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A New Approach to Assess Rock Reinforcement in Underground Openings Located in Seismic areas - A Case Study

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ABSTRACT

The performance of underground structures can be adversely affected during earthquake events. These include rail, road, and hydropower tunnels & large underground caverns such as powerhouses etc. Several cases have been reported internationally on damages to underground structures during major earthquakes. The damages to underground structures include instability at the portals, shearing along an active fault intersecting the tunnel, damage in concrete lining due to vibration and rock mass instability in general. Numerical experiments with earthquake loadings have indicated that joints in a competent rock mass model act as waveguides accentuating damage to tunnel support. Many researchers have reviewed the damage cases and have suggested empirical techniques that can be used to assess the risk of damage versus the magnitude of the earthquakes. Their data shows that for earthquakes with magnitude larger than 6 on the Richter scale and with distance from epicentre less than 20 km the tunnels might experience instabilities due to rock mass damage. In this paper a new approach is presented for assessing rock reinforcement in tunnels in seismic areas with an example from a large underground powerhouse in the Himalayas

There exist rock support design methods based on the experiences from rock bursts in deep underground mines. In this manuscript, it was assumed that earthquakes and rock bursts which are causing similar level of dynamic loading to the underground openings can lead to similar type of failure mechanism. After that, a relevant rock support measure can be designed for the identified failure mechanism. A simplified designed method was proposed which follows those steps. A case study from a large cavern in the Himalayas is used for implementing the design approach. The results coinciding well with the detailed numerical modelling results.

Keywords: Tunnel; Earthquake; Stability; Seismic loading; Faults; Rock bolt; Anchors

1. INTRODUCTION

It is a well-known fact that underground structures are less vulnerable to earthquakes than surface structures. However, several cases of underground structures with damages have been reported during earthquake events around the globe such as 1995 Kobe, Japan, the 1999 Chi-Chi, Taiwan, the 1999 Kocaeli, Turkey and the 2008 Wenchuan China, (Aydan et al. 2010).

Four major mechanisms of instability (damage) due to earthquakes have been identified. These include:

- Shearing along active faults
- Damage of concrete lining or thick shotcrete

- Damage at the portal areas of tunnel (slopes)
- Rock mass failure (rock fall)

Many authors have combined the damage of the concrete lining (or thick shotcrete) with rock mass failure and denoted them in one category of damage caused by the ground shaking. For example, Ayden et al. (2010) denoted these as shaking induced damage.

Dowling and Rozen (1978) and Askura and Sato (1998) have collected historical data of underground structure damages during earthquakes. Their data indicates that tunnels do not undergo damages until the ground surface acceleration reach 0.19g. Power et al. (1998) have summarized all the available case histories with underground damages and earthquakes and have excluded data without exact mechanism of failure. Their data indicates that with the earthquake magnitude between 6.4 to 8.0 the peak ground acceleration required to cause damages should be above 0.2g.

Ayden et al. (2010) compiled case histories of damages in tunnels caused by earthquakes and developed a data base for classification of all the categories of the damages (Table 1 and Fig. 1). Based on their analysis they have postulated that if the magnitude of earthquake is larger than 6 and the distance between epicentre and tunnel is less than 20 km then the earthquake can cause damage in the rock mass. However, for extra strong earthquakes with magnitude above 8, damages in tunnels can occur even up to distance of 100 km. It may be noted that for some recurring earthquakes of even smaller magnitudes (M ~ 2 to 5) within a range of 100 km, may also cause gradual development of support pressure around the underground opening located in the proximity of thick shear or fault zone. In such cases the strains may keep accumulating after each earthquake shock along the fault/ shear zone in seismic areas. Extra rock support pressures in addition to those suggested by Barton (1984) in seismic areas may be warranted. Goel et al, 2013 provide a useful overview on behaviour of underground openings subjected to dynamic loading due to earthquake and blast/explosion. Mitra and Singh (1997) also provided useful insight into the long-term behaviour of large powerhouse cavern by analyzing data for over ten-year period for Chhibro underground powerhouse complex housed in dolomitic limestones of seismically active region of Lesser Himalayas.

