**Proceedings of 12th IOE Graduate Conference**
*Peer Reviewed*
ISSN: 2350-8914 (Online), 2350-8906 (Print)
Year: 2022 Month: October Volume: 12

# A Study on the Influence of Discontinuities Orientations on Seismic Response of Shallow Rock Tunnel, using Phase2

Mahesh Upadhyaya [a], Subrat Subedi [b], Sagar Paudel [c]

[a, c] *Department of Civil Engineering, Pashchimanchal Campus, IOE, Tribhuvan University, Nepal*
[b] *NEA Engineering Company Limited, Nepal*
[a] er.mahesh8848@gmail.com, [b] subrat.sbd@gmail.com, [c] paudel.sg30@gmail.com

**Abstract**

A pseudo-static analysis of a shallow rock tunnel has been carried out in a Phase2 to study the effect of an earthquake with different dipping orientations of rock joints. For the analysis, rock mechanical properties of intact rock have been collected from the Phukot Karnali Hydroelectric Project (PKHEP) and parametric analysis has been performed, referencing various literature. The seismic coefficient for the analysis has been taken from the site's seismic hazard analysis report.Thirty numerical models have been prepared and analyzed under varying discontinuity orientations and seismic force directions.The significance of the seismic effect varied on the orientation of discontinuities. For the combinations analyzed, joint sets with orientation dipping 60°/ 120°joint set orientations are found to be most unfavorable.

**Keywords**

Pseudo-static analysis, shallow rock tunnels, structural discontinuities, effects of an earthquake

## 1. Introduction

Nepal lies in one of the most seismically active regions in the world. Small-scale earthquakes occur frequently in Nepal. The latest medium-scale catastrophic Gorkha earthquake occurred in 2015. While it is well accepted that deep underground structures are far less vulnerable to earthquakes than superficial ones, several tunnels have suffered sever damage in large scale earthquakes such as the 1999 Chi-Chi earthquake (Taiwan), 1995 Kobe (Japan), etc. So, consideration of seismic load while designing tunnels is vital, in the context of Nepal.

Ground failure and ground shaking are the major effect of earthquake on tunnels. Ground failure encompasses liquefaction in soil, slope stability problem at the tunnel portals, and fault displacements. Liquefaction in rock tunnel is not a problem, but slope instability and fault displacement can impact tunnels. During an earthquake event, seismic waves travel through the earth's crust which produces variable ground deformations and has a tendency of ovaling and racking of the tunnel section is termed as ground shaking [1], [2].

Dowding and Rozen [3] made one of the first compilations of damage to rock tunnels due to earthquakes by collecting the information from the 71 tunnels and their findings can be summarized as:

- Tunnels collapsed due to shaking under extreme conditions only.
- When PGAs and PGVs were lower than 0.19g and 0.2 m/s respectively, no damage occurred.
- When PGAs and PGVs were lower than 0.5g and 0.9 m/s respectively, minor to moderate damage occurred.
- When PGAs and PGVs were larger than 0.5g and 0.9 m/s respectively, moderate to heavy damage occurred.
- The tunnel collapsed only in the case of the movement of an intersected fault.

Sharma and Judd [4] extended the database of Dowding and Rozen [3] of earthquakes damages to tunnels based on overburden, rock type, types of support, geographic location, the magnitude of the earthquake, and epicentral distance and found that the shallow tunnels (overburden of less than 50 m) were heavily damaged. Power et al. [5] studied the effect of earthquakes on the extent of damage in different linings and also found that the damage to the linings became pronounced for earthquakes with PGA greater than 0.5g as presented in figure 1.

Nick Barton [6] made one of the earliest seismic load considerations in the design of underground structures using the Q system of the rock mass classification by assuming that $Q_{seismic}$ is half of the $Q_{static}$. One

