated using the mapped rock mass rating (RMR) during subsequent tunnel excavation. RMR can be calculated from Q- system of rock mass classification proposed by Barton et al. (1974). The disturbance factor (D) is assumed to be zero for controlled blasting. The material constant  $m_i$  for Meta-sandstone is taken as 19. Dilation angle is taken as zero for the weak rock mass. In this study, Rock Mass Rating (RMR) and GSI values are calculated using equation 4 and 5 proposed by Barton (1995) and Hoek and Diederichs (2006) respectively [7]. The intact rock strength of Meta-sandstone is taken as 74 MPa [8]. The estimated rock mass properties are summarized in Table 3.

$$RMR = 15 * \log Q + 50 \quad (4)$$

$$GSI = RMR - 5 \quad (5)$$

**Table 3.** Estimated Rock Mass Properties

Description	Meta sandstone
Intact rock strength ( $\sigma_{ci}$ ), MPa	74
Intact Modulus ( $E_i$ ), MPa	18500
Modular ratio	250
Material constant ( $m_i$ )	17
GSI	44
Disturbance factor (D)	0
Poisson's ratio ( $\nu$ )	0.14
Dry unit weight ( $kN/m^3$ )	26.5
Rock Overburden (m)	35
$m_b$	2.301
s	0.002
a	0.509
Cohesion angle (c), MPa	3.964
Friction angle( $\phi$ ), degree	33.27
Dilation angle ( $\psi$ ), degree	0
Tensile strength of rock mass ( $\sigma_{tm}$ ),	-0.064
Uniaxial compressive rock mass	3.124
Modulus of deformation of rock mass	9627.94

### STABILITY OF ASSESSMENT OF KU CAVERN

#### *Empirical approach*

Singh et al. (1992) proposed an empirical approach based on the rock mass classification scheme that is used to assess squeezing of rock. The approach presented by Singh et. al. (1992) is based on 39 case histories with data on rock mass quality index Q (Barton et.al. 1974) and overburden depth. This



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

approach gives a clear demarcation line differentiating squeezing cases from non-squeezing cases. If the overburden is greater than  $350Q^{1/3}$ , squeezing will occur and if overburden is less than  $350Q^{1/3}$ , squeezing will not occur. In this case the existing overburden is 35.0 m while the limiting value of overburden is 323.12 m for the value of  $Q$  is 0.785. Therefore, there is no possibility of squeezing.

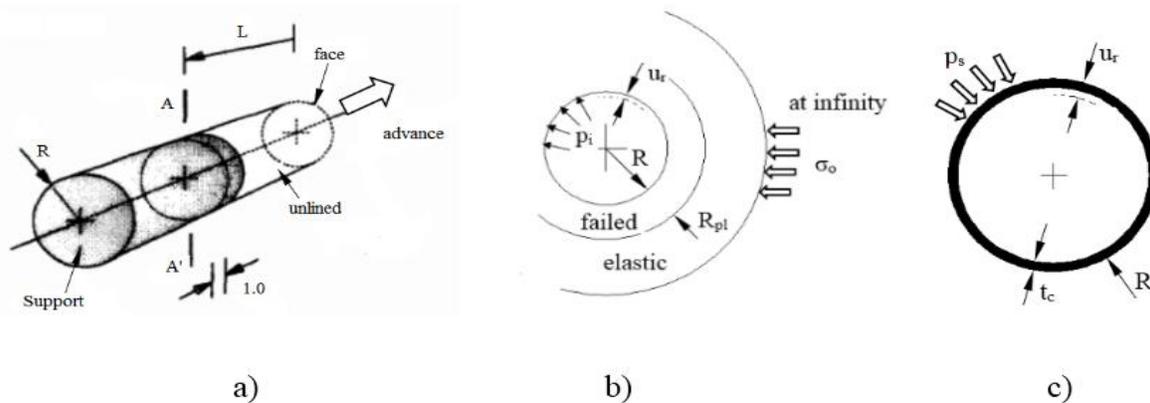
### Analytical Method - Confinement Convergence Method

Convergence- confinement Method (CCM) is a procedure that allows the load imposed on a support installed behind the face of a tunnel to be estimated. When a section of support is installed in the immediate vicinity of the tunnel face, it does not carry the full load to which it will be subjected eventually. A part of the load that is redistributed around the excavation is carried by the face itself.

As the tunnel and face advance (i.e., away from the installed support), the face effect decreases and the support must carry a greater proportion of the load that the face had carried earlier. When the face has moved well way from the support in question, it carries effectively, the full design load [12].

A cylindrical tunnel of radius  $R$  is through a rock-mass that is assumed to be subject initially to a uniform (i.e., hydrostatic) stress field. A circular support is installed at a section A-A' located a distance  $L$  from the face of the tunnel (Figure 4a). The objective of the analysis is to determine the load that the rock-mass will transmit to the support from the time of installation until the time when the face has moved ahead, sufficiently far that the face effect has disappeared.

