PROGRAM AND ABSTRACT BOOK

1st INTERNATIONAL CONFERENCE ON SPACE UTILIZATION AND RESEARCH IN UNDERGROUND STRUCTURES

MAY 5-6 , 2022

SURUng

ABOUT

With the start of the 'Tunnel Era' in Nepal, the construction of tunnels for transportation and water divergence is increasing. As of February of 2022, more than 20 transportation tunnels have been proposed with 4 tunnels under construction. In the current fiscal year 2078/79, Nepal government alloted approximately NRs. 1 billion for the study phases in tunnel construction. As a result, more research and development works are required in underground structure planning, design and construction. Only after the completion of the necessitated research works will the transition into the 'Tunnel Era' in Nepal gain momentum. These research works will also provide foundations for the development works that follow. In this regard, Kathmandu University is organizing the first ever international conference on Space Utilization and Research in Underground Structures (SURUng-I) to bridge the gap between industry and academia and allow knowledge sharing among them .

SURUng-I will be the first event in the SURUng conference series that brings together academicians, researchers, engineers and industry experts involved in multiple disciplines of Space Utilization and Underground Structures. SURUng-I, sponsored / co-sponsored by Nepal Electricity Authority (NEA), Nepal Construction & Engineering Corporation Pvt. Ltd. (NCEC), Energize Nepal, Hydro Tunnelling and Research Pvt. Ltd. (HTR) and supported by Energize Nepal, University Grnat Commission (UGC), Nepal and NORHED II project (Paschimanchal Campus, NTNU) will be held in Kathmandu University, Dhulikhel, Nepal on May 5 and 6, 2022.

SURUng-I will provide a leading edge, scholarly forum for researchers, engineers and students alike to share their state-of-the art research and developmental work, new ideas and breakthroughs in areas of Space Utilization and Underground Structures. Prospective authors are invited to submit their papers reporting original works as well as tutorial overviews in all areas of Space Utilization and Underground Structures.

MESSAGE FROM VICE-CHANCELLOR

Prof. Dr. Bhola Thapa,

Vice- Chancellor

Kathmandu University

It is a matter of great pleasure that the First International Conference on Space Utilization and Research in Underground Tunneling (SURUng-I) is being held at Kathmandu University (KU), on the 5th and 6th of May.

With the increasing construction of tunnels and underground structures for transportation, water divergence, and water storage in the country, around NRs. 2 billion has been allocated for underground structures in the current 2078/79 fiscal year. There is thus a need for more R&D works and close collaboration between industries and academia for development and progress in underground engineering.

I thus take this conference as a great opportunity to bring together academics and practitioners involved in this field. This conference is both a forum for networking among professionals and researchers, and a platform through which a substantial amount of knowledge in the core and relevant fields of the tunnel engineering can be shared. And the theme of this conference briges the gap between the research and application in the increasing trend of construction of underground structures in the country.

I am grateful to all the collaborating institutions, authors, sponsors, and well-wishers who have contributed to making this program into reality and I look forward to their support in the coming days as well. The never-ending work of SURUng-I members and all members who have contributed to the program deserves a huge appreciation as well.

I wish "SURUng-I" a huge success.

MESSAGE FROM DEAN, SOE

Prof. Dr. Manish Pokhrel

Dean, School of Engineering

Kathmandu University

With immense pleasure I am glad to note that 'SURUng', the First International Conference on Space Utilization and Research in Underground Structures jointly organized by Kathmandu University, Norwegian University of Science and Technology, Seoul National University, and Hydro Tunneling and Research Pvt. Ltd. is being hosed at Kathmandu University.

With the increasing construction of tunnels and underground structures in the country, it has become essential to bridge the gap between industry and academia. SURUng-I provides the platform to connect professionals, experts and academicians. With the prioritization of tunnels for development in the country, Government of Nepal has also declared the start of 'Tunnel Era' in the country. In this regard, Department of Civil Engineering has been working in building human resource capacity through the introduction of underground structures course from Undergraduate level in the country.

Kathmandu University has also been constructing a tunnel passing through the hillock of the university. The commencement of the construction phase of this tunnel is preceded by years' worth of thorough research and development and planning. The culmination of all this hard work will surely give fruitful result.

Tunnelling has been the current development priority of the nation, on this note Kathmandu University has started a tunnel construction planning within the University itself from 2012. The theme of this conference blends with the tunnel which is being constructed in Kathmandu University which will provide a leading-edge scholarly forum for researchers, engineers, and students alike to share their state-of-the-art research and development works, new ideas, and breakthroughs in areas relevant to tunnel and Underground Engineering. I am sure this conference will pioneer the phase of extensive conference on the topic of tunnelling and research in underground structures.

I therefore would like to thank everyone for working tirelessly to make this conference a grand success and wish all the participants an efficacious experience at the conference.

MESSAGE FROM SNU

Prof. Seokwon Jeon
Professor, Seoul National University
President-Elect, International Society
for Rock Mechanics and Rock Engineering

Dear distinguished guests, committee members, speakers, participants, ladies and gentlemen,

First of all, I would like to sincerely congratulate on your organization of the 1st International Conference on Space Utilization and Research in Underground Structures (SURUng-1), which is the first international meeting among its series in the field of underground structures. I thank the Organizing Committee for inviting me to make a keynote lecture in the Conference.

Nepal is a Himalayan country consisting of topographic, geological and geotechnical varieties. In my past two visits to Nepal, I travelled to lesser Himalayan areas and saw many challenges with high mountains and steep slopes which have high risk of rock fall, landslide and earthquake damage. In this natural location, there is high demand for tunneling and underground space utilization including hydropower system, transportation, water conveyance, utility lines and more. In this regard, the inauguration of the first international conference is very timely and of great importance.

For the past two and half years, the whole world has been suffering from COVID-19 pandemic. Now, the situation gets much better and we anticipant to go back to normal very soon. The SURUng-1 is the first offline international meeting in our 'rock mechanics community' since the pandemic outbreak. Time spotting of the Conference is excellent so that many participants meet together face to face. Big congratulations again and I hope this conference series will continue to provide the forum to share of knowledge and experience among researchers and practitioners.

Last but not least, I would like to acknowledge the great effort of the organizing committee to hold this Conference. I hope that all the participants will find this Conference fruitful and enjoyable.

MESSAGE FROM NTNU

Prof. Krishna Kanta Panthi
Department of Geoscience and Petroleum
Norweign University of Science and Technology

Department of Geoscience and Petroleum of NTNU and myself feel proud to be a part of the first ever conference titled "Space Utilization and Research in Underground Structures (SURUng-I)" that is being organized at the Kathmandu University in May 5-6, 2022. It is a proud moment to organize such event in collaboration between the Academic institutions like Kathmandu University (KU), Norwegian University of Science and Technology (NTNU) and the industrial and research partner "Hydro Tunneling and Research Pvt. Ltd. More importantly, the financial as well as participatory support received from organizations / projects such as Nepal Electricity Authority (NEA), NORHED II Project 70141 6; Capacity Enhancement in Rock and Tunnel Engineering in Nepal, Energize Nepal, Nepal Society for Rock Mechanics (NSRM) and Nepal Tunneling Association (NTA) makes this event even more meaningful. I should emphasize here that there is quite a lot to be learned here in Nepal within rock and tunnel engineering so that more innovative, cost-effective, sustainable and environmentally friendly solutions suitable to Himalayan rock mass are developed at the local level. The event as this will certainly help to move one step forward in this direction. My congratulations to the organizational team who worked hard to bring this event to reality. I on behalf of NTNU wish the organizing committee all the success in this endeavor.

MESSAGE FROM NSRM

Dr. Pawan Kumar Shrestha

President

Nepal Society for Rock Mechanics

Respected dignitaries, speakers, participants and everyone

I would like to warmly congratulate the SURUng-I team for organizing the 1st International Conference on Space Utilization and Research in Underground Structures (SURUng-1). I thank the organizing committee, primarily the Kathmandu University, Department of Civil Engineering, for inviting me to be one of the keynote speakers in this Conference.

A mountainous country like Nepal requires Tunnels and Underground Space for infrastructural development. Whether as waterways in hydropower schemes or as roadways in transportation system, tunnels contribute to development of the country. As the geology of Nepal composes of weak to competent rockmass, these varieties give rise to technical challenges when constructing underground structures. When tackling these challenges, it requires appropriate knowledge in the field of tunneling and underground space creation. For such purpose, not only academic learnings and professional practices, sharing of working experiences in design and construction of underground structures through such Conferences will help participants to gain knowledge and be benefited quickly.

I am sure this conference will greatly help the motivated attendees.

At last, I would once again congratulate everyone involved in making this conference a grand success. I look forward to subsequent conferences on this topic.

MESSAGE FROM CHAIR

Dr. Shyam Sundar Khadka

Associate Professor and Head

Department of Civil Engineering

Kathmandu University

With immense pleasure, on behalf of Team SURUng-I and the Department of Civil Engineering of Kathmandu University, I would like to welcome and thank all organising Partners, International Scientific Advisors, Co-sponsors, Supporters and Volunteers for successfully organising the First International Conference on Space Utilisation and Research in Underground Structures (SURUng-I) being held in Kathmandu University on 5th and 6th of May of 2022.

Tunnels, also known as 'Surung' in Nepali, are an indispensable aspect of Civil engineering; Especially in the mountainous and hilly terrain of Nepal where construction of roads without facing any sort of obstruction of hills is next to impossible. The authentic presence of the National and International dignitaries together can raise the conference dignity and arise an important discussion on the topic of tunnelling.

The history of tunnelling in Nepal dates back to 1917A.D with the construction of Churia tunnel near Hetauda. In recent times we are gaining this lost momentum with the announcement of a plan for the construction of 23 tunnels in Nepal. Among these, 4 are already under construction. This sudden ardour by the government of Nepal has sparked an excitement in the minds of tunnel enthusiasts and the general population.

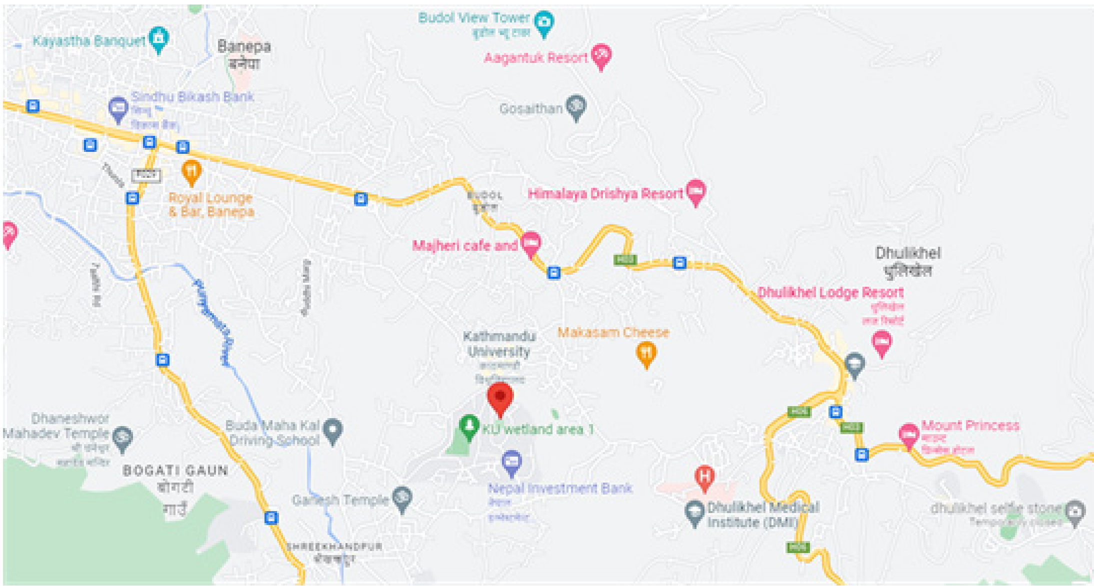
This is the first conference of its kind in Nepal that incorporates not only the governmental and non-governmental organizations but also people from every walk of life and Kathmandu university has played a key role in organizing this event. The conference will be addressed by distinguished personalities, keynote speakers and invited speakers. Technical sessions during the two day event will provide a cutting edge forum for networking between stakeholders, academics, students, professionals, industries and policy makers' alike and build up the commitment for the betterment of the future of tunnelling in Nepal.

SURUng-I brings together academicians, researchers, engineers, and industry experts involved in disciplines of Tunnels and Underground Structures, both from the National and International level. I am confident the knowledge shared in this conference will be of immense significance and a fruitful experience for all the attendees.

CONFERENCE VENUE

Kathmandu University, Dhulikhel, Nepal

SURUng-I will be held in Kathmandu University, Dhulikhel, Nepal. Established in 1991, the university's motto is 'Quality Education for Leadership'. Kathmandu University is located in the heart of Kavrepalanchok district, in between the cities Dhulikhel, Banepa and Panauti.



Location Map

Kathmandu University is located about 30 km east of Kathmandu. The university is 2.8 km from Banepa. Dhulikhel is 2.5 km away.