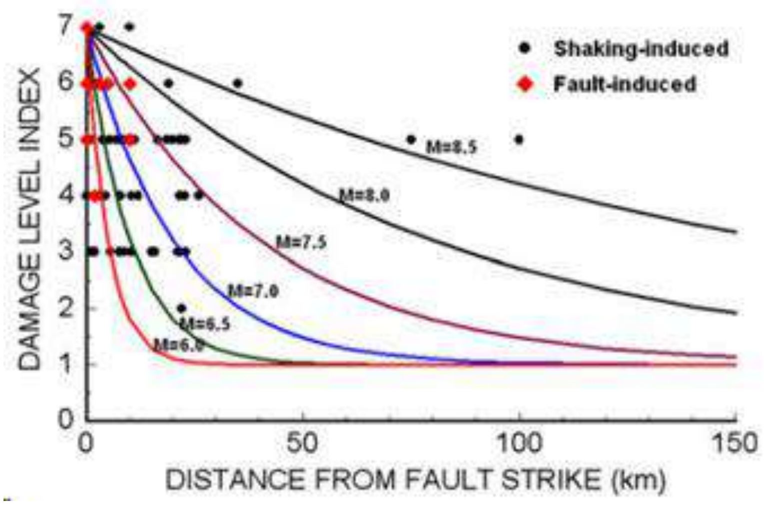
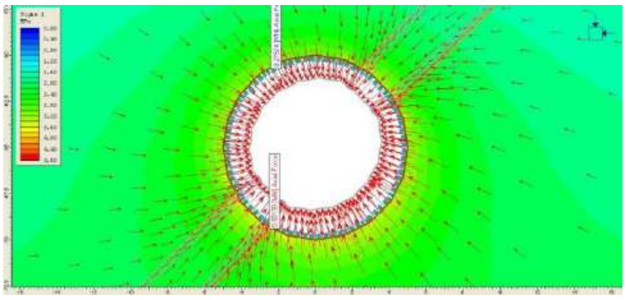


Fig. 1 - Relation between distance from surface trace of the fault and damage level index (DLI) (Aydan et al., 2010)

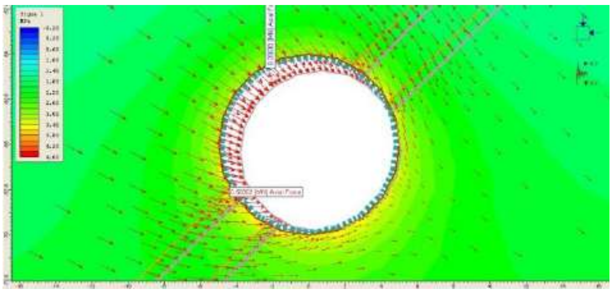
Table 1 - Earthquake-induced damage level index (DLI) for underground structures with considering of support member (Ayden et al., 2010)

Damage level index	Remarks
1	No cracking of concrete lining and shotcrete, no plastic deformation of rock bolts and steel ribs, no invert heaving
2	Hair cracking of concrete lining and shotcrete, non-noticeable deformation of rock bolt platens and steel ribs, no invert heaving
3	Visible cracking of concrete lining, shotcrete, noticeable plastic deformation of rock bolt platens and steel ribs, slight invert heaving
4	Exfoliation of concrete lining and shotcrete, noticeable bending deformation of rock bolt platens and steel ribs, invert heaving; however, it is structurally stable
5	Spalling of concrete lining and shotcrete, considerable plastic deformation of rock bolt platens, bending of steel ribs and invert heaving. It is structurally problematic and requires repairs and reinforcement
6	Collapse of concrete lining and shotcrete, extreme deformation of rock bolt platens, rupturing rock bolts, buckling of steel ribs, buckling, and rupturing of invert, collapse of blocks of ground from roof and shoulders. It is structurally unstable and requires immediate repairs and reinforcement
7	Complete closure of the section by failed surrounding ground. Crushing of concrete lining and shotcrete, rupturing of rock bolts, twisted steel ribs and extreme heaving of invert. Underground openings are to be either abandoned or re-excavated with extreme precautions.

Hashash et al. (2001) proposed a design technique for tunnels against earthquake loads in soil and weak rocks. In such type of tunnels, the rock support mainly consists of thick lining, which can be loaded due to their higher stiffness compared to the surrounding rock mass. Bhasin and Pabst (2014) have performed numerical experiments to show that joints in competent rock masses act as stress collectors or waveguides accentuating damage to tunnel support. Their experiments (Fig. 2) in jointed hard rocks showed that for various earthquake loading conditions the maximum axial force on the tunnel lining occurred at the intersection between the joints and the lining causing damage to the tunnel support. For weak rock masses their numerical experiments indicated that maximum axial force on the lining did not always occur at the intersection between the joint and lining because weak rocks are able to deform independent of joints during earthquake loadings.