Hydrostatic far field stress,  $\sigma_o$ , acting on the rock-mass,  $R_{pl}$  represents the extent of the plastic zone that develops around the tunnel,  $u_r$  is the radial deformations occur in a plane perpendicular to the axis of the tunnel and  $p_i$  is the uniform internal pressure at the section (Figure 4b). Similarly,  $t_c$  is the support thickness,  $R$  is the external radius installed at the section A-A',  $p_s$  is the uniform pressure which represents the load transmitted by the rock-mass to the support and  $u_r$  represents the radial displacement which is induced by the load  $p_s$  (Figure 4c).



**Figure 4.** (a) Cylindrical Tunnel of Radius  $R$  driven in the Rock Mass, b) Cross-section of Rock Mass at Section A-A'. c) Cross-section of the Circular Support Installed at Section A-A' (after Torres C & Fairhurst C, 2000)



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

Longitudinal Deformation Curve (LDP), Ground Reaction Curve (GRC) and Support Characteristic Curve (SCC) are three major components of CCM. GRC is the relationship between decreasing internal pressure and increasing radial displacement of tunnel. The relationship depends upon mechanical properties of rock mass and can be obtained from elasto-plastic solution of rock deformation around an excavation [12]. LDP is the graphical representation of the radial displacement that occurs along the axis of unsupported tunnel. Hoek (1999) suggests the following empirical best-fit relationship between radial displacement of the tunnel and distance of the face [12]:

$$\frac{u_r}{u_r^M} = \left[ 1 + \exp\left(\frac{-x/R}{1.10}\right) \right]^{-1.7} \quad (6)$$

where,  $u_r$  is radial displacement in meter,  $u_r^M$  is maximum displacement when internal pressure is zero in meter,  $x$  is the distance from the excavation face in meter and  $R$  is the radius of excavation in meter.

Support characteristic Curve (SCC) is defined as the relationship between increasing pressure on the support. Support characteristic curve using bolt, shotcrete, blocked steel, blocked steel with bolt and blocked steel with shotcrete are constructed.

Support characteristic curve for bolt: The maximum pressure provided by the support system assuming that the bolts are equally spaced in the circumferential direction is given by (Carranza-Torres and Fairhurst, 2000):

$$p_{smax} = \frac{T_{bf}}{S_c * S_l} \quad (7)$$

$$\frac{1}{K_s} = S_c * S_l \left( \frac{4l}{\pi * d^2 * E_s} \right) + Q \quad (8)$$

where,  $p_{smax}$  = maximum pressure provided by support system in MPa,  $T_{bf}$  = ultimate load obtained from pull out test in MN,  $s_c$  = circumferential bolt spacing in meter,  $s_l$  = Longitudinal bolt spacing in meter,  $K_s$  = Elastic stiffness constant,  $Q$  = deformation load constant [m/MN],  $E_s$  = Young's modulus of bolt [MPa],  $d_b$  = bolt diameter [m]

Support characteristic curve for concrete or shotcrete linings:

The maximum pressure provided by the support in case of closed ring concrete or shotcrete is given by (Carranza-Torres and Fairhurst, 2000):



$$p_s^{max} = \frac{\sigma_{cc}}{2} \left[ 1 - \frac{(R - t_c)^2}{R^2} \right] \quad (9)$$

and the elastic stiffness is given by:

$$K_s = \frac{E_c}{(1 - \nu_c)R} \frac{R^2 - (R - t_c)^2}{(1 - 2\nu_c)R^2 + (R - t_c)^2} \quad (10)$$

where,  $\sigma_{cc}$  is the unconfined compressive strength of the shotcrete or concrete in MPa,  $E_c$  is Young's modulus for the concrete or shotcrete in MPa,  $\nu_c$  is Poisson's ratio for shotcrete or concrete,  $t_c$  is the thickness of the ring in m and  $R$  is the external radius of the support equal to that of tunnel in m.

Support capacity curve for blocked steel set support: Considering steel sets spaced a unit length apart in the direction of the tunnel axis and tightened against the rock by wood blocks that are equally spaced circumferential. The maximum pressure that of the system is given by equation 11:

$$p_s^{max} = \frac{3}{2} \frac{\sigma_{ys}}{S R \theta} \frac{A_s I_s}{3 I_s + D A_s [R - (t_B + 0.5D)] (1 - \cos \theta)} \quad (11)$$

The elastic stiffness is given by

$$\frac{1}{k_s} = \frac{S R^2}{E_s A_s} + \frac{S R^4}{E_s I_s} \left[ \frac{\theta(\theta + \sin \theta \cos \theta)}{2 \sin^2 \theta} - 1 \right] + \frac{2S\theta t_B R}{E_B B^2} \quad (12)$$

where,  $B$  is the flange width of the steel set and the side length of the square block in m,  $D$  is the depth of the steel section in m,  $A_s$  is the cross-sectional area of the section in  $m^2$ ,  $I_s$  is the moment of inertia of the section in  $m^4$ ,  $E_s$  is Young's modulus for the steel in MPa,  $\sigma_{ys}$  is the yield strength of the steel in MPa,  $S$  is the steel set spacing along the tunnel axis in m,  $\theta$  is half the angle between blocking points in radians,  $t_s$  is the thickness of the block in m,  $E_s$  is Young's modulus for the block material in MPa and  $R$  is the tunnel radius in meter.