THINGS TO DO AROUND KATHMANDU UNIVERSITY

Dhulikhel Bazaar

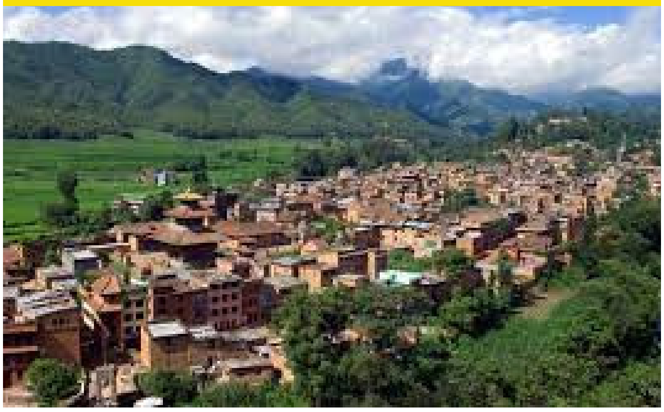
The main town of Dhulikhel is located 2.5 km from Kathmandu University. Dhulikhel's main attractions are the view of rows of snow-capped peaks, the Devithan hill rock renowned for viewing sunrise and sunset, historic Newar settlement and the artistic temples of the town. Devithan, Narayanthan Temple, Bhagawati Temple, Harisiddhi Temple along with the numerous parks are a must visit while nearby. You can also walk around the town and enjoy the beauty of Newari Architecture juxtaposed with narrow, colorful and vibrant streets.

Namo Bauddha Stupa

Namobuddha is one of the most important religious sites of the Buddhists after Kathmandu's, Swayambhunath and Boudhanath. It is located on a picturesque hill and can be reached along the Banepa or Panauti route. You can book a ticket at the bus park counter for Namo Buddha (a bus which goes up to Dhapcha passes through Namo Buddha, so you can take that bus as well). The bus goes from Banepa past the Dhulikhel bus station.

You can also trek from Banepa to Namo Buddha passing through Panauti and Sangkhu.

Panauti Bazaar



The ancient town of Panauti is located 6 km south of Banepa and 5km from Kathmandu University. The town carries huge religious and historical significance and the town, also proposed as a UNESCO world heritage site, is full of ancient temples, indigenous tradition and cultural heritage. The houses constructed in medieval designs, holy pilgrimage sites, artistic temples add to the attraction of this town.

The famous Makar Festival takes place at Panauti's Triveni, a confluence of three rivers, Punyamati, Lilawati and Rudrawati, every 12 years. Millions of devotees from different parts of the country visit Panauti during the festival.

ORGANIZING COMMITTEE

Conference Chair

Dr. Shyam Sundar Khadka, Assoc. Prof. and Head, Department of Civil Engineering, KU

Advisory Committee

Prof. Dr. Bhola Thapa, Vice Chancellor Kathmandu University

Prof. Dr. Subodh Sharma, Registrar, Kathmandu University

Prof. Dr. Manish Pokharel, Dean, School of Engineering

Mr. Brijesh Adhikari, Associate Dean, School of Engineering

Prof. Dr. Ing. Ramesh Kumar Maskey, Academician, NAST

Prof. Bhupendra Bimal Chettri, School of Engineering

Mr. Bhoj Raj Regmi

Scientific Committee

Prof. Seokwon Jeon, Elected President ISRM (2023-2027), Seoul National University, South Korea

Prof. Krishna Kanta Panthi, Norwegian University of Science and Technology, Norway

Prof. Dr. Ing. Ramesh Kumar Maskey, Academician, NAST

Prof. Tao Wang, Wuhan University, China

Dr. Gyanendra Lal Shreshta, President, Nepal Tunneling Association

Dr. Pawan Kumar Shrestha, President, Nepal Society of Rock Mechanics

Mr. Prem Krishna KC, Hydro Tunnelling and Research Center Pvt. Ltd.

Local Organizing Committee

Publicity and Web Service Team - Ramesh Adhikari, Rashmi Manandhar

Administrative Team - Manish Prakash, Sudip Bajgain

Logistic Team - Vimesh Paudel, Suman Shrestha

Registration Team - Kameshwar Sahani, Bimal Chhushyabaga

Technical Team - Santosh Chaudhary, Jenisha Dumaru

Finance Team - Megh Raj Shakya, Arun Deoju

Local Arrangement Team - Buddha Shrestha, Mahesh Raj Bhatt, Raju Bhele

General Secretary

Sujan Karki

PROGRAM SCHEDULE

Day I: May 05, 2022		
Opening Session		
8:30	Registration and Tea	
9:30	Seating of delegates	
9:35	Inauguration – Panas Lighting by chief guest	Chief Guest: Prof. Dr. Bhola Thapa, VC, KU
9:40	National Anthem followed by KU Anthem	
9:50	Cultural Dance	
9:55	Inauguration Speech	Chief Guest: Prof. Dr. Bhola Thapa, VC, KU
10:05	Welcome Speech	Dean, School of Engineering
10:15	Welcome Speech	Special Guest, Mr. Kulman Ghising, NEA
10:25	Welcome Speech	Nepal Tunneling Association
10:30	Welcome Speech	President, Nepal Society for Rock Mechanics
10:35	Keynote Speech I	Prof. Seokwon Jeon, SNU
11:20	Keynote Speech II	Prof. Krishna Kanta Panthi, NTNU
Lunch: 12:05 - 13:00		

Technical Session I ***(Numerical Modeling and Computer Application)***
Session Chair: ***Prof. Krishna Kanta Panthi***
Time: ***13:00-15:00***

SN	Paper Title	Authors
1	Prediction of strength and deformation of jointed rocks using ANN	Phibe Khalkho, Mahendra Singh
2	An Artificial intelligence-based method in underground rockburst hazard prediction	Prabhat Man Singh Basnet, Suresh
3	Study of the Mechanical Response of Lining Tunnel Support to Fault Movement at Different Orientation	Bimal Chhushyabaga, Sujan Karki, Shyam Sundar Khadka
4	Tunnel Intersection analysis using the Post-Peak Failure Approach	Bikram Thapa
5	Performance study of steel beams and reinforced ribs of shotcrete as tunnel support in weak grounds	Sujan Karki, Bimal Chhushyabaga, Shyam Sundar Khadka
6	Stability Analysis of Rock Slope from Nepal Lesser Himalaya	Nitesh Shrestha, Bikram Thapa

Tea Break: 15:00 – 15:15

Technical Session II ***(Rock Mechanics and Mining Engineering)***
Session Chair: ***Mr. Ram Hari Sharma***
Time: ***15:15 – 17:15***

SN	Paper Title	Authors
1	Technical Investigation of Tunnel Support Technology of Hydropower Tunnels Located in the Himalayan Region of Nepal	Sujan Karki, Bimal Chhushyabaga, Shyam Sundar Khadka, Pawan Kumar Shrestha, Krishna Kanta Panthi, Seokwon Jeon
2	Long-Term Impact on unlined tunnels of hydropower plants due to frequent start/stop sequences	Bibek Neupane
3	Uncertainty analysis of rock mass quality for a tunnel case from Nepal	Nitesh Shrestha, Krishna Kanta Panthi
4	Effect of Rock Pillar Width on the Stability of Rock Mass Around Large Cavern in the Himalayas	Abhishek Dongol, Prawal K. Khatri, Shyam Sundar Khadka
5	The influence of soil nail inclination on slope stability - A review paper	Jenisha Dumaru, Shyam Sundar Khadka
6	Seismic response analysis of underground structures in the Himalayan Region	Umesh Thapa, Shyam Sundar Khadka

Welcome Dinner
Time: 17:15 onwards

PROGRAM SCHEDULE

Day II: May 06, 2022		
Venue: CV Raman Auditorium		
8:30	Registration	
9:00	Welcome Speech	President, Nepal Society for Rock Mechanics
9:10	Welcome Speech	EnergizeNepal
9:20	Keynote Speech III	Dr. Sandip Shah, Immediate Past President, NTA
10:05	Keynote Speech IV	Dr. Pawan Kumar Shrestha, HTR
10:50	Tea Break	

Technical Session III (*Hydropower and Hydraulics)* Session Chair: *Dr. Ramesh Kumar Maskey, Mr. Prem Krishna KC* Time: *11:00 – 12:40*		
SN	**Paper Title**	**Authors**
1	Study on the Implementation of Direct Stiffness Matrix Method in Underground Framed Structures	Bini Neupane, Sandeep Shrestha, Kameshwar Sahani
2	Slope stability analysis using limit equilibrium: A case study of Mai cascade Hydropower Project	Manab Rijal, Pratik Tiwari, Aadarsha Shakya
3	Evaluation on unlined/shotcrete lined headrace tunnel for Tamakoshi V hydroelectric project	Kundan Chauhan, Krishna Kanta Panthi
4	Effect of damaged zones around an excavation due to blasting	Anim Shrestha, Abhishek Dangol, Shyam Sundar Khadka
Technical Session IV (*Tunnel Design and Construction Practice)* Session Chair: *Dr. Pawan Kumar Shrestha* Time: *11:00 – 12:40*		
1	Performance of Concrete Lining used in Tunnel Support by Physical Scaled Model and Finite Element Modelling	Bimal Chhushyabaga, Sujan Karki, Shyam Sundar Khadka
2	Finite Element Analysis of Underground Water Tank Considering Soil Structure Interaction and Water Sloshing Phenomena	Sabin Acharya, Hemant Thapa, Mahesh Raj Bhatt
3	Review paper on design of shotcrete lining	Sudip Bajgain, Shyam Sundar Khadka
4	Stability Analysis and Optimization of Support System for Cavern in Weak Rock Mass	Shrijan Dhakal, Samrit Dumre, Manoj Shrestha, Rabindra Karmacharya, Shyam Sundar Khadka
5	Presentation from NCEC	Mr. Siddharth Mani Rajbhandari
Lunch: 12:40 -13:30		
Plenary Session ***Title***: Importance of underground space in Nepal Himalayas for sustainable development ***Panelist***: Prof. Krishna Kanta Panthi, Prof. Tara Nidhi Bhattarai, Prof. Dr. Ramesh Kumar Maskey, Mr. Naresh Man Shakya ***Moderator:*** Dr. Pawan Kumar Shrestha ***Rapporteur:*** Mr. Bimal Chhushyabaga ***Time:*** 13:30 – 15:00		

Closing Session		
15:00	Start of closing session	
15:05	Closing Remarks	Dr. Rita Kumar, NTNU
15:10	Closing Remarks	KU Official
15:15	Token of appreciation	
15:20	Cultural program	
15:25	Vote of Thanks	Kathmandu University Officials
15:30	Tea Break and End of session	

Title: Prediction of Strength and Deformation of Jointed Rock Using ANN

Authors: Phibe Khalkho and Mahendra Singh

Affiliations: Civil Engineering Department, Indian Institute of Technology Roorkee, India

Corresponding e-mail: msingh.civil@gmail.com

Keywords: *ANN, Strength, Deformation, Jointed Rocks, Prediction*.

Abstract

Strength and deformation of jointed rocks are important parameters taken into account for design purposes of geotechnical structures such as underground tunnels, earth dams and embankments. Mostly, in the field, rock is found to be heterogeneous in nature and is encountered with joints, folds, sheared zones, bedding planes. In order to get the strength and deformation values, preparation of specimens is needed as per ISRM standards which sometimes becomes difficult for schistose rocks or when rock mass is weak, fragile and is highly anisotropic. To overcome this issue, non- destructive testing is done or based on the past empirical correlations on the basis of physio-mechanical properties are used. But these correlations have major limitations as they fail to reflect the effect of physio-mechanical properties on the dependent variable. Or these correlations are not universally applicable. In the recent years, artificial intelligence has become a sharp tool in the digital world for prediction. Such an example can be seen in the cell phones which just scan the unknown language and gives the prediction in a known language it is asked for. Artificial neural network (ANN) is a branch of artificial neural network which is widely being used in the geotechnical applications too (Maji and Sitharam,2008; Rafiai and Jafari, 2011; Rafiai et al.,2013; Kounda, 2014; Rukhaiyar and Samadhiya,2017; He et al, 2021). An application of Artificial neural network is presented in this paper to predict the strength and deformation behavior of jointed rocks when limited input parameters are known. Parameters such as P-wave velocity, orientation of joint and joint wall roughness. The database for training the neural network is formed from the uniaxial compression tests being conducted on model jointed specimens of Ultrarock material. The model jointed specimens are oriented at θ° equal to 0°, 15°, 30°, 45°, 60°, 75°. For each orientation, three distinct roughness values (JRC equal to 2-4, 12-14, 14-16) were introduced, according to Barton and Choubey (1977). Feed forward back propagation algorithm was used for training a three layered network. About 70 % of data was used for training and 30% of data was used for testing the data. It has been concluded that artificial neural network was found to be quite efficient in predicting the strength and deformation behavior of jointed rocks.

1. Methodology

After the specimens were prepared and cut and polished as per ISRM standards (1981) at Geotechnical Engineering laboratory of IIT Roorkee, India. P-wave velocity was measured using ULTRASONIC CONCRETE TESTER UX 4600M for all jointed specimens. The characterization was done and the mechanical properties of rocks are shown in Table1. The material is categorized in "DL" Deere-Miller classification, it belongs to low strength and of low modulus ratio. The jointed specimens were then tested in the Cyclic cum static rock triaxial test setup under uniaxial condition at a displacement controlled rate of 0.002mm/s. The results are plotted in the form of stress-strain curve. The tangent modulus is obtained by drawing the tangent at 50% of the elastic portion of the curve. For all the jointed specimens, peak strength, modulus values and P-wave velocities were obtained and plotted as shown in Fig.1. It was observed that UCS, Ej and Vp are inter related to each other. The relationship of Vp with θ, UCS with θ and Ej with θ are all following the same trend being minimum at θ=45° and 60° and higher at θ=30° and 75°.