Seismic loading h=0.3



Seismic loading v=-0.3

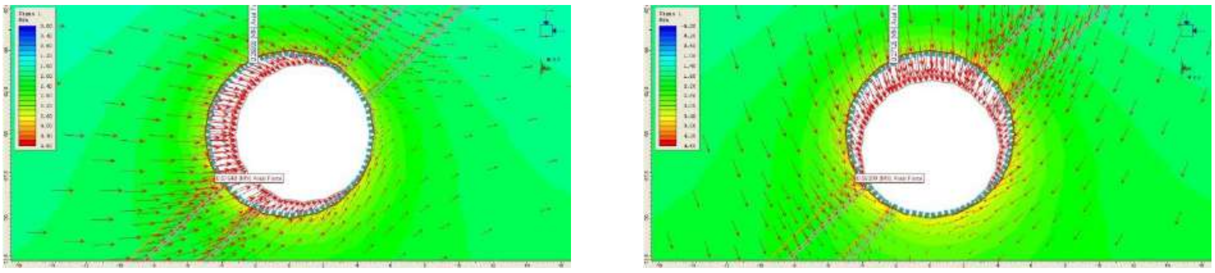


Fig. 2- Effect of two parallel joints in competent rock during static & seismic loading. Note how the joints act as waveguides and stress collectors (right figures) when seismic loading is applied (Bhasin and Pabst, 2014)

This article aims at studying the rock mass failure (rock fall) in underground opening due to earthquakes that are not supported by lining or thick shotcrete (Shabanimashcool and Bhasin, 2020). The knowledge and experiences from rock bursts in mining underground openings is used for deriving a rock support strategy which can be applicable for tunnels under dynamic loads. The knowledge from rock bursts in deep mines will be utilised to assess the rock mass failure mechanism which is then used to assess the required rock support.

A case study from a large cavern in the Himalayas is used for implementing the simplified design approach for strengthening of the rock mass under dynamic load.

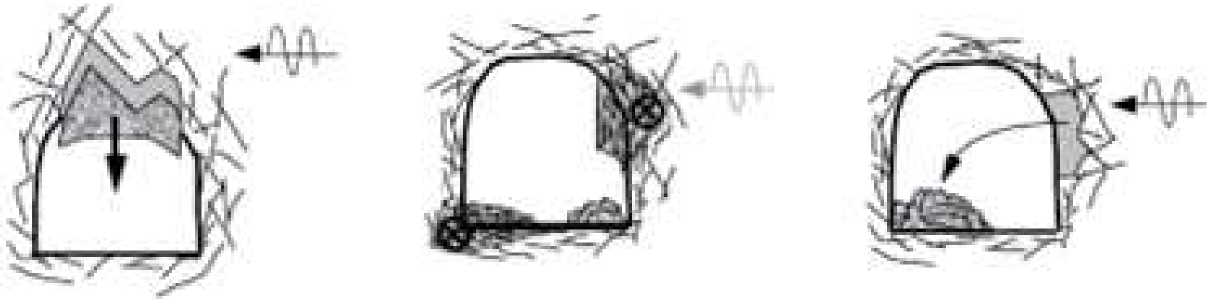
2. DESIGN OF EARTHQUAKE RESISTANT UNDERGROUND OPENINGS USING DAMAGE CAUSED BY ROCK BURST

Rock burst is generated because of two different mechanisms; strain bursts and reactivation of an existing fault. The strain burst happens when the stress concentration in the rock mass around the tunnel reaches a level higher than the compressive strength of the rock mass resulting in brittle failure of the intact rock. This type of failure does not occur in underground opening subjected to earthquake loads and is therefore not considered in this manuscript. Reactivation of an existing fault occurs because of redistribution of stresses caused by mining. The reactivation of a fault generates a seismic wave that propagates in the rock mass. These waves can affect an underground opening that is in the proximity of an epicentre in the following manner (Kaiser et al., 1996), (Fig. 3).

(i) *Seismically induced rockfall*: This is induced by seismic waves and cause the slightly stable rock blocks to become instable.
(ii) *Rock fracture with dilation*: This happens when the rock mass is loaded close to its failure stress. A small increase of stress due to seismicity causes the rock to go under brittle failure and bulking.
(iii) *Block ejection*: This type of failure occurs when the stress waves can cause a block to be dynamically ejected from underground wall opening.