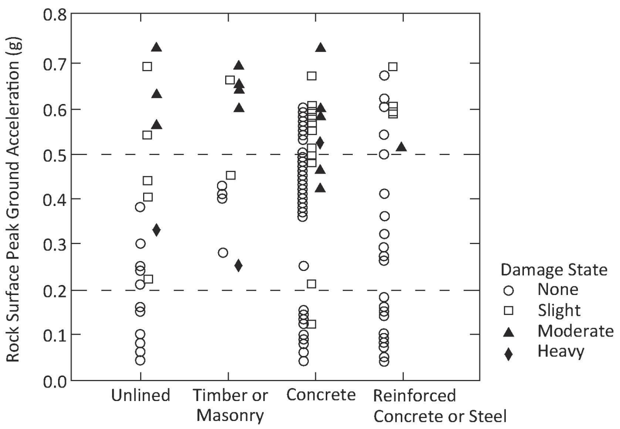


**Figure 1:** Effect of PGA magnitude on different tunnel lining as per [5]

of the basic understanding about the issue involved in the seismic design of subsurface excavation and underground structures was given by John and Zahrah [7]. Further, Hashash et al. [1] presented the state of the art of seismic design and analysis philosophy of underground structures [1].

The main objective of the present research work is to assess out the earthquake impact on shallow rock tunnels regarding the varying dipping angles of rock joint sets. Fifteen combination models have been prepared from the different dipping angles of the rock joint sets, and are 0°, 30°, 60°, 90°, 120°, and 150°. Initially, static analysis of all the model have been performed to estimate the static load and deformation, and later pseudo-static analysis of the models have been performed. Based on the estimated stress-strain data, a favorable and unfavorable combination of the joint set has been predicted. During the analysis, an inverted D tunnel of 8m span in augen gneiss with shallow overburden has been considered.

## 2. Methodology

The general methodology step followed during the study is given in figure 2. The research work commenced with the desk study followed by continuous literature review, field visit for data

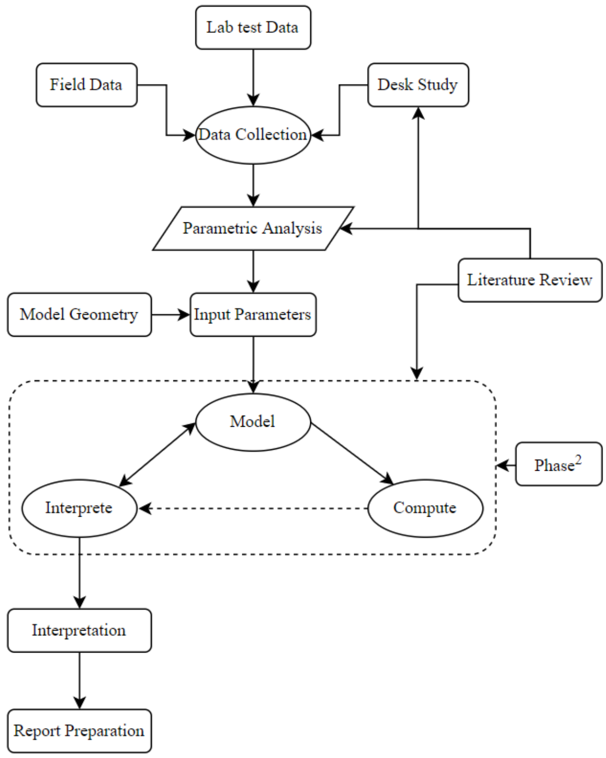


**Figure 2:** Methodology steps followed during the study

collection, numerical modeling, interpretation, and compilation as a paper. The necessary data for the analysis like joint mapping, JRC, JCS, rock types, and Q-values have been recorded during the field visit work at the site (Inlet Portal of HRT of Phukot Karnali Hydro Electric Project). Modulus of elasticity, poisson's ratio, and UCS have been taken from laboratory testing of rock core samples. Deformation modulus, rock mass strength, cohesion, frictional angle, etc has been estimated through literature reviews and are given in table 1.

### 2.1 Assessment of Input Parameters

The input parameters required for the numerical modeling are rock mass properties, properties of a support system, and in-situ stresses. The lab test data of the rock core sample is presented in table 2.