The GRC has been constructed for tunnel and cavern considering same overburden height, equal to 35 m, the overburden pressure is 1 MPa. The GRC and SCC of both tunnel and cavern (Figure 5a) illustrates that the internal pressure is decreased, when the radial displacement increases. The maximum deformations of tunnel and cavern are 0.35 mm and 0.7 mm respectively when internal pressure is zero



(Figure 5). The combination of bolts, shotcrete and steel sets are used for support structure. The details are given in Table 5 and 6. The given combination of support is sufficient for controlling the deformation of tunnel and cavern. The maximum pressures provided by the existing support are 1.33 MPa and 0.61 MPa for the tunnel and the cavern respectively (Figure 5). The maximum pressure that the support experience for both tunnel and cavern is 0.1 MPa respectively (Figure 5) which can be determined where the GRC and SCC cross each other. These are the maximum loads transmitted by the rock mass to the supports. The combination of support can sustain the more load than the demand one.

The maximum displacement of tunnel and cavern are 0.35 mm and 0.7 mm when the excavation face is away to two times diameters of the tunnel and the cavern respectively (Figure 6). It also gives the radial deformation of wall, behind the face, at the installation of support. There is 0.36 mm wall displacement at cavern during the installation of support and 0.24 mm in tunnel (Figure 6). It is clear that there is no effects of advancing tunnel on the rock mass at a distance of 2 times of excavation diameter because the radial displacement is gradually decreases zero at that distance (Figure 6).

**Table 4.** Data used for Plotting GRC

Parameters	Value
Equivalent radius of cavern and tunnel	1.9 m and 3.75m
In-situ Stress ( $\sigma_0$ )	1MPa
$m_i$	19.00
GSI	44.00
$m_b$	2.57
s	1.98E-03
a	0.50
$\sigma_{ci}$ (MPa)	15

**Table 5.** Data used for Plotting SCC with Bolts

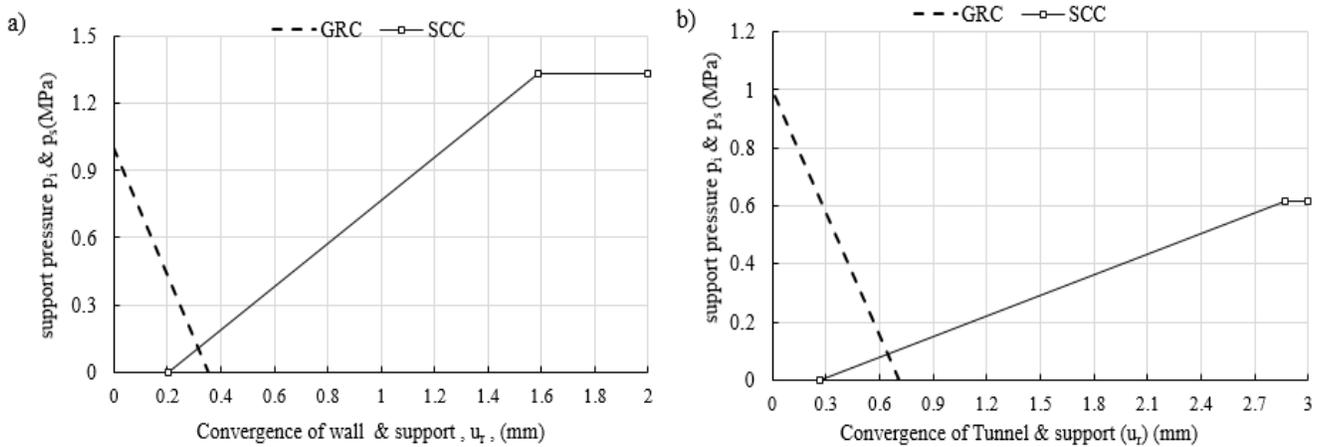
Parameters	Value
Bolt diameter	25 mm
Ultimate load	0.1 MN
Deformation load constant	0.03 m/MN
Young Modulus	210 GPa
Longitudinal bolt spacing	1.5 m
Bolt Length	3 m