It may be noted that the frequency of the seismic waves also plays a significant role in loading of the rock mass. In large volumes of rock, high frequency waves can result in both compression and extension of rocks and their effect may get self-cancelled so that the resultant force on the rock block may become zero. Thus, opposing high frequency particle acceleration cannot accelerate a block and consequently no extra force is transferred to the rock block. Therefore, only low frequency waves with sufficiently large wavelengths can accelerate volume of a rock block in one direction and cause damage. Based on this analogy Kaiser & Maloney (1997) mentioned that seismic waves with frequency smaller than 100 HZ are most critical for stability of the underground openings.

Furthermore, it is a known fact that if the damage is not initiated by the early part of the incoming waves, then it will not likely be triggered by the later portion, because velocity and acceleration of the wave decreases due to attenuation. Hence, only peak particle velocity or acceleration at a frequency which can accelerate an entire volume of rock block is relevant for design of rock support against earthquake loading conditions.



Seismically induced fall　　Rock fracture with dilation　　Block ejection

Fig. 3 - Rock burst damage mechanisms (Kaiser et al., 1996)

Kaiser and Maloney (1997) implemented the following scaling law to obtain peak particle velocity of seismic waves from rock bursts in mines:

$$\log(Rv) = a\, Log(M_0) + \log(C) \quad (1)$$

where R = the distance from source,
v = peak particle velocity,
M_o = seismic moment, and
a, C = scaling constants.

C is related to stress changes ($\Delta\sigma$) in the source. Based on their analysis, $a = 0.5$ and $C=0.2 - 0.3$ when $\Delta\sigma < 2.5$ MPa.

Hashash et al. (2001) claimed that damages in underground structures are related to peak ground velocity and displacement rather than acceleration. They suggested Table 2 to relate the ratio of peak ground velocity to peak ground acceleration with earthquake magnitudes and distance to the source. Table 3 was suggested to determine the relationship between ground motion at depth and at the ground surface.

Table 2 - Ratio of ground velocity to ground acceleration at surface in rock (Power et al., 1996)

Moment magnitude of earthquake	Ratio of ground peak velocity (cm/s) to peak ground accretion (g)		
	Source to site distance (km)		
	0-20	20-50	50-100
6.5	66	76	86
7.5	97	106	97
8.5	127	104	152

Table 3 - Ratio of ground motion at depth to motion at ground surface (Power et al., 1996)

Tunnel depth (m)	Ratio of ground motion at tunnel depth to ground surface
<6	1.0
6 - 15	0.9
15 - 30	0.8
>30	0.7

In this paper, Tables 2 and 3 are utilized to calculate peak velocity to estimate the dynamic failure mechanism in tunnels.

It has been shown that when peak particle velocity *(PPV)* $\approx$ *50 mm/s* there is possibility of rock fall type of damage in tunnels. When *PPV* $\approx$ *300 mm/s* then fracturing of intact rock may occur, and when *PPV* $\approx$ *600 mm/s* rock block ejection with severe damage can occur in tunnels (Kaiser et al. 1996). Based on these damage mechanisms, various types of rock support were suggested to mitigate these damage mechanisms. Table 4 can be used to assess the damage mechanism in underground structures and required type of rock support. Rockfall in Table 4 occurs due to joints in the rock in contract to fracturing which occurs in intact rocks.

Table 4 - Range of peak velocity for different damage mechanism and required rock support type (Kaiser et al., 1996)

PPV range (mm/s)	Damage mechanism	Required rock support type
10 - 300	Rockfall	Add load capacity.
300 - 1000	Fracturing of intact rock	Add load and displacement capacities.
> 1000	Severe damage and block ejection	Add load, displacement, and energy observation capacities.

3. ROCK SUPPORT IN TUNNELS WITH EARTHQUAKE LOADS

The rock support in tunnels is mostly designed based on empirical methods using for example the Q-system (NGI 2015) and RMR approaches. In these approaches the rock mass properties are mapped in the field or logged from borehole data. The data includes properties of rock joints, intensity of rock jointing (indirectly rock block size), frictional resistance of joint surface and in-situ stress. However, the various mechanisms which can lead to instability in tunnels are not visualised at this stage. This makes it difficult to generalise the design curves or tables from rock mass classifications for rock support design in earthquake conditions.