The mean value of rock core properties has been selected for further analysis. Rock mass strength of homogeneous, massive, and brittle rock has been estimated by adopting the relationship with the strength of intact rock [11].

$$Rock\,mass\,strength\,(\sigma_{cm}) = \frac{\sigma_{ci}^{1.6}}{60} = 25.241\,MPa \quad (1)$$

**Table 1:** Summary of the input parameters

| Description | Unit | Esti. Value | Remarks |
|---|---|---|---|
| GSI | - | 59 | Estimated from Q = 9.71 as RMR = 15log(Q)+50, and GSI = RMR-5. |
| Material constant (mi) | - | 28 | Practical Rock Engineering Book by evert Hoek [8]. |
| Disturbance factor (D) | - | 0.8 | Poor blasting in hard rock tunneling [9]. |
| mb | - | 2.56 | From Hoek-Brown failure criterion [9]. |
| Constant (s) | - | 0.00227 | using equation as mentioned in [9]. |
| Constant (a) | - | 0.50288 | using equation as mentioned in [9]. |
| Constant (c) | MPa | 0.473 | Equivalent cohesion [9]. |
| Peak frictional angle | deg. | 65.205 | Equivalent frictional angle [9]. |
| Tensile strength | MPa | 0.086 | using equation as mentioned in [9]. |
| Deformation modulus | GPa | 5.514 | as suggested in [10] and [11]. |
| JRC | - | 12 | from field visit. |
| JCS | - | 90 | from field visit. |
| Residual frictional angle | deg. | 27 | assumed . |
| Normal Stiffness $K_n$ | GPa | 3.724 | using equation suggested in [12]. |
| Shear stiffness $K_s$ | GPa | 1.644 | using equation suggested in [13]. |
| Spacing of the joints | meter | 2 | from field visit |

**Table 2:** Lab test properties of augen gneiss

| Statistical Parameters | $\sigma_{ci}$ in MPa | $E_{ci}$ in GPa | $\nu$ |
|---|---|---|---|
| Minimum | 69.400 | 19.700 | 0.1000 |
| Mean | 97.200 | 21.325 | 0.1325 |
| Maximum | 111.800 | 22.700 | 0.1800 |
| S.D. | 12.200 | 1.600 | 0.0300 |

Similarly, the deformation modulus $(E_{rm})$ of the rock mass has been estimated by the equation below [10].

$$(E_{rm}) = E_{ci}\frac{\sigma_{cm}}{\sigma_{ci}} = 5.514\,MPa \tag{2}$$

Mohr-Coulomb failure criterion and an elastic perfectly plastic material model has been adopted for the analysis. For the slip criterion of joint, Barton and Bandis slip criterion [14] has been adopted to estimate the shear strength of the discontinuities as:

$$\tau = \sigma_n \cdot tan(JRC \cdot log\frac{JRC}{\sigma_n} + \phi_r) \tag{3}$$

Unit weight of the augen gneiss has been assumed as 27 $KN/m^3$. The overburden depth above the crown has been taken as 20m. Seismic hazard assessment of the site (Phukot Karnali Hydro Electric Project) recommends the 84th percentile of the response spectrum for design purposes. The PGA for this event is approximately 0.45g under the free field condition. For underground condition (confined condition), the PGA value has been estimated as per the relation mentioned in table 3 [5].

**Table 3:** Ratios of ground motion at depth to motion at the ground [5]

| Tunnel Depth (m) | Ratio of ground motion at tunnel depth to motion at ground surface |
|---|---|
| $\leq 6$ | 1 |
| 6-15 | 0.9 |
| 15-30 | 0.8 |
| $> 30$ | 0.7 |

The seismic coefficient at 20m depth can be estimated with reference to table 3 as:

$$Horizontal\ seismic\ coefficient = 0.36g$$

$$Vertical\ seismic\ coefficient = -0.18g$$

Vertical seismic coefficient has been estimated by taking half value of horizontal seismic coefficient as suggested by Melo and Sharma [15] and -ve sign indicates the downward direction of seismic coefficient.

The in-situ gravitational stress best representing an elastic homogeneous condition, such that the stress anisotropy (k) can be calculated from the relationship with poisson's ratio as:

$$stress\ anisotropy\ (k) = \frac{\nu}{1-\nu} = 0.1527 \tag{4}$$

A composite liner of concrete and shotcrete has been applied at the contour of the blasted tunnel. The applied support has been analyzed under the elastic condition. The properties of the applied support system are mentioned in table 4.