Table 1 Mechanical properties of Model rock

Properties	Symbol	Value	Unit
Unit Weight	γ	21.3	kN/m^3
Cohesion	c	13.34	MPa
Angle of internal friction	ϕ	35.1	Degree
UCS of intact rock	σ_{ci}	46.55	MPa
Tangent Modulus	E_{t50}	9.09	GPa
Brazilian Tensile Strength	σ_t	3.14	MPa
Deere-Miller Classification (1966)	DL (Low strength and low modulus ratio)		

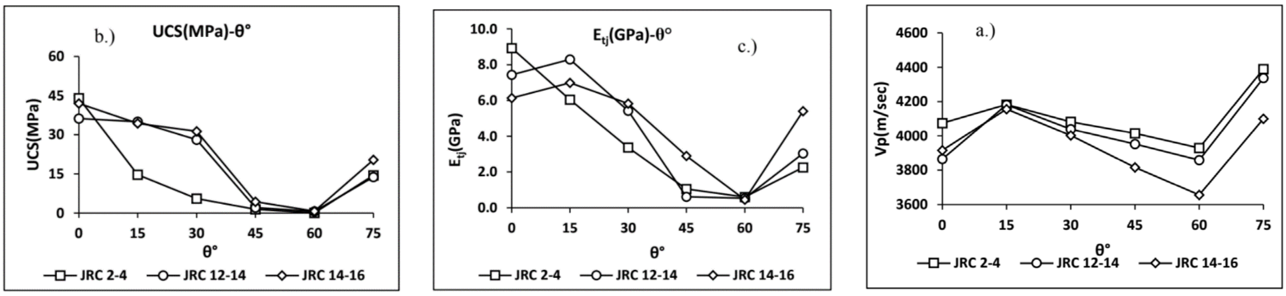


Fig 1 *Variation of a.) Vp with θ°, b.) UCS with θ° and c.) Etj with θ° for all three JRC's.*

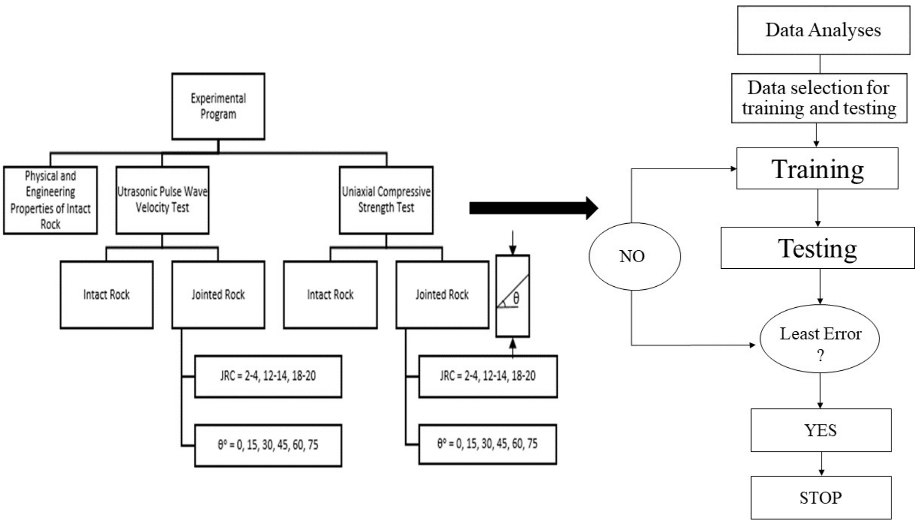


Fig 2: Flow chart showing the experimental procedure following it evaluation of a model.

The representative flowchart is illustrated in Fig.3. Once the database is prepared from the experiments conducted, about 70% of the data is trained and 30% of the data is tested in the ANN toolbox of MATLAB (2020b) by using the feed forward back propagation algorithm. The datasets were normalized from 0 to1 when LOGSIG transfer function was used and when the dataset was scaled from -1 to 1, the TANSIG transfer function was used. The Levenberg– Marquardt (LM) predicting efficiency is used in the study. The three layered network was featured by varying the input neurons from 2 to 3. In the first case, two input neurons were used, i.e, Jf and Vp. Jf accounts the overall effect of joint combining the frequency, joint orientation and roughness of the joint. For the second case, the JRC, θ° and Vp was used as the input neurons. A neural network as shown in Fig.4 for the prediction of one output i.e., σcj consisting of hidden neurons and input neurons when LOGSIG transfer function is used. Similarly, for the prediction of Ej, the neural networks developed are shown in Fig.5 respectively when 2 and 3 input neurons are used with LOGSIG transfer function. The weights and biases associated with the network were changed till the Mean Square Error (MSE) comes out to be minimum. The testing set is being compared with the target set in terms of regression value to check the efficacy of the network. Fig. 8-9 shows the performance validation curves and plots for regression during the training, testing and validation of ANN model for Ej when number of neurons are changed from 2 to 3 respectively (LOGSIG transfer function). The training is stopped when the criterion of maximum performance gradient is reached.

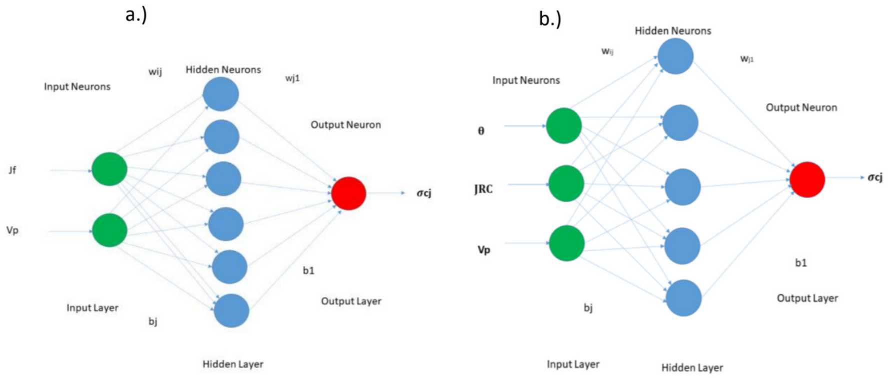


Fig. 3 *Neural structure developed for ANN for prediction of* σ_{cj} *when LOG sigmoid function was used when number of input neurons a.) 2 and b.) 3.*

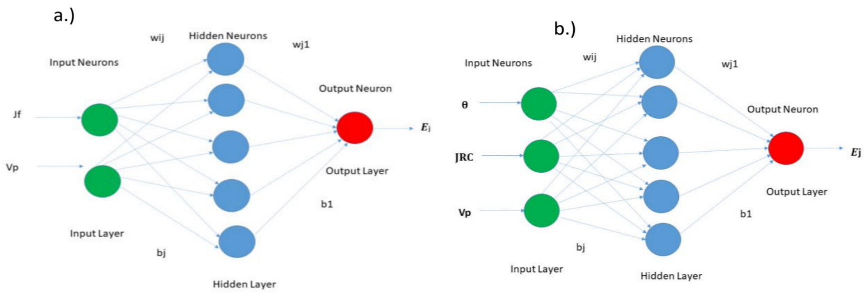


Fig. 4 *Neural structure developed for ANN for prediction of* E_j *when LOG sigmoid function was used when input neurons are a.) 2 and b.) 3*

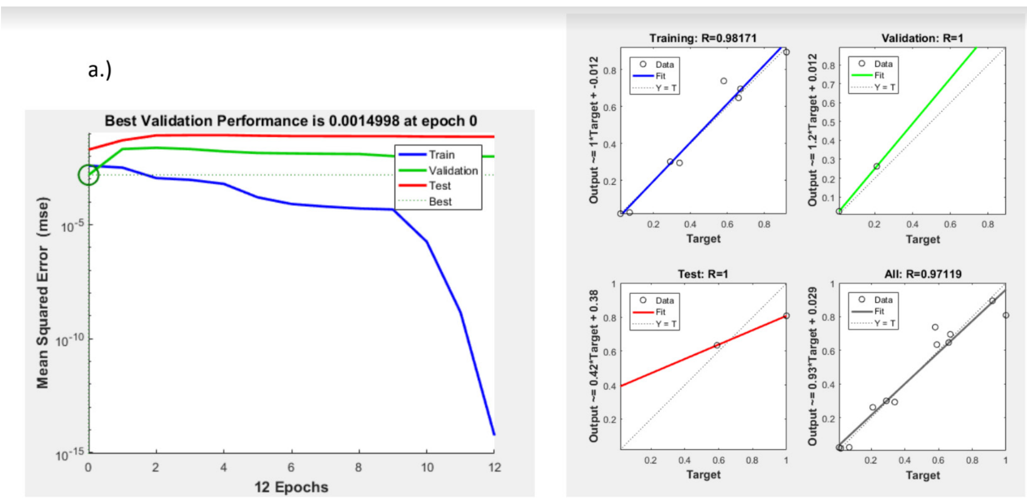


Fig. 5 *a.) Mean square error and best validation performance b.) Regression plot for Ej when LOGSIG transfer function is used for 2 input neurons*

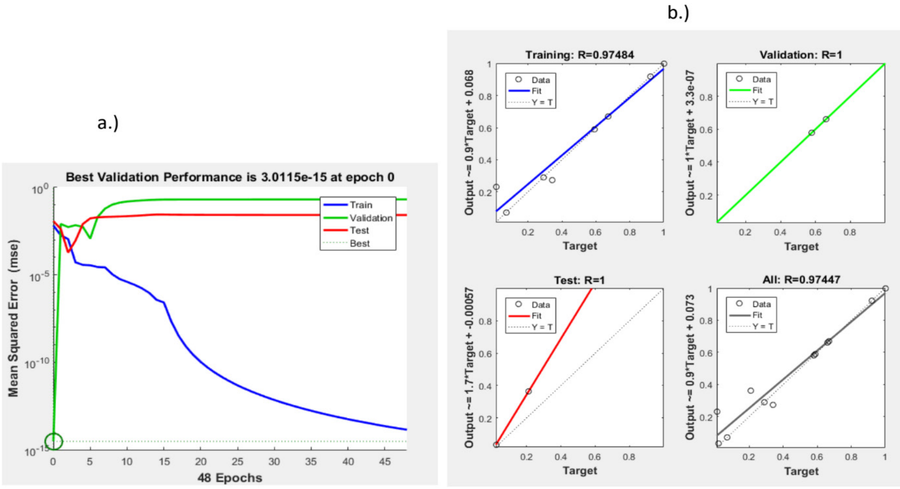


***Fig. 6** a.) Mean square error and best validation performance b.) Regression plot for Ej when LOGSIG transfer function is used for 3 input neurons*

Fig.6 a) and b) shows the relationship of predicted σcj and observed σcj when number of input neurons are 2 and 3 respectively when LOGSIG transfer function is used. It was observed that both the predicted and observed values are in good agreement with each other. The implicit trial and error is done to get the best regression value where both predicted and observed values are in agreement with each other. Similarly, Fig. 8 a) and b) gives the plot for predictedand observed value for prediction of Ej when number of neurons are 2 and 3 respectively.

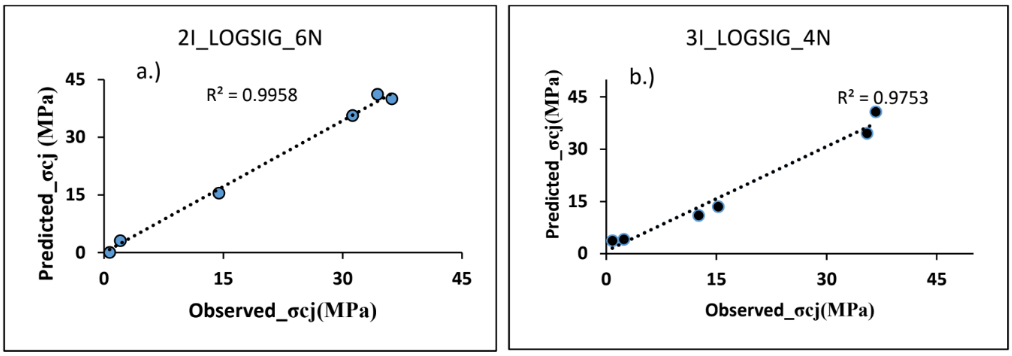


***Fig.** 7 Relationship of predicted σ_{cj} and observed σ_{cj} when number of input neurons are a.) 2 and when b.) numberof input neurons are 3*

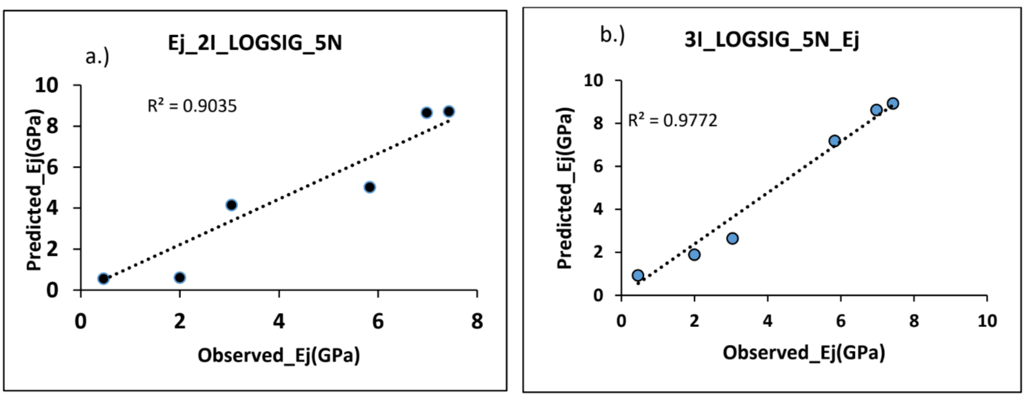


***Fig. 8** Relationship of predicted Ej and observed Ej when number of input neurons are a.) 2 and when b.) numberof input neurons are 3.*

2. Conclusions

Four neural network models were developed to predict the strength and deformation behavior of jointed rocks. Two neural models were developed for the prediction of σcj and other two were featured for the prediction of Ej using LOGSIG transfer function. For each network, three layered network consisting of input neurons linked with weights and biases to the next hidden neurons and then to the single output neuron. Weights and biases were modified until the mean square error is minimum. The trials and errors are made by varying the hidden neurons to get the maximumregression value for each case. It was observed that best prediction for σcj comes out with 6 hidden neurons when number of input neurons is 2 whereas for 3 input neurons, predicted and observed values are in good agreement when3 input neurons are used. Similarly, for the prediction of Ej, even though the number of input neurons are changed from 2 to 3, the hidden neurons remain same on using LOGSIG transfer function but the good agreement comes out when number of input neurons are more. ANN has been disapproving the fact that it is a black box but has been foundto be efficient enough to predict the strength and deformation behavior of jointed rocks. The testing data should be set carefully so that it falls in the range of training data as ANN does not extrapolate. Further, for the present study, less number of datasets were used. It is encouraged that ANN gives much better prediction when large database is available. Various predictive efficiencies can be varied to get the best results.