Each rock support component in the empirical approach is intended to perform one of the three functions: (1) reinforcing the rock mass to make it strong (short rock bolts with small spacing between them), (2) retain broken rock mass to prevent key block failure and unravelling (shotcrete with thickness ≤ 10 cm), and (3) hold key blocks in place and securely tie back the retaining elements to stable ground (rock bolts or rock anchors) (Kaiser et al. 2000). While each support element may simultaneously provide more than one of the above functions, it is convenient to consider each function separately.

Figure 4 shows the rock support functions correlated with the Q-system of rock mass classification (Kaiser et al. 2000). Earthquakes are known to generate smaller peak ground velocity compared to rock bursts, because of wave attenuation due to the relatively large distance from the source. Except for the poor quality of rock mass, which is supported by thick shotcrete, the rest of rock mass classes may experience only rock fall type of damage (peak velocity ≤ 300 mm/s) in jointed rock masses. In such type of damage, the load capacity of the rock support needs to be increased to hold the unstable blocks in place. Depending upon the quality of rock mass, different rock support measures with extra loading capacity will be required.

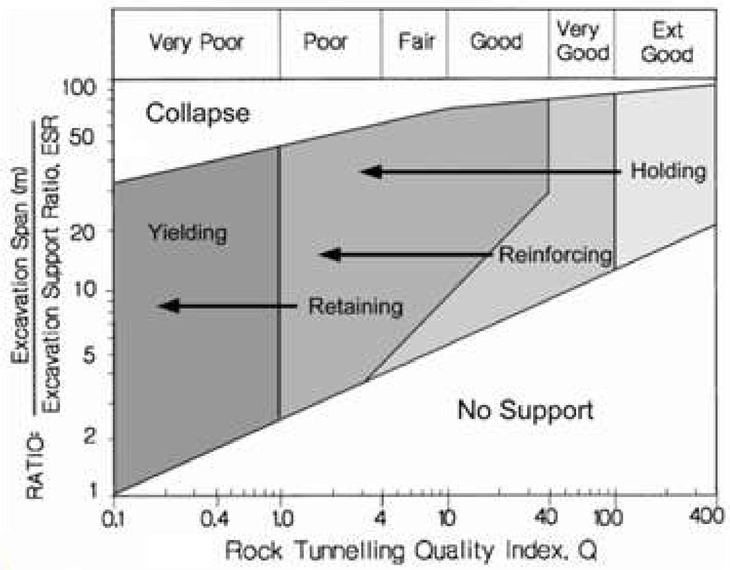
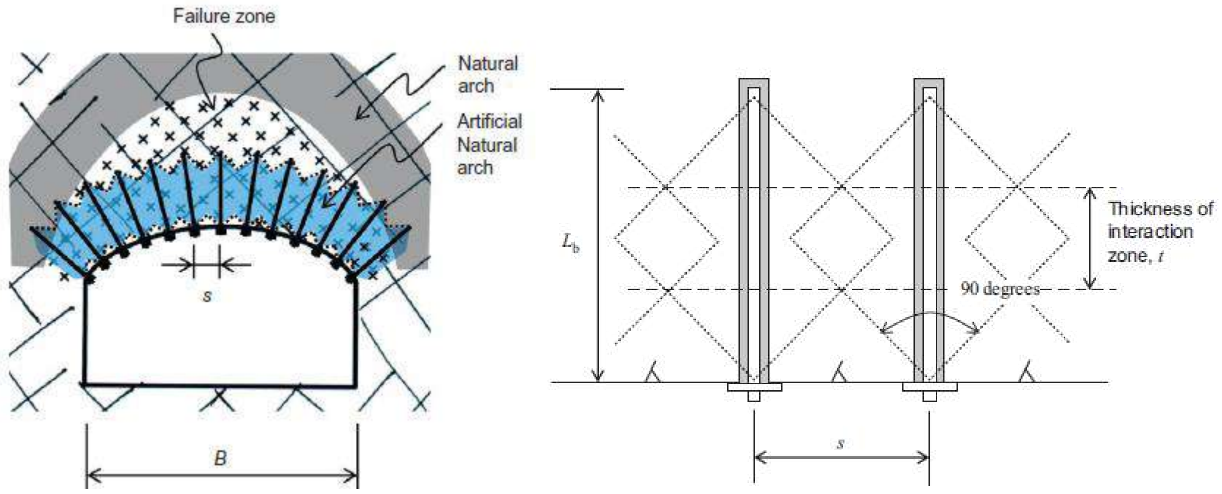


Fig. 4 - Rock support functions in the stability graph of Q-System (Kaiser et al., 2000)

In very good quality of the rock mass, the rock support function (Fig. 3) corresponding to holding element consists of spot bolting to stabilise the rock wedges, which might be generated by unfavourable discontinuity orientation. In this type of rock one can estimate the size of the biggest rock wedge that can be generated and then calculate the required rock support considering earthquake loading using the principles outlined by Li (2017).