**Table 4:** Properties of the applied support

| Composite liner of | Shotcrete | Concrete |
|---|---|---|
| Liner type | Standard beam | Standard beam |
| Young's modulus MPa | 25000 | 35000 |
| Poisson's Ratio | 0.2 | 0.2 |
| Thickness m | 0.1 | 0.15 |
| Peak compressive strength MPa | 35 | 35 |
| Residual compressive strength MPa | 5 | 5 |
| Peak tensile strength MPa | 5 | 5 |
| residual tensile strength MPa | 0 | 0 |

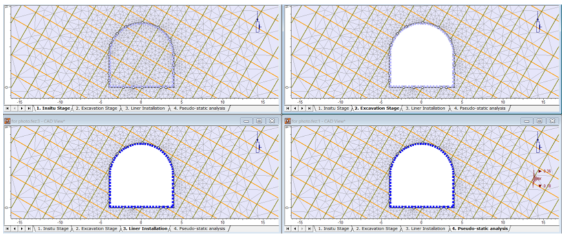

**Figure 3:** All four stages of model preparation in (60° and 120° dipping joint sets)

## 3. Analysis and Results

### 3.1 Numerical Modeling

Phase$^2$ is a two-dimensional finite element method-based windows program of the Rocscience package. It is widely popular and applicable for the analysis of underground/surface excavation in rock mass or soil. Basically, model (pre-processing), compute (processing), and interpret (post-processing) are the three fundamental program modules availale in phase2 program.

Modeling of an inverted D tunnel of an eight-meter span has been performed in four stages as shown in figure 3. The first stage represents the in-situ stage, the second stage represents the excavation stage, the third stage represents a support application stage, and the fourth stage represents application of seismic load stage. The properties of the loading, material, joints, liner, and composite liner have been defined and assigned in this module. To analyze the numerical models in phase$^2$, axisymmetric and Plane-strain analysis are the two available techniques. The axisymmetric analysis of the jointed rock mass is not possible because of the concentration of the stress along the rock joints. Although the rock joints need to be modeled with 3D modeling programs like unwedge, RS3, FLAC3D, or 3DEC, due to the available features of ubiquitous joint modeling in phase$^2$, the models are analyzed with plane strain analysis. The per-processing phase have been computed with 0.001 tolerance value and 5000 iterations using gaussian elimination methods. Absolute energy and the square root energy are the two available convergence criteria in phase$^2$, among them, the absolute energy convergence criterion has been adopted.

A typical example of a 0° and 30° dipping combination of joint sets is shown in figure 4. Initially, an elastic analysis has been performed. In the interpret module, the strength factor of the rock mass has been found to be less than one as shown in figure 4. It represents the ratio of available rock mass strength to induced stress, at the given point. The elastic analysis cannot incorporate beyond the yield strength of the material, and hence plastic analysis has been adopted for further analysis. During the plastic

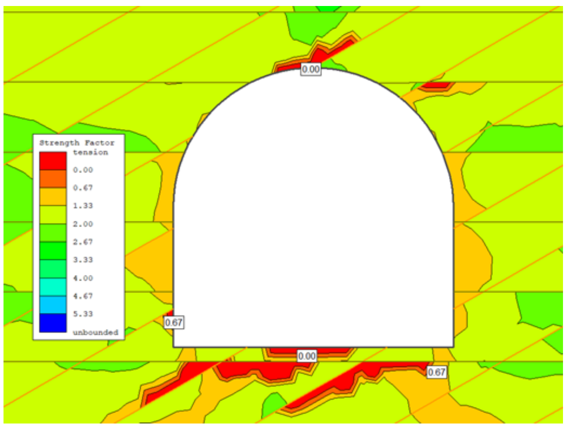


**Figure 4:** Strength factor during an elastic analysis of 0° and 30° joint set dipping model

analysis, an elasto-plastic material model has been adopted. The assessment of the effect of an earthquake has been performed by analyzing the maximum axial force in the composite liner and the total deformation at the crown, in each combination of joint dipping orientations (models). Figure 5

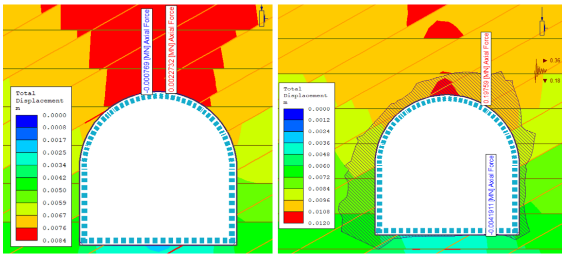

**Figure 5:** Variation of axial force in the liner in static condition (left) and dynamic condition (right)

represents a significant increase in the axial force in the liner element due to the consideration of surplus seismic load (seismic load coefficient left to the right case). The increase in the maximum axial force due to earthquake in the liner based on the different dipping orientations of the joint sets and the different directions of application of horizontal seismic coefficient (right to left and left to right) is presented in the plot as shown in figure 6.