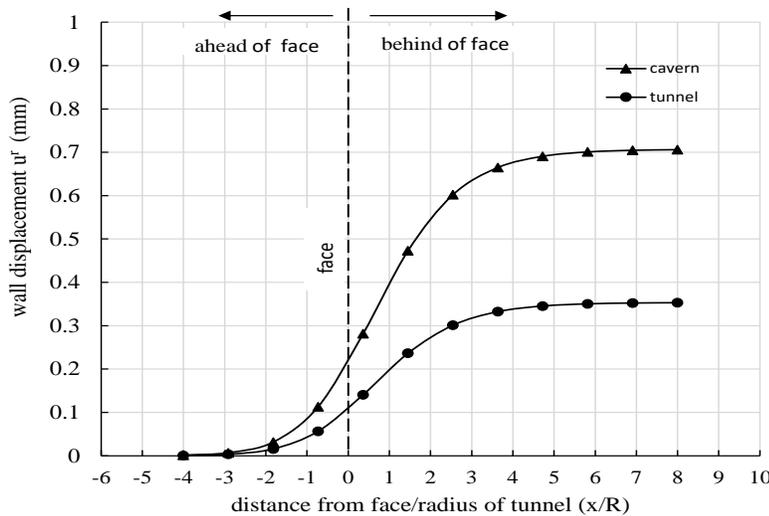
Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

**Table 6.** Data used for Plotting SCC with Concrete Linings

Parameters	Value
Unconfined compressive strength of concrete, $\sigma_{cc}$	25 MPa
Poisson ratio of concrete	0.15
Steel Fiber Reinforced Shotcrete (SFRS), $t_c$	100 mm



**Figure 5.** Ground Reaction Curves and Support Capacity Curves a) Tunnel b) Cavern



**Figure 6.** Longitudinal Deformation Profile of Tunnel and Cavern



### NUMERICAL APPROACH

The numerical analysis has been performed on RS<sup>2</sup>, a finite element program for soil and rock applications, developed by Rocscience Inc.. It is used for 2-dimensional analysis and design of underground excavations and other geotechnical area. It uses both Mohr-Coulomb and Generalized Hoek-Brown failure criteria and others for material model [13]. The rock mass is modeled as strain softening case with the generalized Hoek-Brown failure criteria. The residual strength parameters are taken half of the peak values in strain-softening case [14]. For tunnel support shotcrete lining is modelled as an elastic material. The rock mass strength and properties are chosen from the estimated values given in Table 3.

The vertical stress is linearly increases with the depth due to its overburden weight and is estimated by:

$$\sigma_v = \gamma z \quad (13)$$

where,  $\sigma_v$  is the vertical stress in MPa,  $\gamma$  is the unit weight of rock in MN/m<sup>3</sup> and  $z$  is the depth of overburden in meter.

It is more difficult to estimate the horizontal stress at shallow depths, leading to hydrostatic state in a deep environment. It is obtained from the equation suggested by Sheorey et al. (2001) and is given by [14];

$$\sigma_h = \frac{\nu}{1 - \nu} \sigma_v + \frac{\beta E_m G}{1 - \nu} (H + 100) \quad (14)$$

where,  $\beta = 8 \times 10^{-6}/^\circ\text{C}$  (coefficient of linear thermal expansion),  $G = 0.024^\circ\text{C}/\text{m}$  (geothermal gradient),  $\nu$  is the Poisson's ratio,  $E_m$  is the deformation modulus of rock mass, MPa.

By using the equations (13) and (14), the vertical stress is 0.92 MPa and the horizontal stress is 3 MPa.

In this study full face tunnel excavation has been considered by drill and blast method. A plane strain model has been developed in RS<sup>2</sup> (Fig.9), and that relaxes an internal pressure on the tunnel boundary from a value equal to the applied in-situ stress to zero. The maximum wall displacement and radius of the plastic zone far from the excavation face respectively are determined in final stage with zero internal pressure inside the excavation. In the model, the results from the last stage are used since there is zero internal pressure in this stage. Then the internal pressure is determined in which the excavation is yielded prior to the installation of support. The displacement is determined by knowing the maximum displacement and radius of plastic zone far from the excavation face. After determination of the internal pressure, the support is installed in the last stage of model where internal pressure is zero. The properties of rock support used for numerical study are given in Table 7.