However, in good to poor rock masses the reinforcing mechanism of support is used (Fig.4). Reinforcement is mostly carried out through implementation of short bolts which are closely spaced as shown in Fig. 5. These bolts interact with surrounding rock mass and generate a reinforced zone which serves as an artificial arch around the underground opening for stabilisation. In this mechanism shotcrete is used to hold the rock mass together and prevent it from disintegrating. Under earthquake loading, this reinforcing element needs an extra holding mechanism (increased loading capacity) to stay in place. This is achieved through longer rock bolts which connect the artificial arch to the natural arch above (Fig. 5a).



(a) (b)

Fig. 5 - (a) Stablishing of artificial arch above the tunnel with rock reinforcement and (b) Reinforcement mechanism of rock mass (Li, 2017)

4. DESIGN APPROACH FOR STRENGTHENING OF ROCK MASS TO PROTECT FROM EARTHQUAKE LOADING

To assess the effect of earthquake loading on a rock mass in an underground openings tunnel, the following steps are followed:

- Assessment of the expected peak ground acceleration at the site (based on local seismic hazard maps or standards)
- Assessment of the expected maximum ground velocity at tunnel depth using Tables 2 and 3
- The expected failure (damage) mechanism and the required rock support type can then be assessed using Table 4.
- Design of the required rock support for earthquake loading conditions considering the estimated rock support function and supporting mechanism

5. CASE STUDY

Several large underground structures related to hydropower projects have been built in the Himalayas over the past few years. Goel et al, 2012, have eloquently described many of the challenges and important considerations in the design of such underground structures. The above design procedure for strengthening rock mass for earthquake loading is implemented for a powerhouse cavern at a hydroelectric project site in the Himalayas (Bhasin and Pabst, 2014). This cavern lies in the proximity of an active fault (> 20km), which is called MCT (Main Central Thrust) in the Himalayas. The powerhouse cavern has width of 20 m and a height of 57 m and is constructed in gneiss rock. The maximum horizontal in-situ stress is 14 MPa which is parallel with the axis of the cavern and minimum horizontal stress is 7 MPa which is perpendicular to the cavern. The cavern is located at depth of 400 m from the surface of the ground. The Q-value of the rock mass ranges between 1 – 4 signifying poor rock mass quality. The rock support is implemented as a function of the reinforcing system (see Fig. 4) with 12 m long post-tensioned bolts with a spacing of 1.5m c/c (centre to centre) in addition to 10 cm of shotcrete. Table 5 shows the properties of the rock mass.

Based on the seismic hazard map of the site, the expected peak ground acceleration is 0.24g. The peak ground surface velocity is estimated as 190 – 270 mm/s (Table 2). At the depth of the cavern (400 m), the peak velocity will be around 142.5 to 202.5 mm/s, based on Table 3. As per Table 4, considering the peak velocity range between 10 – 300 mm/s, the damage mechanism is "rockfall" type and required rock support type is "add load capacity" to the rock mass.

Table 5 - Properties of rock mass (Bhasin and Pabst, 2014)

Properties	Value
Density (kg/m^3)	2700
E of intact rock (GPa)	30
GSI-value	40
E of rock mass (GPa)	6.40
Cohesion of rock mass (MPa)	3.41
Friction angle of rock mass (^{o})	27

Numerical analysis using RS2 (Rocscience) showed a failed zone with a thickness of about 5-15 m generated around the cavern (Fig. 6). Rock bolts of 12m length were installed to stitch this failed zone. The closely spaced installed rock bolts in the damaged zone works as a reinforcement system in accordance with the stabilization effect shown in Fig 5. Now in case of an earthquake this reinforced zone is expected to undergo a peak velocity of 202.5 mm/s. Holding type of rock support is required to carry the extra load imposed to the reinforced zone during an earthquake. The cable bolts are intended to use which their design will be demonstrated by using the following methodology.