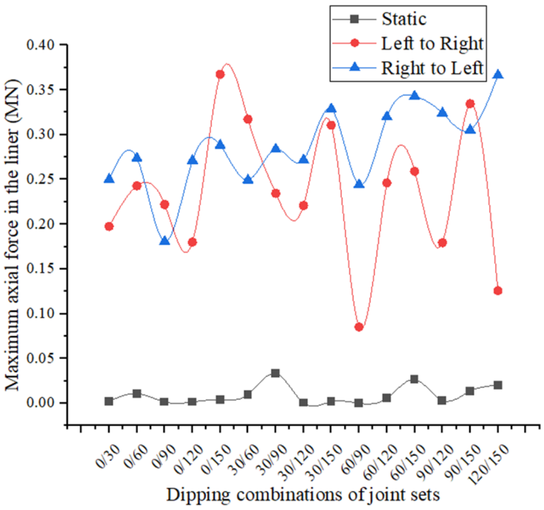


**Figure 6:** Maximum induced axial force in the liner in both static and dynamic conditions (from left to right direction of horizontal seismic coefficient, and from right to left direction of horizontal seismic coefficient.)

The total displacement has been observed maximum at the crown in all the combinations regarding the direction of application of seismic loading. So, the total displacement at the crown has been queried and plotted in both static and dynamic loading condition as shown in figure 7.

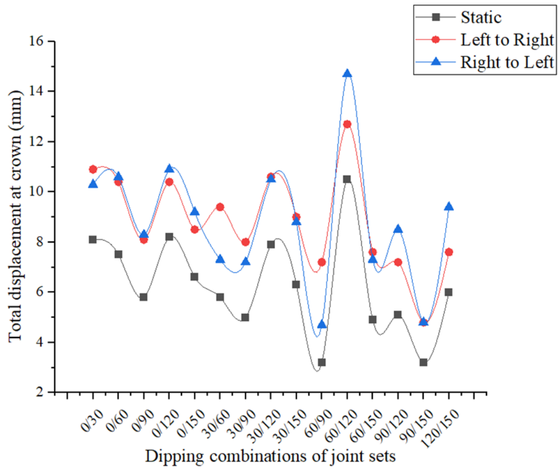


**Figure 7:** Total displacement at the crown in both static and dynamic conditions (direction of horizontal seismic coefficient from left to right, and direction of horizontal seismic coefficient from right to left)

## 4. Discussions

The model with a joint set of 60° and 90° dipping combination generates the minimum value of displacement at the crown concerning other combinations of joint sets as shown in figure 7. This combination also gives the lowest value of induced axial force in the liner. So, this combination of the joint set can be considered as a favorable orientation provided the constraints of the input parameters.

The model with a joint set of 60° and 120° dipping combination induced a maximum displacement at the crown of the tunnel in either direction of seismic loading condition as presented in figure 7. So this combination can be considered as an unfavorable dipping orientation of two join sets.

The deviation in the force and displacement was observed in a different direction of seismic loading (i.e., left to right and right to left direction of horizontal seismic loading). This might be due to the direction in which the blocks have a tendency to displace.

The most unfavorable combination ( model with 60° and 120° dipping orientation combination of joint sets) predicted in the above analysis because a wedge is formed at the roof of the tunnel which is quite unstable in the unsupported condition, as shown in figure 8. To support this unstable block, rock bolts have been applied as initial support, and in the interpret module, no bolts have been found to be yielded. For the final lining design, reinforced concrete has been applied and the support capacity has been plotted as presented in figure 9.