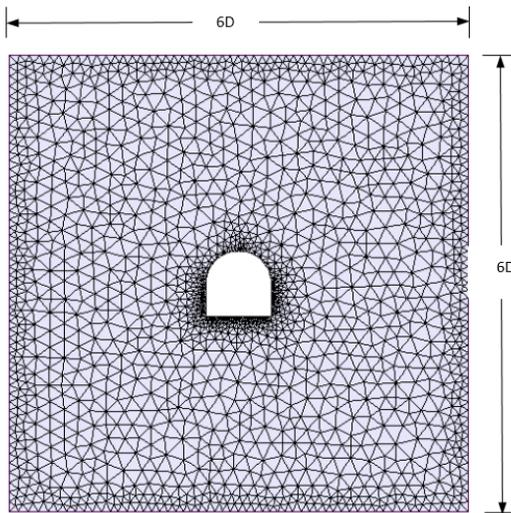


**Table 7.** Estimated Properties of Shotcrete and Steel Rib

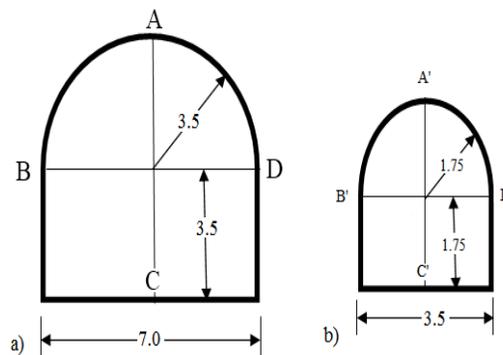
Parameters	Shotcrete	Steel rib
Young's Modulus (GPa)	30	200
Poisson's ratio	0.15	0.25
Cross-sectional area (m <sup>2</sup> )		0.0247
Section depth (m)	0.1	0.106
Moment of Inertia (m <sup>4</sup> )		4.7 × 10 <sup>-6</sup>

An inverted- D shaped tunnel, 3.5 × 3.5 m, and inverted- D shaped cavern of 7.0 × 7.0 m (Figure 8) were modeled without support and with support. The geometry of the model was taken as 6 times the equivalent tunnel diameter and restraint along x- and y- direction (Figure 7) for both tunnel and cavern. The thickness of model is 1 m. The model consists of 3381 six noded triangular elements and 7075 nodes for tunnel and 3308 six noded triangular elements and 6918 nodes for cavern. The numerical results obtained from 2D analysis are shown in Table 7 and 8 for tunnel and cavern respectively. The stresses and displacements observed in crown, walls and invert of tunnel and cavern are represented as A', B', C', D' and A, B, C, D respectively (Fig.8).

In both tunnel and cavern, the induced stress at invert, is more than that of walls and crown before the support installation (Figures 9(a) & 11(a)). The induced stresses are given in Table 7 and 8 for tunnel and cavern respectively.



**Figure 7.** Finite Element 2D Model Developed in RS2



**Figure 8.** Inverted-D Shaped a) Cavern b) Tunnel



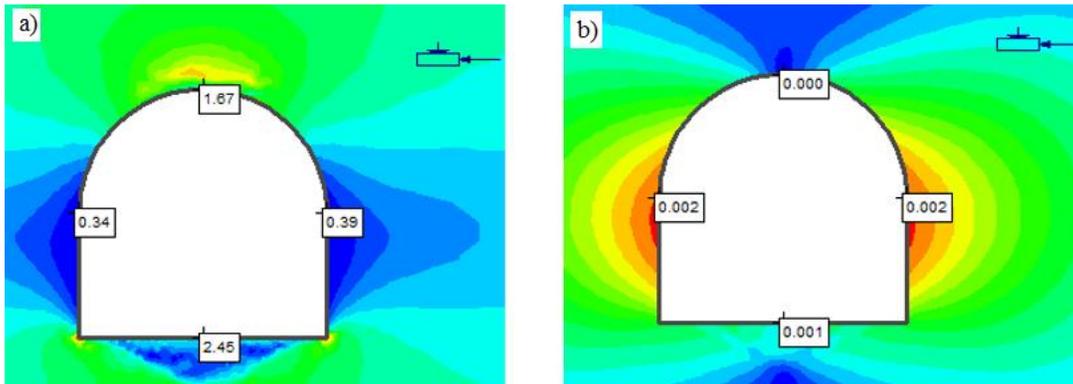
Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

**Table 7.** Stresses and Displacements of Tunnel Before and After Support Installation in 2D Analysis

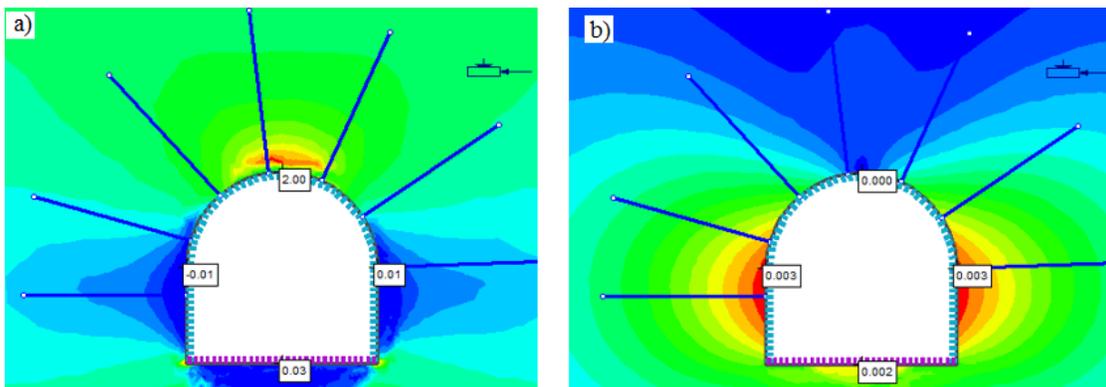
Description	Stress (Sigma1, MPa)				Total Displacement (m)			
	A'	B'	C'	D'	A'	B'	C'	D'
Before Support Installation	1.67	0.34	2.45	0.39	0	0.002	0.001	0.002
After Support Installation	2.0	0.01	0.03	0.01	0	0.003	0.002	0.003