The dynamic energy which is imposed to the reinforced zone of rock mass in the cavern wall during an earthquake can be calculated as:

$$E_d = \frac{1}{2}mv^2 \tag{2}$$

where *m* is mass of the reinforced rock (rock mass in the yield zone) and *v* equals to 202.5 mm/s.

Considering that the holding rock support will be cable bolts installed in a systematic pattern. The mean thickness of reinforced zone is t = 12 m (which is the rock bolt length installed in the cavern). Therefore, the cable bolts should be larger than 12 m. Eq. 2 can be written as:

$$E_d = \frac{1}{2}\gamma t s^2 v^2 \tag{3}$$

Where γ = rock mass density in kN/m^3,
s = spacing of the cable bolts in meter (m), and
t = thickness of the reinforced zone in meter (m).

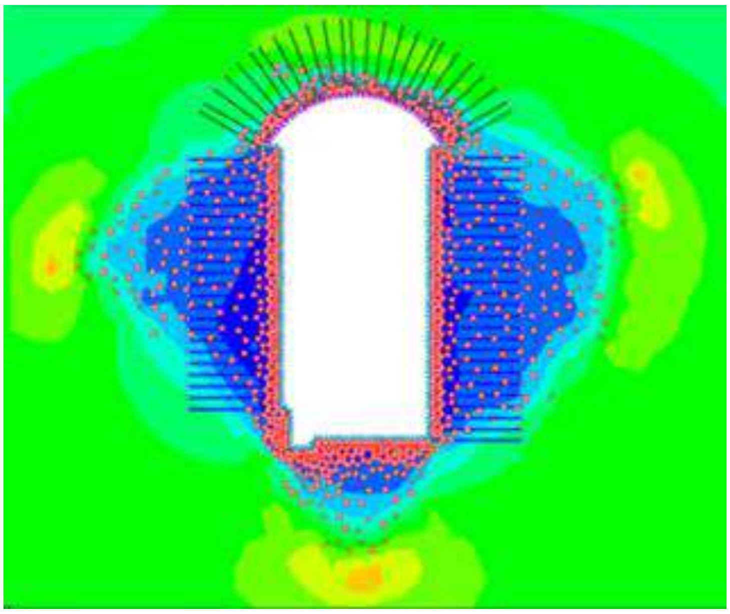


Fig. 6 - Rock mass damaged zone (shown by small circles for shear failure and cross for tension failure) around the cavern (Bhasin and Pabst, 2014)

The dynamic energy, which should be stored by the cable bolt, intended to hold or withstand the moving mass of the reinforced zone is equal to:

$$E_a = \frac{\pi d^2 l}{8} n\sigma_y\varepsilon_y \quad (4)$$

where n = total number of strands in the cable bolt,
σ_y = yield stress of cable material,
ε_y = strain of cable strand's steel material at yield ($\varepsilon y = 0.002$),
d = diameter of each strand, and
l = free length of the cable bolt which undergoes extension at the time of loading.

The free length of the cable bolts should be designed based on the allowable convergence of the cavern walls during earthquake. In this case, since we mostly need holding type of rock support; this needs to hold all the rock mass in place during the earthquake. It may be noted that we do not need an energy absorption type for which the peak velocity due to earthquake is higher than 600 mm/s. The free length of the cable, therefore, should be the shortest length which can carry the dynamic load from an earthquake.

Since the energy which gets transferred to the reinforced zone around the powerhouse is equal to the energy that will be stored in the rock bolts, Eq. (3) and (4) should be equated as:

$$E_d = E_a \quad (5)$$

This equation can be used to find the required dimensions of the cable bolts for stabilising the cavern.

Table 6 shows the various options for stabilization of the cavern with different number of strands included in the cable bolts. The bonding length of the cable bolt needed beyond the reinforced zone is dependent upon the mechanical properties of the grout bonding cable and rock.

The shear resistance of grout (τ_g) is usually assumed as about 8% of its uniaxial compressive strength. Thus, for a grout with uniaxial compressive strength of 40 MPa the following calculations can be performed for determining the bonding length (l_{bond}):

$$l_{bond} = \frac{\sigma_y d}{4\tau_g} = \frac{1718 \times 0.0229}{4 \times 3.2} = 3.07 \text{ m} \tag{6}$$

The calculation shows that, the cable bolt needs about 3 meters of bonding length. Generally, a factor of safety of 2 is used in design of the bond length. Therefore, implemented bonding length should be 6 m.