Stage 10 (a legend in figure 9) is a static load application stage while, stage 11 is an application of seismic coefficient. The concrete is safe with a factor of safety of 1 in the static condition and become unsafe in the dynamic condition. The effect of an

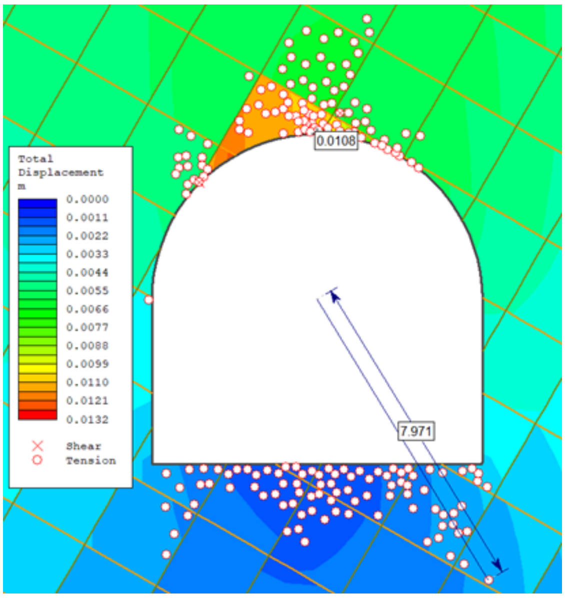


**Figure 8:** Plastic radius and maximum displacement

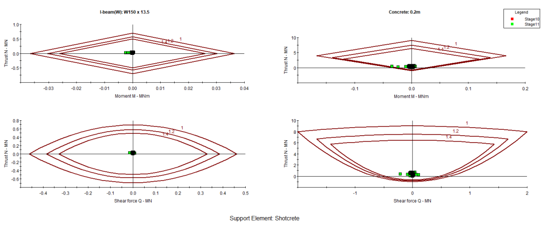

**Figure 9:** Support capacity plot of reinforced concrete with different factor safety

earthquake has been studied with a varying spacing of the rock joint sets (in 60° and 120° dipping orientation of rock joint sets model) and it is found that the effect decreases as the total displacement at the crown decreases with increase in spacing of the joint sets as shown in figure 10. Furthermore, induced the axial force in the liner in both static and dynamic loading conditions has been estimated, and found that the increment of axial force in the liner decreases with an increase in the spacing of the rock joints as shown in figure 11.

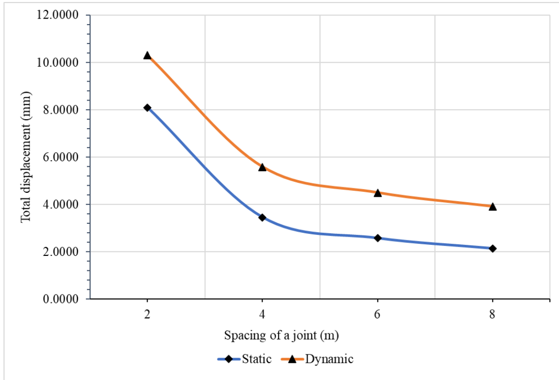


**Figure 10:** Total displacement at the crown in both static and dynamic loading with varying spacing of the joint sets

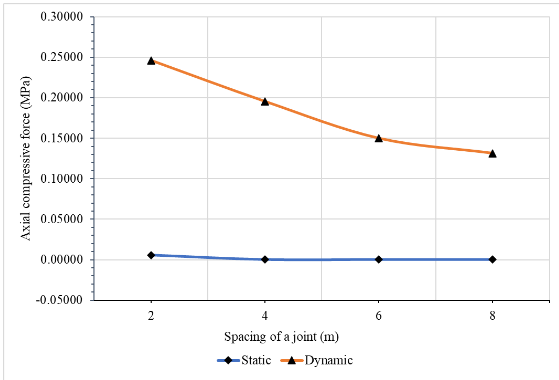


**Figure 11:** Maximum axial force in the liner in both static and dynamic loading with varying spacing of the joint sets

## 5. Conclusions and Recommendations

Concerning the results of the above analysis data, the following conclusions can be drawn.

- The intersection of these two joint sets forms rock wedges and tends to fall after the excavation of the tunnel section. In the case of water supply tunnel, this tendency to form an unstable wedge can be minimized by deviating/shifting an alignment. But in the case of the road/railway tunnels, due to the constraints of the radius of curvature, sometimes the alignment may not be possible to shift. If this condition encounters, the tunnel engineer needs to be aware that the joint orientation is one of the governing factors for the stability concern in dynamic loading conditions, by keeping all the input parameters constant.
- When the orientation of one joint set varies

between 90° to 120° while another joint set maybe 0°, 30°, 60°, and 90°, Unfavorable condition of the joint increases.