**Table 8.** Stresses and Displacements of Cavern Before and After Support Installation in 2D Analysis

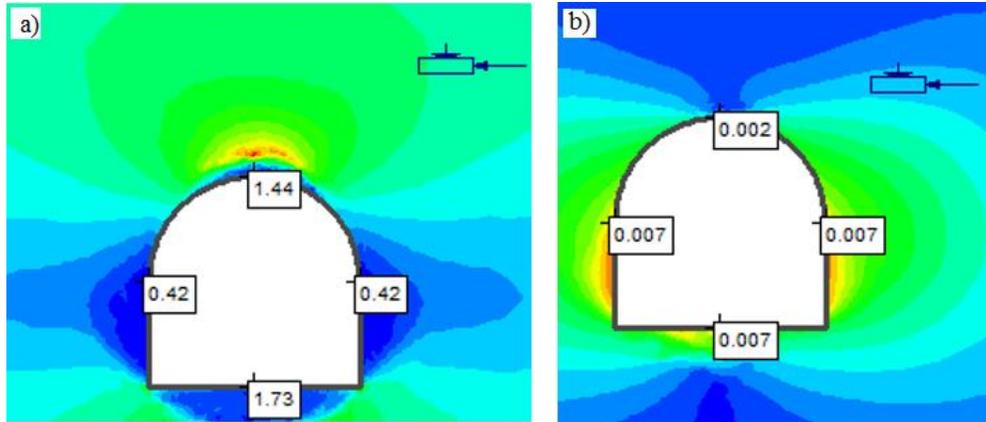
Description	Stress (Sigma1, MPa)				Total Displacement (m)			
	A	B	C	D	A	B	C	D
Before Support Installation	1.44	0.42	1.73	0.42	0.002	0.007	0.007	0.007
After Support Installation	1.47	0.35	0.49	0.43	0.002	0.007	0.007	0.013



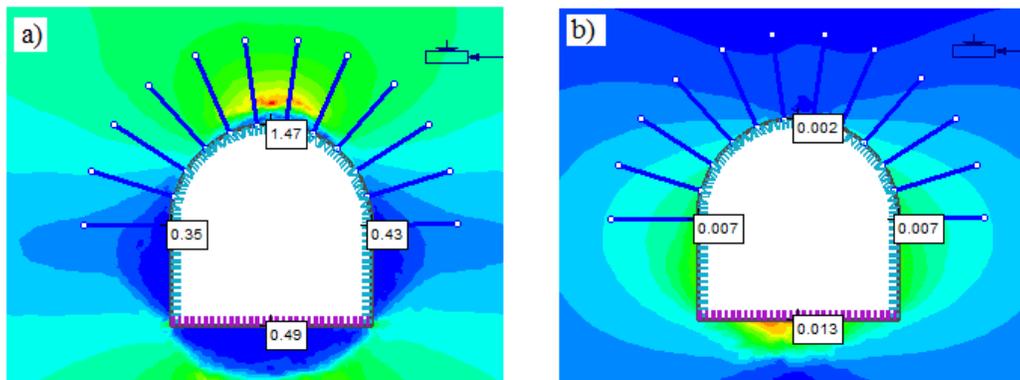
**Figure 9.** a) Stresses and b) Total Displacements Around the Tunnel Before Support Installation