3. References

He M, Zhang Z, and Li Ning. Deep Convolutional Neural Network-Based Method for Strength Parameter Prediction of Jointed Rock Mass Using Drilling Logging Data. International Journal of Geomechanics. 2021:21(7). doi/10.1061/%28ASCE%29GM.1943-5622.0002074

Kaunda R. New artificial neural networks for true triaxial stress state analysis and demonstration of intermediate principal stress effects on intact rock strength. J Rock Mech Geotech Eng. 2014;6(4):338–347.

Maji VB, Sitharam TG. Prediction of elastic modulus of jointed rock mass using Artificial Neural Networks. Geotech Geol Eng. 2008;26:443–452.

Rafiai H, Jafari A. Artificial neural networks as a basis for new generation of rock failure criteria. Int J Rock Mech Min Sci. 2011;48(7):1153–1159.

Rafiai H, Jafari A, Mahmoudi A. Application of ANN-based failure criteria to rocks under polyaxial stress conditions. Int J Rock Mech Min Sci. 2013;59:42–49.

Rukhaiyar, S., and N. K. Samadhiya. A polyaxial strength model for intact sandstone based on Artificial Neural Network.International Journal of Rock Mechanics and Mining Sciences 95 (2017): 26-47

Title: An Artificial intelligence-based method in underground rock burst hazard prediction

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Keywords: *Machine learning, Non-linear SVM, rockburst prediction*

Abstract

Rock burst is common geological hazard often encountered in Mining and tunneling engineering that causes damage to an excavation and lead to casualties, hence prediction of its occurrence is a crucial task. However, due to complex mechanism and development process of rockburst it is always hard to reliably predict the rockburst severity manually. Therefore, this paper introduces artificial intelligence based machine learning (ML) approach using Nonlinear support vector machine to predict the rockburst intensity. Four indices including tangential stress, Stress ratio(SR), Brittleness ratio(BR) and elastic strain energy are considered as an input indicator to create a prediction model where 108 rockburst cases from various internationally published article has been collected and used as a training and validation sample. After data standardization, cross-validation and hyperparameter optimization the accuracy of the model reached 86% to reliably predict the rockburst intensity level in an intelligent way. This approach is feasible in replacing manual effort in rockburst prediction task.

Title: Study of the Mechanical Response of Lining Tunnel Support to Fault Movement at different Orientation

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Keywords: *Tunnel, Fault, Elastic-plastic failure model, Strain softening model, Plastic, Elastic Zone*

Abstract

Fault is type of a discontinuity which effects the stability of the tunnel support system and surrounding rock massaround tunnel. A good understanding of mechanical behavior of surrounding rock mass of tunnel is vital to have adequate and well performing support system. The orientation, distribution of fault with respect to the tunnel opening and its thickness of fault play important factors resulting critical stresses and deformation in tunnel and its surrounding rock mass. Therefore, Finite Element Models are developed to study the influence of fault in tunnelsupport in which location of the fault, and its orientation, are varied with respect to tunnel opening. These modelshave been developed for three different cross sections i.e., Circular, Inverted-D and Horseshoe.

For mechanical response of effect of fault in support system, six segments in tunnel lining i.e., Crown, Left Shoulder, Right Shoulder, Invert, Right Side Wall, and Left Side Wall have been defined. The influence has beenstudied in terms of displacements field, plastic zone in rock mass and internal forces such as shear force, axial force and bending moment in tunnel support. Fault has been placed at the perpendicular distance of 'Ds' from a section of the tunnel with fault thickness 'T'. For example: if a fault is located in crown segment, it has been defined as the crown fault. Strength properties of rock mass include tensile strength, compressive strength, *frictional angle,* and cohesive strength. The displacement of fault has been taken as 120 mm on the basis of averagerate of displacement given by GPS measurements after 2015 Gorkha Earthquake. Elastic-plastic failure model hasbeen used for modelling of tunnel and surrounding faulted rock mass. The displacement and stress in key points such as crown, invert, shoulders, walls on the tunnel lining are chosen to be analyzed as the representative of the displacement and stress of the tunnel.

In the absence of fault, the displacement of the surrounding rock mass reduces gradually from the tunnel lining tothe boundary of the outer surrounding rock mass and becomes zero on the boundary. With presence of a fault, thedisplacements of the surrounding rock mass have discontinuous nature. It is concentrated more to the tunnel and fault boundary. The displacement of tunnel and rock mass surrounding tunnel are more directed towards the movement of the fault. From the results obtained from the displacement, crown fault is more critical. When the fault is located in crown of the tunnel the displacement is maximum in the crown of the tunnel. The tunnel liningbecomes critical in terms of displacement, shear force, axial force and bending moment when there is presence offault. However, it is not proportional for all the sections i.e. a fault that is located in specific place does not increasethe parameter of stress or displacement in same place. For example, the displacement has increased in circular andhorse shoe tunnel in crown, right shoulder and left shoulder when the fault is located crown, right shoulder and left shoulder respectively. However, it is not in the case of inverted D tunnel. Similarly, the shear force, bending moment and axial force in the tunnel lining don't increase in the place where the fault is located. For example, incase inverted D tunnel, there is maximum shear force at right bottom and right-side wall due to right bottom fault.Similarly, in case of horseshoe tunnel, shear force in left bottom is maximum due to left side wall fault and in caseof circular tunnel left bottom has maximum shear force due to left side wall fault. It is similar with the case of bending moment, displacement and axial force in all three-cross section of tunnel.

Title: Tunnel Intersection analysis using the Post- Peak Failure

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Keywords: *Underground space Utilization, Tunnel intersection, Pillar, Strain-softening, FEM*

1. Introduction

Tunnel intersection or Junctions are a recurring part of any underground project. Due to different purposes and safety reasons tunnel junctions are used in underground designs. The part of rock massmade available between adjoining tunnels or openings are commonly known as pillars(Mortazavi, Hassani and Shabani, 2009). Intersections are also very commonly adopted in Hydro and road tunnels. Junctions thus createdshow some form of stress concentration and in some cases, failure zones are observed in the pillarseparating both openings. Quantifying the nature of failure at such sections is extremely difficult dueto limited and unreliable information regardinggeological features, orientation, and in-situstresses. To compensate for the shortcomings many empirical methods are used on day to day basis in most tunnel design cases in Nepal and across the rock engineering community, with someadded numerical analysis. This paper, thus, tries tostudy tunnel junction cases from Eastern Nepal in the lesser Himalayas and provide the possible reasons for the phenomenon by using the availablemethods and criteria.

2. Post Peak Failure Approach and Numerical analysis

A 2d continuum phase^2 model is used for analysis. Phase^2 is an implicit Finite Element method (FEM) from Rocscience (Rocscience, 2011). Post peak failure conditions in Generalised Hoek and Brown can be addressed by using residual parameters. GSI is used to differentiate the nature of failure mode and residual parameters in the analysis. There are three main modes of rock mass behavior as per GSIvalue: Brittle (GSI>75), Strain Softening (25<GSI<75) and perfectly plastic (GSI<25).

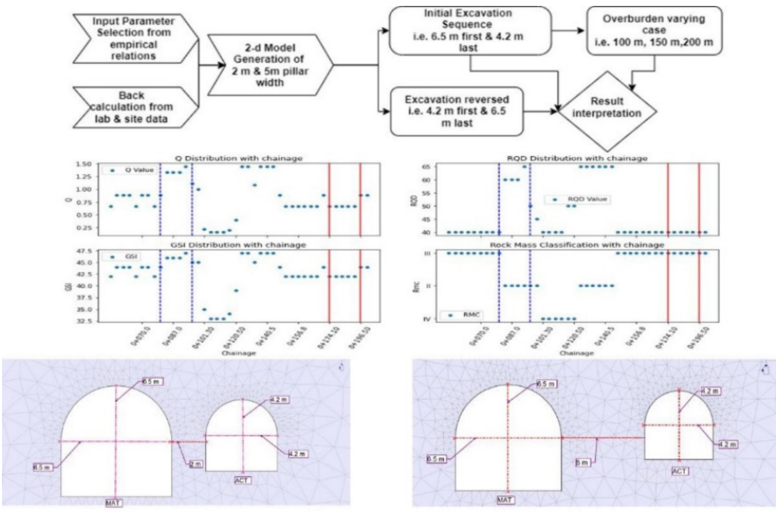


Figure 1 *Design Work Flow, Field data for interpretation and Phase 2 model*

Rock Mass	Designation / Chainage (m)	Peak	Residual
σ_{ci} (MPa)	Intact compressive strength of rock	40±4	
GSI	Geological Strength Index	41±4	24±2*
Mi+	Intact Rock Constant	20±3	20±3
M_b**	Material constant	1.687±0.2	1.17±0.2
S***	Constant	0.0003833±0.0002	~0
a!	Constant	0.5	0.5
D	Disturbation Factor	0.5	0
υ	Poisson Ratio	0.26	-
γ (MN/m³)	Specific weight	2.78	-

*$GSI_r = GSI*e^{-0.0134*GSI}$ (Cai et al., 2007), +$Mi = \sigma_{ci}/\sigma_t$,

** $mb = mi * exp(\frac{GSI-100}{28-14D})$

*** $s = exp(\frac{GSI-100}{9-3D})$

! $a = \frac{1}{2}+\frac{1}{6}(e^{-\frac{GSI}{15}} + e^{-20/3})$

Overburden(m)	(σv)Mpa	(σh-inplane) Mpa	(σh-outplane) Mpa	K
100	2.7	1.69	3.19	0.63
150	4.05	2.17	3.77	0.54
200	5.4	2.64	4.14	0.49

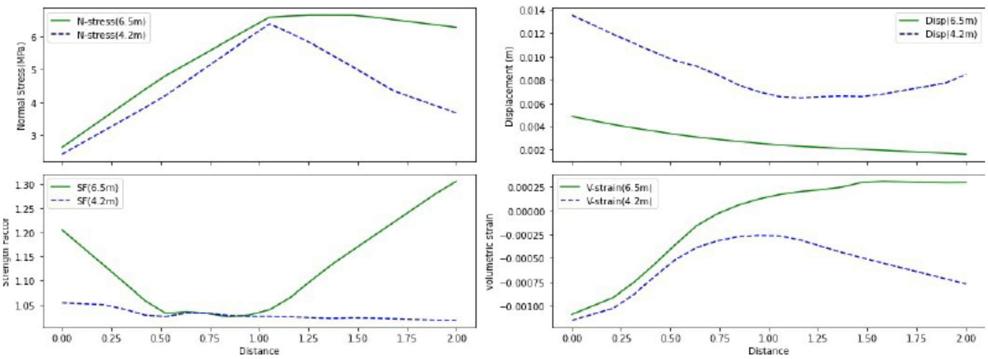
- Input Parameters are based on lab field observation and empirical relations
- In-situ stress and tectonic stress is based on back calculation

***Figure 2** Input parameters for the Phase 2 modelling*

3. Results

Stress redistribution effect on 2 m intersection

***Figure 3** Stress redistribution in 2 m wide pillar*



Overburden variation case on 2 m intersection

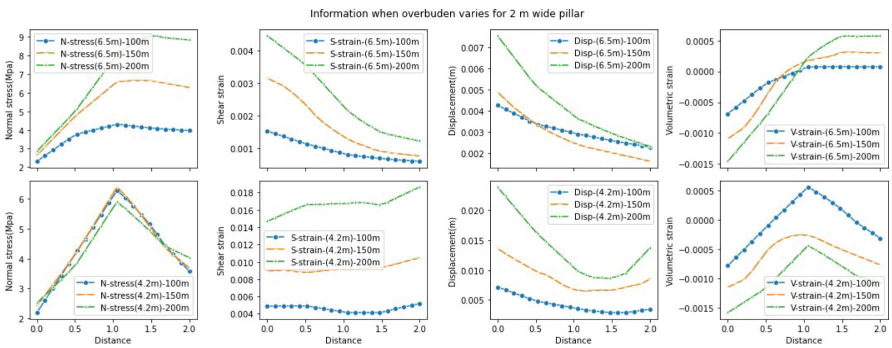


***Figure 4** Stress redistribution along with different overburden for 2 m pillar width*

4. Conclusion and recommendation

The stress re-distribution effect was very sensitive with the 2m wide pillar as compared to 5m wide pillar. Overburden variation also proved that sufficient pillar depth is very important for higher confinement.