Considering that the reinforced zone is about 12 m thick, we can utilise 12 m free length for the cable bolts. Since the cable bolts also need 6 m of bonding; the total required cable bolt length will be ca. 18 m. In case of assuming factor of safety 1 for the bond length of the cable bolts, the total length of the cable bolts decreases to 15 m.

Table 6 - Calculations showing the required cable bolting to stabilise the cavern

Properties	Case 1	Case 2	Case 3
Yield strength of steel (MPa)	1718	1718	1718
Young's modulus of steel (GPa)	200	200	200
Diameter of one strand (mm)	22.9	22.9	22.9
No. of strands	1	3	5
free length (m)	12	12	12
Strain at yield (x 10^{-3})	2	2	2
E_a (x 10^{-4} MJ)	85	255	424
Density of rock (x 10^{-3} MN/m^3)	27	27	27
length of failed zone in rock (m)	12	12	12
Peak velocity (mm/s)	202.5	202.5	202.5
Allowed spacing of bolts (m)	1.13	2.00	2.50

Bhasin and Pabst (2014) showed by numerical analysis that at the above-mentioned rock support comprising of 12 m long bolts are not enough to stabilise the cavern under earthquake loading. They concluded that anchors with a length of 20 m to tie back the reinforced rock mass far into the natural arch will stabilise the cavern under dynamic loads (Fig. 7). The calculations performed in this manuscript using the simplified design methodology matches the outputs from their numerical results.

However, it should be mentioned that the present design technique here and the numerical modelling by Bhasin and Pabst (2014) are both conservative, since they do not consider damping of the dynamic energy inside the rock mass.

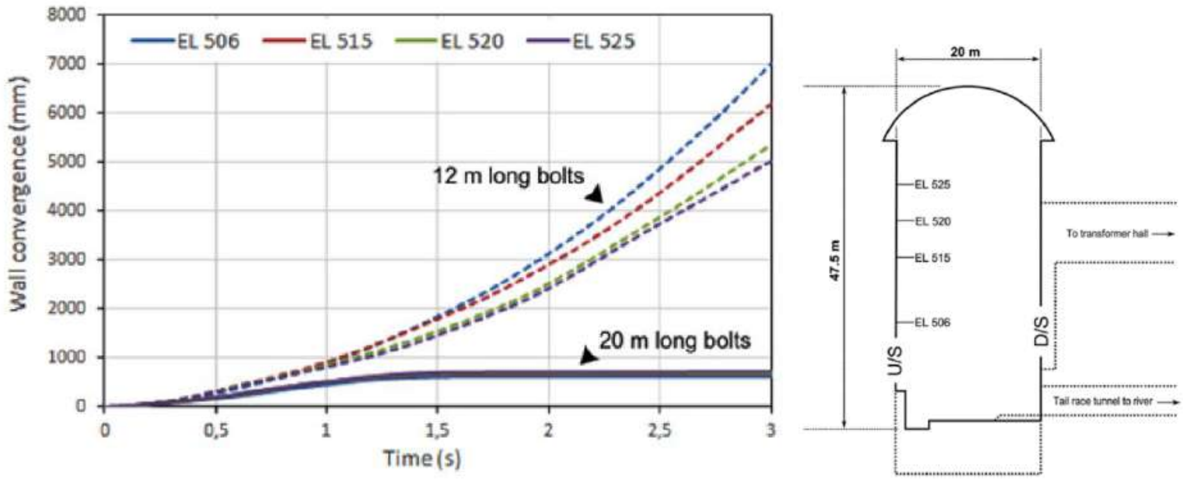


Fig. 7 - Convergence of walls of the cavern with time obtained by dynamic analysis (Bhasin and Pabst, 2014)

6. CONCLUSIONS

A new and simplified design approach for strengthening of the rock mass to withstand earthquake loads in underground openings is presented in this manuscript. This approach supplements the empirical techniques, such as the Q-system, which are used for selecting rock support under static loading conditions. The design approach is based on estimating the expected peak ground velocity at the site during earthquakes followed by identifying the failure mechanism and required type of support measure. After that, the required rock support can be designed in detail by considering the dynamic loading from earthquake.

A case study from a large underground powerhouse cavern is taken to illustrate the design methodology. The result from this new approach matches the output results from detailed numerical analysis of the powerhouse cavern.

The technique presented in this manuscript can be helpful in preliminary design of rock support against earthquakes. It is intended to help engineers to have a preliminary idea regarding the failure mechanism of the underground opening before following any complex numerical modelling.

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