**Figure 10.** a) Stresses and b) Total Displacements Around the Tunnel After Support Installation



**Figure 11.** a) Stresses and b) Displacement Around the Cavern Before Support Installation



**Figure 12.** a) Stresses and b) Displacement Around the Cavern After Support Installation

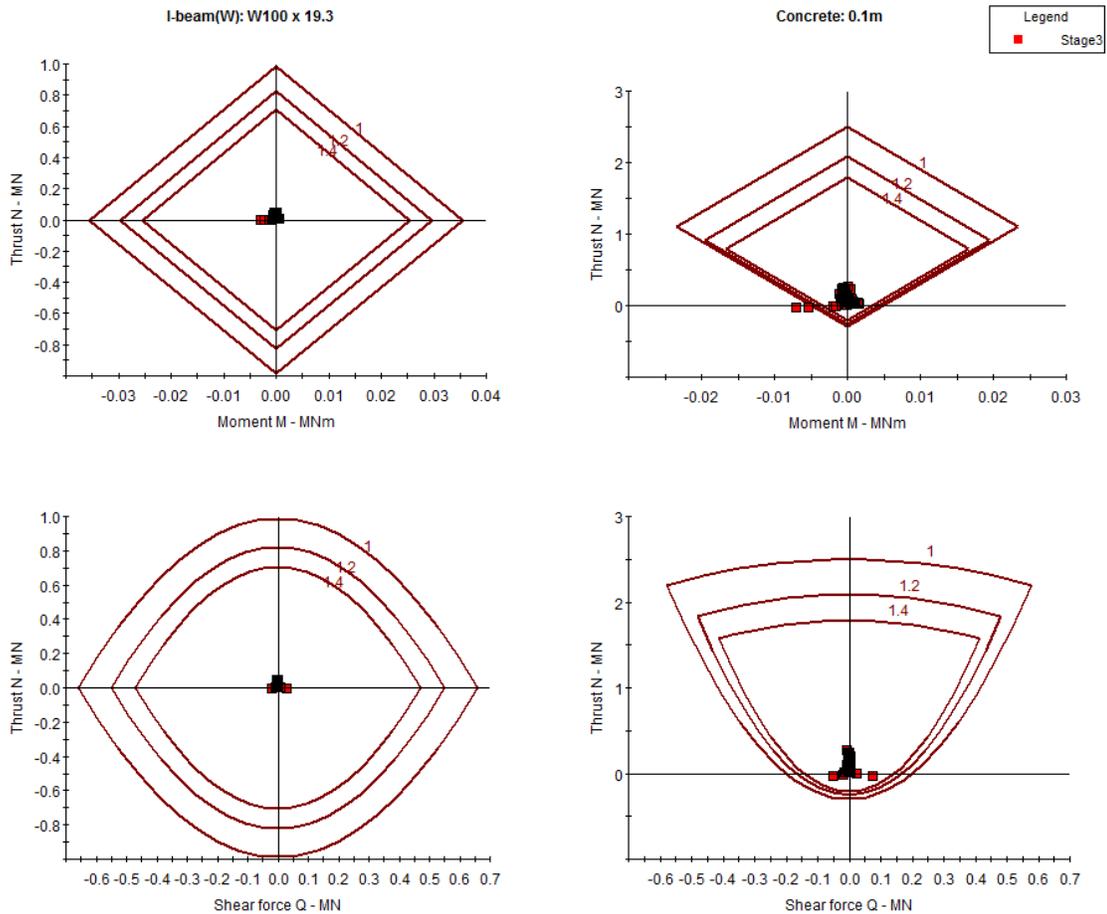
The maximum displacement of tunnel is 2 mm before the application of support and it is increased to 3 mm after application of support (Figures 9(b) & 11(b)). Similarly, the maximum displacement of cavern is increased from 7 mm to 13 mm when support is installed in invert (Figures 11(b) & 12(b)).

The induced axial thrust in the concrete lining is below the allowable axial force, shown in Table 9. It is clear that the provided supports for the tunnel and cavern are sufficient. In invert for both tunnel and cavern, only shotcrete is applied but in wall and crown for both steel ribs is inserted in shotcrete. In case of the tunnel, the thickness of concrete lining is 100 mm and induced axial force is 0.69 MN, the allowable axial thrust is 3 MN. Similarly, for the cavern the concrete thickness is 100 mm and the induced maximum axial force is 0.527 MN, the allowable is 3 MN, Table 9. Therefore, there will be no cracking in the concrete lining. The steel sets are placed each 1.0m spacing. In the numerical analysis, the bolts and liner are not yielded. From the numerical analysis, the given support, combination of bolts, reinforced concrete with steel sets is satisfactory. Support capacity diagrams (Figures 13 & 14) for tunnel and cavern respectively are self-generated by the program. It is the capacity envelopes of axial force, shear force and bending moment which gives the factor of safety of concrete liner and steel sets. The



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

computed liner values fall inside an envelope which means that they have a factor of safety greater than the envelope value. It is concluded that factor of safety of the liner exceeds the design factor of safety.



**Figure 13.** Support Capacity Curve for Tunnel Lining

**Table 9.** Induced Internal Forces on the Tunnel and Cavern Lining

	Actual support		AF (MN)		SF (MN)		BM (MNm)	
	Concrete thk. (mm)	Allowable AF (MN)	Max.	Min.	Max.	Min.	Max.	Min.
Tunnel	100	3	0.69	-0.047	0.107	-0.07	0.004	-0.016
Cavern	100	3	0.527	-0.07	0.043	-0.027	0.007	-0.014