- The excavation sequence impacts stressdistribution path. Larger excavation in thefirst phase is better than smaller ones in small pillars, but it was similar with biggerpillar.

- 2-D implicit method is not equipped to demonstrate the stresses unloading mechanism properly. Other explicit or implicit codes for critical pillar sections that can capture the dynamic unloadingconditions is required.

- As a thumb rule, angle between two tunnels bifurcating should be obtuse tohave sufficient pillar width

- Modeling on intersections with different scenarios reflects that the same type and quantity of support system may not berequired for stability reasons.

5. References

Alejano, L. et al. (2012) 'Impact of post-failure rock mass behavior on excavation response', American Rock Mechanics Association, (July 2014).

Alejano, L. R., Rodríguez-Dono, A. and Veiga, M. (2012) 'Plastic radii and longitudinal deformation profiles of tunnels excavated in strain-softening rock masses', Tunnelling and Underground Space Technology, 30, pp. 169–182. doi: 10.1016/j.tust.2012.02.017.

Cai, M. et al. (2007) 'Determination of residual strength parameters of jointed rock masses using the GSI system', International Journal of Rock Mechanics and Mining Sciences, 44(2), pp. 247–265. doi: 10.1016/j.ijrmms.2006.07.005.

Cai, M. (2008) 'Influence of stress path on tunnel excavation response - Numerical tool selection and modeling strategy', Tunnelling and Underground Space Technology, 23(6), pp. 618–628. doi: 10.1016/j.tust.2007.11.005.

Cai, M. (2009) 'Practical Estimates of Tensile Strength and Hoek – Brown Strength Parameter mi of Brittle Rocks Practical Estimates of Tensile Strength and Hoek Brown Strength Parameter m i of Brittle Rocks', Rock Mechanics and Rock Engineering, (May). doi: 10.1007/s00603-009-0053-1.

Cogan, J. P. (1950) The Mechanics of Rock Failure. Colorado School of Mines.

Crowder, J. and Bawden, W. (2004) 'Review of Post- Peak Parameters and Behaviour of Rock Masses: Current Trends and Research', Rocscience website, (January), pp. 1–14.

Hoek, E. and Brown, E. T. (1997) 'Practical estimates of rock mass strength', International Journal of Rock Mechanics and Mining Sciences, 34(8), pp. 1165–1186. doi: 10.1016/S1365-1609(97)80069-X.

Lorig, L. and Varona, P. (2013) 'Guidelines for numerical modelling of rock support for mines', Australian Centre for Geomechanics, pp. 81–105. doi: 10.36487/acg_rep/1304_04_lorig.

Mortazavi, A., Hassani, F. P. and Shabani, M. (2009) 'A numerical investigation of rock pillar failure mechanism in underground openings', Computers and Geotechnics, 36(5), pp. 691–697 10.1016/j.compgeo.2008.11.004.

Nepal, K. M. (1999) 'A review of in-situ testing of rock mechanical parameters in hydropower projects of Nepal', Journal of Nepal Geological Society, 19, pp. 1– 8.

Panthi, K. (2006) Analysis of Engineering Geological Uncertainties Related to Tunnelling in Himalayan Rock Mass Conditions, Norwegian University of Science and Technology.

Panthi, K. K. and Basnet, C. B. (2018) 'A dynamic analysis of in-situ stress state at the Upper Tamakoshi Hydroelectric Project area, Nepal', Hydro Nepal: Journal of Water, Energy and Environment, 23(August), pp. 42–47. doi: 10.3126/hn.v23i0.20824 .

Rocscience, I. (2011) 'Phase 2'. Canada, Toronto. Available at: https://www.rocscience.com/software/rs2.

Shrestha, P. K. and Panthi, K. K. (2015) 'Interpretation of deformation characteristics at kaligandaki headrace tunnel using tunnel monitoring records', 13th ISRM International Congress of Rock Mechanics, 2015- MAY(May).

Thapa, B. and Joshi, J. (2021) 'Review of Underground Space Utilization and its Relevance in Nepal', Journal of Engineering Technology and Planning, 2(1), pp. 46– 54. doi: 10.3126/joetp.v2i1.39228.

Vakili, A. (2016) 'An improved unified constitutive model for rock material and guidelines for its application in numerical modelling', Computers and Geotechnics, 80, pp. 261–282. doi: 10.1016/j.compgeo.2016.08.02

SURUng-I
First International Conference on
Space Utilization and Research in Underground Structures
(May 5-6, 2022)

Title: Performance study of steel beams and reinforced ribs of shotcrete as tunnel support in weak grounds

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Keywords: *Performance study, Reinforced Ribs of Shotcrete, Rock Mass Classification, Capacity diagram*

Abstract

The geographical setting and rock quality of the Himalayas presents both challenges and opportunities for underground construction works. With limited studies in the region, the tunnel and underground caverns follow the empirically established Rock Mass Classification (RMC) approaches – namely the Rock Mass Rating (RMR) and Rock Quality Index (Q). Although RMCis used for classifying the rock mass, the design and estimate of supports however uses combination of both the methods.

Among the two methods, the Q-system is widely adopted in Nepal due to its wide range of rock mass class (from Exceptionally Good to Exceptionally Poor) and its flexible supports. However, the method is only used as a basis for characterizing the rock mass and the support is changed according to the need and the experience of the designer. Due to this reason, the Reinforced Ribs of Shotcrete (RRS), defined in the Q-system, is seldom used for tunnel support in the region. *Instead, steel* ribs are extensively used and concrete is placed as a final lining in weak section. Thisarticle, thus, studies the differences in the performance of RRS and steel ribs, composite with shotcrete, in terms of their mechanical behavior. Capacity diagrams, an established technique of structural analysis and concrete design, has been used to study the difference in their behavior. Two composite sections: Steel sets and Shotcrete (S&S) and Reinforced Ribs and Shotcrete (R&S)have been studied according to the equivalent section method. According to the equivalent sectionmethod, a single homogeneous section is assumed which has equivalent properties as that of the composite section.

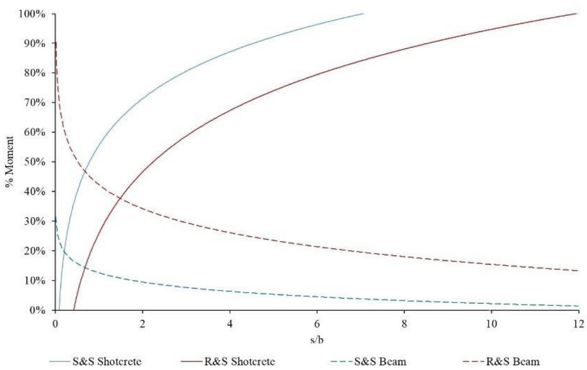

Figure 1 *Percent distribution of moment in individual elements of the composite supports.*

Overall, it was seen that the moment over the S&S composite is less than the moment over the reinforced R&S composite. From the figure above we see that the shotcrete element takes higher moment in case of S&S composite and correspondingly the beam elements take lesser moment. The opposite is also true for the R&S composite. This shows that the load sharing between individual elements is more uniform in case of R&S composite. The steel beams only take minimum amount of the bending moment. It was also seen that the percentage of load taken up by the individual element in the composite structure is independent of the amount of disturbance to the rock mass i.e. the percentage of load acting on an element is same for a given loading configuration and not affected by the Disturbance factor (D) value. The maximum value of moment, although, increases as the ground disturbance increases (for higher value of D).

Title: Stability Analysis of Rock Slope from Nepal Lesser Himalaya

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Keywords: *slope, instability, surge, support, Himalaya*

1. Introduction

Nepal lies in north-central position in South Asia andis sandwiched between two giant countries in the world,India in the south and China in the north. It occupies onethird of 2500 km long Himalayan arch. Being a mountainous country, it has tremendous potential inhydropower development due to steep terrain and fast flowing rivers. Nepal, being a developing country requires a long term & sustainable development of hydropower projects, and cost effective option for waterway is construction of underground tunnels in thesediverse geographical orientations. Nepal Himalayas has young and fragile geological formation which results challenges in underground construction. Despite various geological uncertainties related with tunnels, slope stability has been recurring and hazardous phenomenon in case of Himalayan geology. There has been many instances related to slope and landslide cases in Nepal which has incurred human and property losses every yearin various parts of the country. The stability of slope is dependent on various factors such as slope topography, discontinuity orientation, ground water pressure, in-situ stress condition and seismic activities. Himalayan geology is mostly fragile and comprised of weak rock. and thick soil cover. When such land mass is exposed to torrential rainfall event, occurrence of slope and landslide is more likely. Monsoon rainfall in Nepal mostly equates 80% of total annual rainfall, of which nearly 37% occurs within 24 hours [12]. Authors Dahal et al. [13] has reported that shallow and deep-seated slides occur in monsoon period and are considered the result of torrential rain. In general, shallow depth slides are due to the short-term rainfall, whereas deep seated slides are due to the long rainfall period, which does not seem to be the case with Nepal. During Mid October 2021 torrential rainfall occurred recording rainfall of 200-250 mm [14] near the project study area (Figure 1), which author Dahal et. al [13] inferred as the threshold for shallow seated slide in the area with steeper slope. During this period the cracks were seen first in the surface and then inside the tunnel along surge shaft area. In addition to that many other minor and large slides were observed around Mai Khola and throughout the Eastern Nepal, destroying and hampering many hydropower projects, road and human settlement.

2. Analysis of Slope Stability

2.1 Traditional Limit Equilibrium Analysis

For limit equilibrium analysis, the surge pipe slope has been quantified with the geometry and acting forcesas shown in Figure 10.

$$\Upsilon_r = \text{Sp. Weight of rock mass} = 26 kN/m^3$$

$$\Upsilon_w = \text{Sp. Weight of water} = 10\ kN/m^3$$

2.2 Numerical Modeling od Surge Pipe Slope

A 2D topographic model has been modeled by applying Generalized Hoek and Brown failure criteria. Avalley model along surge pipe was modeled before and after support installation. Excavation of 30 m long tunnelsection is carried out with short round length of 1 m each.Therefore 30 stages are used during analysis which has advantages of analyzing tunnel stability in each stage. Table 1 shows mechanical properties of rock mass underneath surge pipe.

Table 1: Mechanical properties of rock mass underneath Surge Pipe [2]

Initial element loading	Field stress Gravity Type		
Material	Colluvium	Banded Gneiss	Banded Gnesis
Weathering Condition	-	Highly weathered	Weathered
Specific Weight of Rock	18 kN/m³	26 kN/m³	27 kN/m³
Elastic type Insitu Stress (σ_o)	Isotropic	Isotropic	Isotropic
Intact Compressive Strength	25 MPa	40 MPa	40 MPa
Young's modulus	0.732 GPa	3.98 GPa	4.15 GPa
Poisson ratio	0.30	0.25	0.25
Peak Angle of Int. Friction	35 degree	35 degree	35 degree

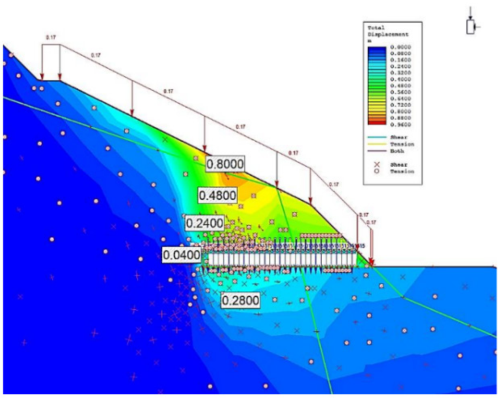

Figure.1 *Total displacement around tunnel after support installation (Stage 30)*

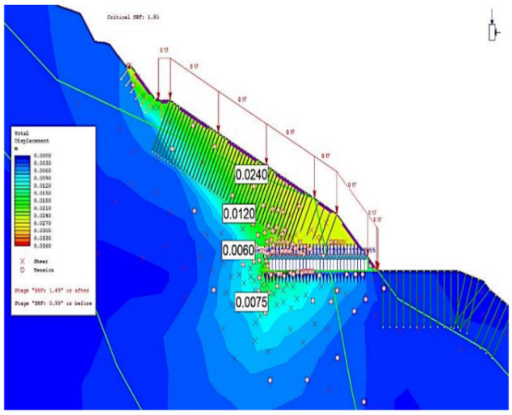

Figure.1 *Maximum displacement variation (with recommended support)*

3. Controlling Slope instability

A thorough study has been done for problems regarding slope stability along surge pipe in this hydropower project and have come to conclusion thatthe striking problem is the poor geology. The alignment has significant colluvium and highlyweathered rock mass has triggered slope instability. Now, that it is not the preliminary phase of project, so surge pipe slope alignment cannot be changed. Therefore, the only way is to modify existing supportsystem by increasing its stiffness. . It has been shownthat the modification of the support can stabilize slopeas shown in Table-2.

Table-2 Support system recommended along Surge Pipe

Recommended Support around Surge pipe slope
Soil nail of 4 m long @2 x 2 m on the crown Soil nail of 12 m @1 x 1 m on the body Soil nail of 8 m @2 x 2 m on the toe Wire mesh shotcrete 100 mm thick over the slope

4. Conclusion

Based on the actual slope stability issue at surge pipeslope at Maibeni Hydroelectric Project, the two dimensional limit equilibrium analysis and the FEM strength reduction method are performed for stability evaluation. From analysis, it is observed that ifcolluvium deposit thickness varies depth of slip circle also varies. Since, the structure has already been constructed in site, it is not possible to remove colluvium deposit underneath structure. Due to this constraint, it is recommended to introduce soil nail of 8-12 m long @ 2x2 pattern in combination with wire meshshotcrete of thickness 100 m with gentle sloping on leftand right side of the surge pipe arrangement so as to break the slip circle. In case of slope containing surge pipe, it is recommended to use soil nail of 4 m long @2x 2 m on the crown of slope. Just beneath it, it is recommended to use 12 m long soil nail spacing 1 by 1to increase FS. If it is not possible to install this soil nail,the length should be at least 8 m long. This constraint may occur around slope just above the HRT-2. For thesesections, 12 m long soil nail with slightly gentle inclination may be carried out. It must also be noted thatcutting slope is maintained with necessary weep holes, berm that has self-draining capacity to drain away fromthe working face.