- Among unfavorable conditions, a model with 60° and 120° dipping joint set orientation are predicted as the most unfavorable combination.
- The most favorable orientation condition of the joint set regarding maximum axial force in the liner and total deformation at the crown is 60° and 90°.
- As the spacing of the joint sets increases, the effect of an earthquake decreases as shown in figures 10 and 11. The axial force in the liner in figure 11 during the static loading is almost zero because the support is applied after the 100 percent relaxation.

The following major points are recommended for further research work to enhance the state of art in the field of influence of earthquake on shallow rock tunnels.

- Finer result can be expected if the interval between the consecutive orientation of the joint set is made finer. In this model, a 30° interval between the consecutive orientation of the joint set (i.e., 0° or 180°, 30°, 60°, 90°, 120°, and 150°) has been used.
- In a shallow rock tunnel, earthquake effects not only depend upon the dipping orientation of the joint set. Therefore, sensitivity analysis should be performed based on the normal joint stiffness, overburden depth of the tunnel, frictional angle, and shear stiffness of the joints.
- The deformations and subsequent stress concentration are seen to occur at the interface between the joints and the tunnel lining. Thus the random position of the joint set with respect to tunnel opening may differ the impact of an earthquake on the rock tunnels.

## Acknowledgments

The authors would like to acknowledge Vidyut Utpadan Company Limited (VUCL) for providing the necessary data and access to the PKHEP site for the analysis.

## References

[1] Youssef M.A. Hashash, Jeffrey J. Hook, Birger Schmidt, and John I-Chiang Yao. Seismic design and analysis of underground structures. *Tunnelling and Underground Space Technology*, 16(4):247–293, 2001.

[2] Carlos A. Jaramillo. Impact of seismic design on tunnels in rock – case histories. *Underground Space*, 2(2):106–114, 2017. Seismic Response of Tunnels.

[3] Dowding C. H. and Rozen Arnon. Damage to rock tunnels for earthquake shaking. *Journal of the Geotechnical Engineering Division, American Society of Civil Engineer*, 104(GT2):175–191, 1978.

[4] Sunil Sharma and William R. Judd. Underground opening damage from earthquakes. *Engineering Geology*, 30(3):263–276, 1991.

[5] M Power, D Rosidi, J Kaneshiro, S Gilstrap, and SJ Chiou. Summary and evaluation of procedures for the seismic design of tunnels. final report for task 112-d-5.3 (c). *National Center for Earthquake Engineering Research, Buffalo, New York*, 1998.

[6] Nick Barton. Effects of rock mass deformation on tunnel performance in seismic regions. *Advances in Tunnelling Technology and Subsurface Use*, 4:89–99, 01 1984.

[7] C.M. St John and T.F. Zahrah. Aseismic design of underground structures. *Tunnelling and Underground Space Technology*, 2(2):165–197, jan 1987.

[8] Evert Hoek. *Practical Rock Engineering*. 2007.

[9] E Hoek, C. Carranza-Torres, Brent Corkum, Evert Hoek, and Carlos Carranza-Torres. Hoek-brown failure criterion - 2002 edition. 01 2002.

[10] Krishna Panthi. *Analysis of Engineering Geological Uncertainties Related to Tunnelling in Himalayan Rock Mass Conditions*. PhD thesis, 02 2006.

[11] Krishna Panthi. Methods applied in the prediction of brittle failure in tunnels and underground caverns. *Hydro Nepal: Journal of Water, Energy and Environment*, 22:5, 01 2018.

[12] N.R. Barton. A model study of rock-joint deformation. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 9(5):579–582, 1972.

[13] B. Singh. Continuum characterization of jointed rock masses: Part i—the constitutive equations. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 10(4):311–335, 1973.

[14] Nick Barton and S Bandis. Review of predictive capabilities of jrc-jcs model in engineering practice. volume 182, pages 603–610, 11 1990.

[15] Cristiano Melo and Sunil Sharma. Seismic coefficients for pseudostatic slope analysis. In *13 th world conference on earthquake engineering, Vancouver, Canada*, 2004.