*Note: AF, Axial force; SF, Shear force; BM, Bending moment*

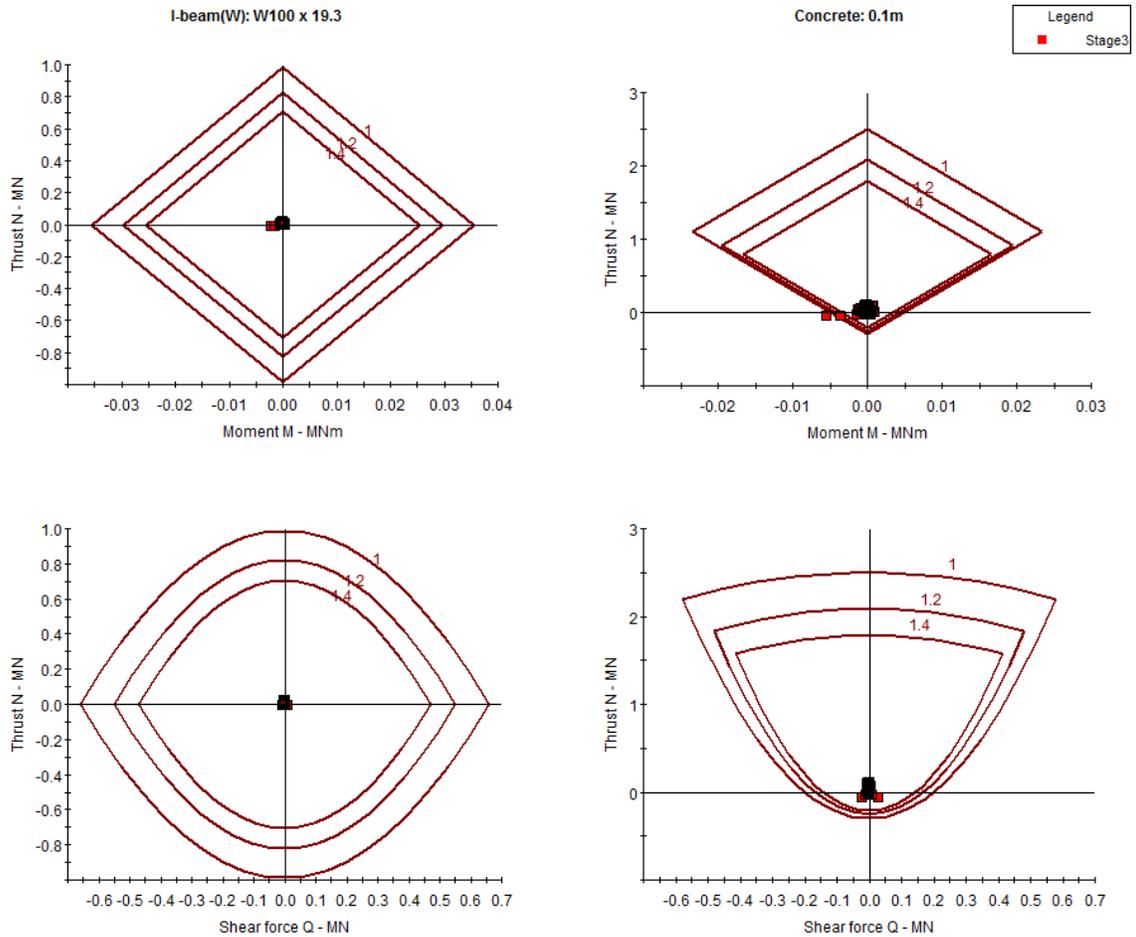


Figure 14. Support Capacity Curve for Cavern Lining

## CONCLUSION

The proposed Kathmandu University Geo lab tunnel and cavern was design and analyzed using the empirical methods, analytical methods and finite element analysis. The tunnel passes through the highly fractured, weathered and weak rock mass. The support pressure and rock deformation were accessed by all these methods and led to the following conclusions:

1. The empirical methods are useful to predict the stability of underground opening in terms of squeezing. The tunnel and carven will not go excessive deformation because the existing rock overburden is quite below the limiting value of rock overburden as per Singh et al. (1992).
2. Convergence –confinement method gives clear idea of ground reaction after excavation and support installation behind the face of opening. The maximum radial displacement of tunnel and cavern are 0.35 mm and 0.7 mm when the face is far from the two times tunnel diameter it becomes zero at two times diameter of tunnel ahead from the face (Figure 6). By knowing the



Khadka & Maskey, Vol. 13, No. I, October 2017, pp 1-19.

maximum radial tunnel deformation, it would be easy to design the support. The existing support stiffness is sufficient for such deformation.

3. The maximum tunnel wall deformation is 3 mm from the numerical modeling and at invert it is 2 mm, for cavern it is 7 mm in the wall and in invert it is 13 mm after installation of support. There is no uniform deformation in opening. Analytical method assumed circular opening and radial deformation are same throughout the opening. Therefore, numerical modeling is necessary to computer tunnel deformation.
4. As there was high deformation in cavern and tunnel compares the analytical methods, new combination of supports is proposed to the tunnel and cavern with shotcrete liner with bolts.
5. The bending moments and shear forces would be higher at corner of tunnel and cavern (Figure 13 & 14), high stresses are induced, than crown, invert and walls. The new proposed concrete liner would be sufficient. But it would be good idea to increase the thickness of concrete lining at the corner of wall and invert.
6. For design and stability analysis of underground structures in such weak geology, besides the existing empirical and analytical approach numerical modeling would give clear understanding to design rock support.

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