5. References

Panthi, K.K., & Nilsen, B. (2006). Numerical analysis of stresses and displacements for the Tafjord slide,Norway. Bullitin of Engineering Geology and the Environment 65: 57–63

Fawaz, A., & Farah, E. (2014). Slope Stability Analysis Using Numerical Modelling, American Journal of CivilEngineering 2(3):60-67

Samling Power Company Pvt. Ltd (2018), Geological report on Maibeni Hydropower Project, Nepal. Detail Design Report.

Panthi, K. K. (2006). Analysis of engineering geological uncertainties related to tunneling in Himalayan rock mass condition. Norwegian University of Science & Technology, Department of Geology & Mineral Resorces Engineering. Norway: Norwegian University of Science & Technology.

Rocscience. (2016). 2D finite element analysis. Version: 9.008, User Manual. Rocscience Inc.

Upreti BN, L. F. (1999). Lesser Himalayan crystalline nappes of Nepal: problem of their origin. Geological Society of America, 225–238.

Nilsen, B. (2016) Rock slope stability analysis according to Eurocode 7, discussion of some dilemmas with particular focus on limit equilibrium analysis.

Bhandary, N. P., Tiwari, R. C., Yatabe, R., & Paudel, S. (2018). SEM-based seismic slope stability and mitigation model for the Jure landslide after the 7.8 Mw2015 Barpak-Gorkha, Nepal, earthquake. In Geotechnical Earthquake Engineering and Soil Dynamics V: Slope Stability and Landslides, Laboratory Testing, and In Situ Testing (pp. 88-97). Reston, VA: American Society of Civil Engineers.

Dahal, R. K., Hasegawa, S., Masuda, T., & Yamanaka, M. (2006). Roadside slope failures in Nepal during torrential rainfall and their mitigation. Disaster mitigation of debris flows, slope failures and landslides,503-514.

DHM, 2021, Precipitation Maps of Nepal, Department of Hydrology and Meteorology, Kathmandu, Nepal

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Title: Technical Investigation of Tunnel Support Technology of Hydropower Tunnels Located in the Himalayan Region of Nepal

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Keywords: *Himalayas, Rock Mass Classification, tunnel supports*

Abstract

The geological setting of the Himalayas provides opportunities for development of low to high head hydropower projects. The mountainous terrain demands tunnel and underground structures for space utilization and development works. However, the challenge lies in safe and economic construction of such structures. Fragile nature of the Himalayan geology along with the presence of various faults and discontinuities has created problems of squeezing, rock collapse and block failures in addition to the other problems created by groundwater and high stress localization. With the increasing construction of underground structures, it has become necessary to properly analyze the stabilization techniques utilized in the region. From observation, it is seen that Rock Mass Classification (RMC) approaches are used for classifying the rock mass and supports are designed accordingly. These method makes use of the Rock Mass Rating (RMR) and Rock Quality Index (Q) – especially Q-system, in account to its flexible supports and wide range of classification – from Exceptionally Poor to Exceptionally Good. From various case histories in the Nepal, it is seen that most of the constructed hydropower tunnels, fall in the Lesser Himalayan Region of the country which has the presence of weak rock mass including phyllites, schist, gneiss, phyllitic schist, etc. The support technologies used in these tunnels differs from the empirically established Q method. It is evident, these supports are adapted according to the encountered rock mass, based on the experience of the field engineer and designer. The lack of numerical modeling practice has also led to failure of these 'modified' support in many cases. This paper thus summarizes the practiced support techniques used in the Himalayan region based on various case studies and performs numerical modeling study to analyze the performance of the supports. It is observed that the supports used in the Himalayas mainly consists of steel ribs, thick concrete lining and forepole/spiling bolts, in addition to shotcrete and rock bolts. The construction technique generally follows the 'build as you go' approach as defined by the New Austrian Tunnelling Method (NATM). Overall, it can be said that due to lack of a concrete guideline, the construction and support techniques follows a hybrid of various approaches given the huge uncertainty and challenges in the Himalayas.

With various cases from the Nepal Himalayas, a database has been prepared for analysis and numerical modeling has been adopted to access the rock support in the region. A modeling framework has been prepared to model the weak rock mass with various supports. Pre-excavation support, such as forepole/spiling bolts, is essential for construction in weak rock mass and should be encorporated in the numerical model to optimize the support requirement. However, pre-support is a 3D problem and due to limitiation of computational power and time, such complex model are rarely adopted in professional practice. Hoek, proposed an alternate to model the forepole in 2D model by introducing an improved zone around the rock mass which has been successfully adopted in the weak rock mass of the Himalayas. A methodology has been developed for the modeling along with highlighting the importance of sequential support installation in the model.

For the weak rock masses, the support recommended by the Q chart has been adapted for the Himalayan region. The supports commonly used in the region has been used. Table 1 shows the changes in the support recommendation from Q system. It is note that the support provided below is only to provide a reference for support design in the region. Yielding liner has also been utilized to stabilize the rock mass in extremely squeezing ground. A case study of Chameliya headrace tunnel has been taken to verify the model which was in agreement with the final support installed in the section. Figure 1 below shows the difference in adopting a liner with and without yielding liners. It can be seen that the same support gives safe result when yield zone is introduced in the support. A final criteria (Figure 2) has been prepared based

on the various sections of different tunnels from Lesser Himalayan region for the introdution of circular section and yieldingliners. Based on stress (σ_{cm}) to strength (P_o) ratio it was seen that circular excavation gives optimum support for σ_{cm}/P_o less than 0.15 and yielding support gives optimum support for σ_{cm}/P_o of less than 0.1.

1. Introduction

The geology and mountainous topography of Nepal provides opportunity for underground structures for the purpose of transportation, water divergence and storage. More than 280 kilometers of tunnels have been constructed in the country with more than 500 kilometers under construction. Tunnels and underground structures are mostly built for hydropower developments in the country and its use for transportation is slowly gaining momentum. However, the construction of the structures comes with its challenges. Since, the Himalaya is fragile and seismically vulnerable, the region consists of various faults and discontinuities. Geologically, three major faults pass through the Nepal Himalayas - Main Boundary Thrust (MBT), Main Central Thrust (MCT) and Main Frontal Thrust (MFT). These faults have created several construction challenges from excavation collapse, squeezing to rock bursts.

Nepalese terrain and geomorphology along with abundant water resources are major factors suitable for hydropower development. Due to energy crisis and growing demand of electricity, development of hydropower seems a sustainable remedy. Numerous hydropower tunnels have been built and new tunnels are planned in the Himalayan region as one of the main components of hydropower development. Tunnel stability needs a special attention in a mountainous country, because of the diverse geology and difficult terrain of the country. Due to the mountainous topography, high overburden pressure are created in the underground structures producing squeezing and other stability problems. This geological difficulty results in a huge financial loss because of heavily reinforced support in the squeezed sections. However, due to few detailed studies on ever changing Himalayan region and geological condition in Nepal, it has been difficult to predict the effect of geology in underground structures. One of the potential causes for delaying hydropower projects in Nepal might be due to the severe stability problems encountered during the construction of tunnel.

2. The Norwegian Method of Tunnelling

The Norwegian Tunnelling Method (NMT) has evolved a successful strategy out of the 50 years of experience adopted in various rock conditions. While the New Austrian Tunnelling Method (NATM) appears most suitable for soft ground, where a smooth profile can be formed, the NMT appears most suitable for good (hard) rock masses even where jointing and high overbreaks are dominant, and where drill and blast method or hard rock TBM's are the most usual form of excavation. The tunnelling process have had many improvements and changes over the time. Over the past 50 years, significant in the process has occurred with advent wet-process shotcrete and Steel Fiber Reinforced Shotcrete (SFRS). Since steel fibers are not continuous, they do not experience corrosion like mesh and Reinforced Cement Concrete (RCC). Another revolution is the development of full-column-grouted resin (Thermo-mechanically Treated (TMT)) bolts whose cost is only a fraction of the concrete lining [1]. In NMT bolting is the dominant form of rock support since it mobilizes the strength of the surrounding rock mass in the best possible way. Potentially unstable rock masses with clay-filled joints and discontinuities would increasingly need shotcrete and SFRS [S(fr)] to supplement systematic bolting (B). In NMT, a thick load bearing ring (Reinforced Ribs of Shotcrete (RRS)) can be formed as needed, and it matches an uneven profile better than lattice girders or steel sets. NMT highlights the importance of shotcrete lining in tunnel support. Shotcrete lining of adequate thickness and quality is a long term support system. It must be ensured that there is a good bond between shotcrete and rock surface. Tensile bending stresses are not found to occur even in the irregular shotcrete lining in the roof due to a good bond between shotcrete and the rock mass in an arched-roof opening. In addition, rock bolts help in better bonding between the lining and rock mass to arrest its bending. However, bending stresses may develop in lining within the faults. Barton et al. summarized the advantages of SFRS as follows [2]:

- high application-capacity rate upto 25 m3 per hour,
- efficient reinforcement,
- lesser rebound in the range of 5-10% which is lower than that in the dry process
- uniform and high quality SFRS,

- less dust than in dry process,
- no mesh is needed and so no air gaps behind shotcrete,
- low permeability due to low water-cement ratio
- no corrosion of short-stainless steel fibers, and
- cost-effective in long tunnels or large caverns. However, technology calls for skilled workers, engineering geologists and rock engineers.

3. Q-system for rock mass classification

The Rock Quality Index (Q) was developed as a part of the Norwegian Method of Tunneling and was introduced in 1973 and has undergone several improvements and modifications after its introduction. The method was birthed with around 210 case records with introduction of shotcrete and grouted rock bolts for stabilizing the tunnels and was updated in 1993 with replacement of wire mesh shotcrete S(mr) with fiber reinforced shotcrete S(fr) based on additional 1050 new case records [3]. With the addition of further 800 case records from Norway, Switzerland and India, the support chart was further modified with removal of a support category with only 8 categories in the new chart, the minimum thickness of recommended shotcrete was greater than 4 cm and the inclusion of Reinforced Ribs of Shotcrete (RRS) for very poor rock (and below) quality [4]. The advantage of the Q-system of classification compared to other classification method as highlighted by [4] is the use of logarithmic scale, which covers rock quality within six orders of magnitude (10−3 − 103), compared to the linear rock mass quality scales of Rock Mass Rating (RMR) and Geological Strength Index (GSI), which cannot predict the non linearity and anisotrophy of rock mass. The parameters of the Q-index (Jn/Jr) also allows for predicting of possible overbreaks (Jn/Jr > 6) and thus can be used for assessing the working methods of the contractors while blasting. The best use of the method is made when a histogram is provided for every parameters instead of giving a single representative Q value. The variable nature of the rock mass can be best described and interpreted when a range of values is provided as a histogram [4].

3.1 Design of Supports

The Q-value gives a description of the rock mass stability of an underground opening in jointed rock masses. High Q-values indicates good stability and low values means poor stability. Based on 6 parameters the Q-value is calculated using Equation (1).

$$Q = RQD/Jn \; * Jr/Ja \; * Jw/SRF \qquad (1)$$

where, RQD is the Rock Quality Designation, Jn is the Joint Set Number, Jr is the Joint Roughness Number, Ja is the Joint Alteration Number, Jw is the Joint Water Reduction Factor and SRF is the Stress Reduction Factor. The Q-system can thus be considered a function of three parameters which are approximate measures of:

i. Block Size (RQD/Jn): It represents the overall structure of rock masses.
ii. Inter block shear strength (Jr/Ja): It represents the roughness and frictional characteristics of the joint walls or filling materials.
iii. Active stress (Jw/SRF): is an empirical factor describing the active stress.

Based on the obtained Q value, the rock mass is classified into eight class (A to G) from Exceptionally Good to Exceptionally Poor as shown in Figure 1 [5]. The support to be provided is also determined from the same graph. In the chart, on Y-axis the term ESR refers to the Excavation Support Ratio which signifies the factor of safety requirement. There will be an increasing need for support with increasing span and increasing wall height and the safety requirement will depend on the purpose of the excavation i.e. a road or underground powerhouse tunnel will require higher level of safety than a water tunnel. A low ESR value indicates the need of high level of safety. The values of ESR for different excavation is shown in Table 1. It is recommended to use ESR = 1.0 when $Q \leq 0.1$ for the types of excavation B, C and D. The reason for that is that the stability problems may be severe with such low Q-values, perhaps with risk for cave-in.

The ratio of span (or wall height) to ESR (Equation (2)) gives the Equivalent Dimension. The equivalent dimension refers to the opening dimension factored by the required degree of safety for construction.

$$\text{Equivalent Dimension} = (\text{Span or height in m})/ESR \qquad (2)$$

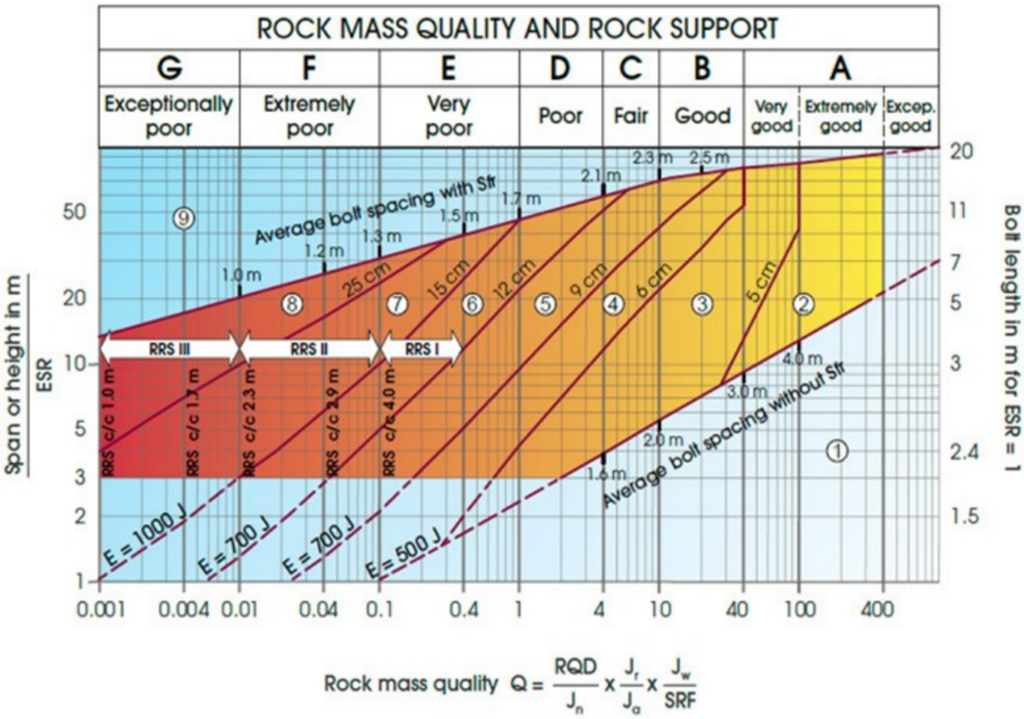

Fig 1: Permanent support recommendation based on Q value [5]

Table 1: *Values of ESR*

	Type of Excavation	ESR
A	Temporary mine openings, etc.	3 - 5
B	Vertical shafts: (i) circular opening (ii) rectangular opening	 2.5 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defense chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitate, factories, etc.	0.8
G	Very important caverns and underground openings with a long life-time, ≈ 100 years, or without access for maintenance.	0.5

4. Numerical Modeling

The use of numerical modeling technique has recently been put into practice for construction of tunnels and caverns in context of Nepalese Himalayas. Numerical modeling utilizes computing power and, using various modeling techniques, can be a precise way of solving very complex problems. Due to the dependency on empirical, analytic methods and experiences, heavy support is provided in the tunnels of Himalayas. This has a negative effect of the cost and time consumption of the project [6]. The precision of numerical modeling lies in the accuracy of the input parameters used to produce the model. So, it is very important to obtain precise data on the properties of the rock mass during preliminary investigation. With the help of modeling, we can obtain a safe and optimum support for every support class. The rock failure mechanism also plays an important role during modeling of underground spaces. Hence, it is important to identify whether the rock fails by Elastic Plastic or by Strain softening method since the failure mechanism has an effect on the residual values of the rock. Numerical modeling must be implemented from the investigation stage of the project to obtain an accurate image of the ground behavior and the support requirement. The results must also be checked against the results of empirical method to confirm the accuracy of the generated models.

For numerical modeling finite element software RS2 developed by Rocscience is used. It is used for 2-dimensional analysis and design of underground tunnels in hard rock, weak rock, jointed rock, and soft ground and other geotechnical works. Multi-stage analysis and advance support design tools simplify the design of tunnel lining system. It has, among other models, Mohr-Coulomb and Generalized Hoek-Brown failure criteria for material model [7]. Both generalized Hoek Brown failure criteria and Mohr-Coulomb failure criteria are widely used for numerical modeling of weak rock mass of tunnel.

For two-dimensional modeling, a plain-strain model has been developed with a finite boundary. The external boundary has been set to six times the diameter of the excavation and increased as per the requirement. The rock parameters have been collected from lab test and various secondary sources and has been input in the model. In case of model with fault, the fault has been aligned within the defined model boundary with the necessary parameters of the discontinuity. Overall, the general steps for modeling for both weak and faulted rock mass can be summarized as:

1. In this study full face tunnel excavation has been considered by drill and blast method. The hydropower tunnels are relatively smaller in cross section in Nepal Himalaya due to naturally available of high head for power generation.
2. A plane strain model has been developed in RS2, and that relaxes an internal pressure on the tunnel boundary from a value equal to the applied in-situ stress to zero. The final stage, with zero internal pressure is used to determine the amount of deformation prior to support installation. The deformation is determined by the empirical relation proposed by Vlachopoulos and Diederichs [7].
3. The factoring of the applied internal pressure over several stages is used to determine the pressure that yields the amount of the tunnel wall deformation at the point of support installation. The support in the tunnel is installed and activated at a distance 2 m behind the tunnel face.
4. The tunnel closure is determined by knowing the maximum displacement of tunnel, at zero internal pressure, and radius of plastic zone.

From the study it is seen that the classification system fails as the rock mass gets weaker, especially in the region of extremely to exceptionally poor rock mass. For rock mass of Q value greater than 0.1 i.e. for very poor rock mass and above, the classification system is sufficient for stabilizing and supporting the sections with minor changes in some situations. Based on the results of study, a support guideline has been prepared for the Himalayan geology considering the commonly used support elements in the country. Since most of the hydropower tunnels have a diameter of less than 10 m in the region, the results of the study is only suitable for span/ESR value less than 10.

The support recommendation graphically shown in Figure 2, has been made based on the Q support guideline (Figure 1) for span/ESR value less than 10. For good rock mass ($Q > 0.1$), the support recommended by the Q system seems sufficient even for the Himalayan rock mass and no changes have been done for those rock mass. The notable differences in the original chart and the provided support recommendation are thus summarized below:

i. Forepole/Spiling bolt (face support) has been introduced for Extremely Poor and Exceptionally poor rock mass.
ii. Forepole/Spiling bolt (face support) has been introduced for Extremely Poor and Exceptionally poor rock mass.
iii. Reinforced Ribs of Shotcrete have not been included, instead steel ribs and concrete lining are defined.
iv. Steel sets have been introduced for Extremely Poor and weaker rock mass class. The minimum spacing of steel ribs is 0.3 m c/c for exceptionally poor rock class, considering the convenience and workability for support installation. Similarly, the spacing must be adjusted between 0.3 m and 1 m for extremely poor rock mass, as per the encountered rock quality. In the graph, the spacing of steel ribs applied to the rock class to the left of where it is defined.
v. Concrete lining has been introduced as final liner for Q value less than 0.04. The provided thickness is the minimum amount for the rock class and should be increased according to the encountered rock quality. Similar to the steel ribs spacing, the thickness of the concrete lining applied to the rock class left of where it has been defined.
vi. Support category 2 has not been included in the chart since it applies to span/ESR value greater than 10.

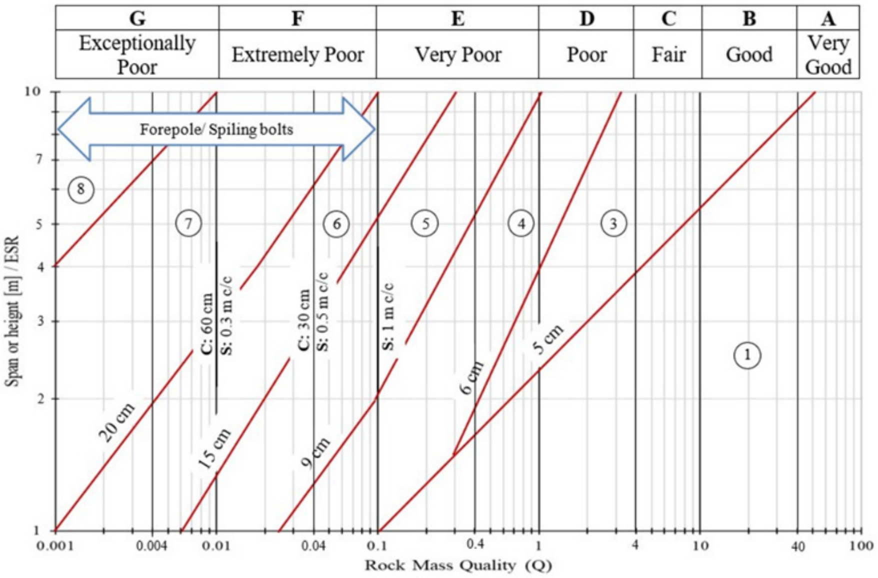

Fig 2: Support recommendation for the underground construction in Himalayan rock mass.

5. Conclusion

Due to the absence of a standard guideline of support design, tunnels in the Himalayas have different support linings for geologically similar rock mass classifi cation. This is because, the designers tend to design the supports based on their own experience with similar rock mass which results in varying designs. There is thus a need to develop a standard framework which provides designers with a reference for support linings from similar rock mass conditions. Since numerical modeling allows a close representation of the ground condition, it is of utmost importance to utilize the modeling techniques for the design and analysis of underground supports. With case histories from Nepal Himalayas, a framework has been developed (Chapter 6) for modeling the weak rock mass in two-dimensional plain-strain model. Although, 2D model has limitations compared to 3D modeling, it is widely preferred in the industry due to less time and computational requirements. 2D model has thus been used for analysis of various cases in this project. Staged model with an improved rock mass around the excavation, to represent pre-excavation support, provided valuable insight to the behavior and liner requirements. Sequence of excavation and support installation should also be considered during modeling. Similarly, faults and other discontinuities should be incorporated in the model to get a better understanding of the rock mass behavior during excavation.

6. References

[1] Tunnelling in Weak Rocks, ser. Geo-Engineering Book Series. Elsevier, 2006, vol. 5, ch. The new Austrian tunnelling method.

[2] N. Barton, E. Grimstad, G. Aas, O. A. Opsahl, A. Bakken, L. Pederson, and E. D. Johansem, “Norwegian method of tunnelling,” World Tunnelling, June and August 1992.

[3] N. Barton and E. Grimstad, “The Q-system following thirty years of development and application in tunnelng projects,” in EUROCK2004 &53rd Geomechanics Colloquim, W. Schubert, Ed. Salzburg, Austria: The Austrian Society for Geomechanics, October 2004.

[4] N. Barton, “Forty years with the Q-system – Lessons and Developments.” Hung Hom, Hong Kong: Institute of Materials, Minerals and Mining Hong Kong, September 2015.

[5] NGI, “Rock mass classification and support design,” Oslo, May 2015.

[6] S. Karki, B. Chhushyabaga, and S. S. Khadka, “An overview of design and construction practices of himalayan hydropower tunnels,” Journal of Physics: Conference Series, vol. 1608, 2020.

[7] RS2, 2D finite element analysis. Version: 9.008 User Manual., Rocscience, 2016.

Title: Long-term impact on unlined tunnels of hydropower plants due to frequent start/stop sequences

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Keywords: *Slope stability, Soil nailing, Soil nail inclination*

Abstract

The concept of unlined pressure tunnel design is well-tested and has a history of more than 100 years. The main design principle for such tunnels is to prevent hydraulic jacking, which is obtained by suitably aligning the tunnel such that the in-situ stresses are sufficient to withstand the internal water pressure, without the use of extensive rock support and lining. Minor rockfalls are accepted during operation as long as they do not develop significantly and increase the frictional loss or cause blockage in the tunnel. However, no significant research has been conducted to understand the long term effect of power plant operation on the rock mass surrounding such tunnels. This research aims to fulfill this knowledge gap through observations and field experiments in operational tunnels and by the use of numerical modelling.

In a demand driven market, the power prices can vary on an hourly basis and the power plants can experience multiple load changes per day to benefit from the variable power prices, causing frequent pressure transients in the waterway. Further, an increasing share of unregulated energy from solar and wind power in the energy system as seen in the recent years will demand more operational changes from regulated hydropower systems which are used to maintain the balance between supply and demand. Such an operation will lead to frequent pressure pulsations and cyclic loading on the rock mass around unlined tunnels, and may contribute to increased instances of block falls as a result of rock mass fatigue.

This research is based on data and experiments conducted in Norwegian hydropower plants, where more than 95% of pressure tunnels are unlined. Further, it is seen that the operational regime of power plants in Norway has changed after the power market de-regulation in 1991. This research is focused on understanding the effects of frequent pressure pulsations in the long-term stability of unlined water tunnels. This work includes observations from the inspection of four dewatered tunnels, instrumentation and monitoring of one tunnel (Fig 1), operational data of 10 hydropower plants and numerical modelling using the distinct element code 3DEC (Fig 2).

Results indicate that pressure transients can have significant influence on the pore pressure variation and joint displacement in the rock mass around unlined pressure tunnels as a result of the time-lag between the pressure transient in the tunnel and the rock mass pore pressure. It is the source of hydraulic stresses in the rock mass and is dependent on their hydro-mechanical properties.

It is seen that 200-400 start/stops and more than 1000 load changes of varying magnitudes occur every year per generating unit in Norwegian power plants, causing frequent pressure transients. It is envisaged that this trend will further increase in the future due to addition of larger share of unregulated power from wind and solar energy. This implies that rock mass fatigue in unlined pressure tunnels may occur at an accelerated rate.

The results indicate that an increased conservatism may be needed in rock support decisions in critical areas where the rock mass permeability permits significant pore pressure changes in the rock mass during pressure transient, especially for tunnels excavated in schistose rock mass, and power plants with multiple load changes within a day.

Power plant operation is seen to have a significant influence on the amount of hydraulic stress acting on the rock mass during pressure transients. The shutdown/opening duration is usually dependent on the individual operator due to lack of standard guidelines for speed of load changes. Results show that the shutdown/opening duration during load changes directly affects the time-lag between pressure in the tunnel water and in the rock mass. It is seen that shorter shutdown/opening duration i.e., faster speed, can cause significantly high hydraulic stresses on the rock mass. Thus, slowing down the load change operation can provide significant benefit in slowing down the fatigue process. Hence, it is recommended that more emphasis should be given towards keeping the speed of load changes consistently slow.

A new term called "Hydraulic impact" is proposed to quantify the hydraulic stress on the rock mass caused by pressure transients in unlined hydropower tunnels. It can also be used to define a suitable shutdown speed of the power plants in order to help slow down the fatigue process. It is recommended to instrument and monitor more tunnels in order to validate and expand the results.

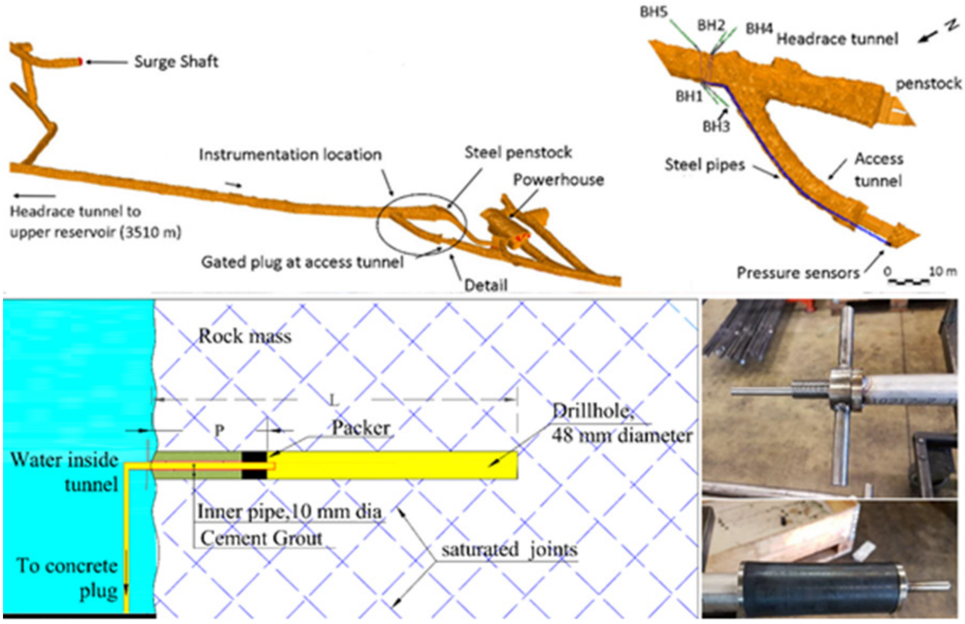

***Figure 1**: Details of the instrumentation carried out in the headrace tunnel of Roskrepp power plant*

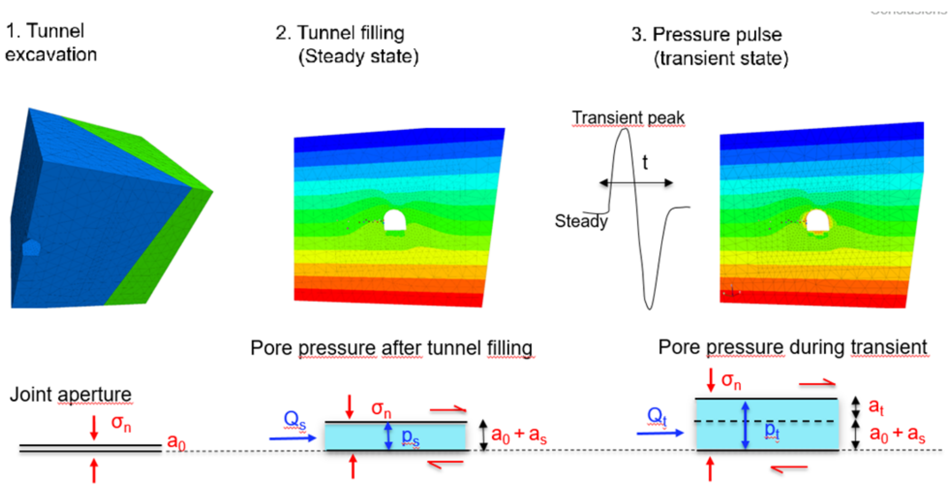

***Figure 2**: Steps of the numerical modelling performed to simulate fluid flow through rock joint during tunnel operation*

Title: Slope stability analysis using limit equilibrium: A case study of Mai cascade hydropower project

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Keywords: *Slope stability, shear strength, SLOPE/W, limit equilibrium*

Abstract

Slope stability is one of the major problems observed in the hydropower sector during construction and operation phase. It is the combined result of the steep terrain, vulnerable geological condition, inadequate geotechnical study and investigations, and improper implementation of the remedy measures in the site. In this paper authors have studied the landslide along the headrace culvert of Sanima Mai Cascade Hydropower Project and have proposed remedy measures to stabilize the landslide. The data obtained from the soil index testing, soil shear strength testing, topographical survey and geological investigation were used to develop the model to imitate the in-situ conditions. This model was analyzed using the SLOPE/W software, a limit equilibrium analysis based software. Furthermore, remedy measures to stabilize the slope were also incorporated in the model and were analyzed to see the effectiveness.

1. Introduction

The slope stability analysis in soil is an important and challenging aspect in geotechnical engineering. The Factor of Safety (FOS) obtained in slope stability analysis can be defined as the ratio of resisting force and driving force. The slope with factor of safety greater than 1 is termed as stable whereas FOS less than unity can be term as unstable.

In geotechnical engineering limit equilibrium method is widely used to analyze the slope stability. The factor of safety can also be expressed as ratio of available shear strength (resisting force) and equilibrium shear stress (driving force). The factor of safety represents the factor by which the shear strength can be divided so that the reduced shear strength can be in just equilibrium with the shear stress. The procedures used to perform such computations are known as limit equilibrium procedure. In limit equilibrium procedure, two different approaches are used to satisfy the static limit equilibrium. In the first approach, the equilibrium is considered for the entire soil mass and in the second approach the soil mass is divided into number of vertical or horizontal soil slices and the equilibrium is considered for the individual slices. The first approach is commonly known as single free body procedure and second approach is known as procedures of slices. In this paper we have adopted procedures of slices method as described as Morgenstern and Price.

2. Mai Cascade Hydropower Project

Mai Cascade hydropower plant (MCHP) is located at the Siwalik geological region, immediately south from the boundary with the Lesser Himalaya, which is separated by the Main Boundary Thrust (MBT). The predominate rock types in Siwalik are sandstone, siltstone and mudstone and its inter-bedding. The soil deposits over these bedrocks are formed from the weathering of Siwalik rock. Generally these soil deposits can be classified from Silty Sand (SM), Sandy Silt (ML). Silty Clay (CL-ML) and Lean Clay (CL).

MCHP utilizes the water from the tailrace of Mai hydropower plant (MHP). The water from the tailrace of the MHP is conveyed using the headrace culvert and headrace canal to generate the electricity at MCHP. MCHP has installed capacity of 7 MW. MCHP has been generating electricity from February 12, 2016. The landslide of our study falls in between chainage 0+300 to 0+350 meters.

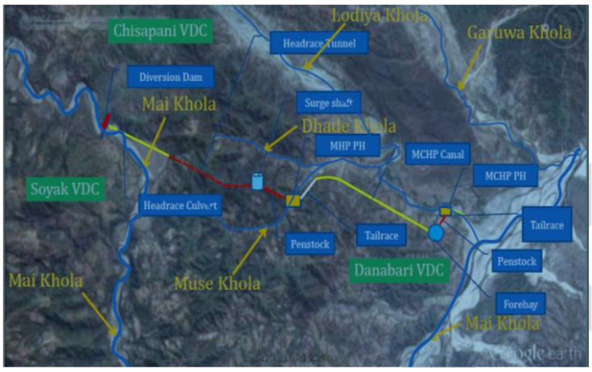

Figure 1: *Schematic layout of MHP and MCHP*

Figure 2: *Landslide zone with distorted surface drain canal*

3. Stability Assessment

As a part of annual technical inspection for civil works in MHP and MCHP, Sanima Hydro and Engineering Pvt. Ltd. (SHEPL) has been conducting technical annual audit since commencement of the Project. The team from SHEPL carries out the thorough walk over inspection of all the civil work structures. The landslide in the chainage 0+300 to 0+350 has been recorded from the technical audit of 2017.

The headrace culvert passes through this landslide zone. The structural integrity is very important for the continuous power generation. The soil in this region behaves very stiff when in dry condition but when it is wetted (fully saturated) it loses all its stiffness. Different preventive measures such as surface run off management, bioengineering and surface drain canal were used to prevent the landslide. This landslide is not active during the dry season but with the monsoon rain the soil in this region gets fully saturated and soil movements are observed. With these soil movements all the preventive measure applied like bioengineering are washed away and surface drain canals are distorted.

3.1 Soil Laboratory Testing

In order to further investigate, analyze and propose the remediation measures detailed investigation was planned after the technical audit 2022. As per the investigation plan, a detailed topographic survey of the landslide zone was carried out and two disturbed soil samples were collected for the soil laboratory testing. As a part of laboratory testing following laboratory test were conducted in both soil samples:

- Grain Size Analysis Test (IS 2720 Part – 4: 1985)
- Hydrometer Analysis (IS 2720 Part – 4: 1985)
- Atterberg Limit Test (IS 2720 Part – 5: 1985)
- Specific Gravity of Soil (IS 2720 Part III/Sec – 1: 1980)
- Direct Shear Test (IS 2720 Part – 13:1986)

Above mentioned laboratory testing were carried out to find out the pertinent soil properties and soil shear strength. These soil properties were used to define the material properties to imitate the in-situ condition in the numerical modeling in SLOPE/W.

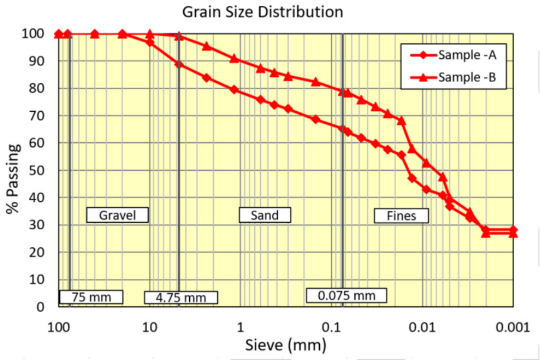

Figure 3: *Sieve and hydrometer analysis results*

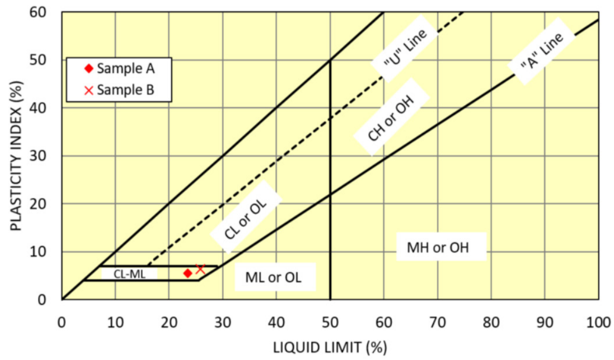

Figure 4: *Plasticity Chart of the soil samples*

Table 1: *Summary of Soil Laboratory Test*

Sample ID	USCS Soil Classification	Specific Gravity	Liquid Limit	Plasticity Index	% Passing No 200 Sieve	% Sand	% Gravel	ϕ’	c’
Sample A	Silty CLAY with Sand (CL-ML), moist, gray, contains gravel	2.64	23.46	5.51	65.16	23.52	11.3	23	2
Sample B	Silty CLAY with Sand (CL-ML), moist, reddish brown	2.584	25.88	6.33	78.84	20.22	0.94	10	36

3.2. Slope Stability Analysis using SLOPE/W

The slope stability issues on the CH 0+300 to 0+350 was analyzed using SLOPE/W software of GeoStudio suite. SLOPE/W does the stability analysis based on the limit equilibrium. Generally for the stability analysis in SLOPE/W there are five components:

- Geometry – Description of the stratigraphy (subsurface strata) and shapes of the potential slip surface.
- Soil strength – Parameters used to describe the soil shear strength
- Pore-water pressure – Means of defining the pore-water pressure conditions to simulate the site condition.
- Reinforcement or soil-structure interaction – soil nails, anchors, retaining structures and so on.
- Imposed loading – Surcharge or dynamic (earthquake) loading.

The geometry for the stability analysis was prepared based on the recent survey conducted on the landslide zone. As no subsurface borings were carried out during this study, subsurface profile for the analysis was prepared based on the communication with the engineering team that supervised those site during construction, as-built drawings and engineering judgment. The subsurface was mainly divided into two region, lower strata (strong strata) below the foundation of the culvert and upper strata which has been creeping. The features like gabion wall at the toe of the hill and culvert was also incorporated in the model to simulate the in-situ condition.

The soils shear strength for the upper strata was adopted as per the direct shear test conducted on the two disturbed samples whereas the shear strength of the lower strata was based on the engineering judgement and experience in those geological region.

The pore-water pressure was simulated by drawing the phreatic line. As per the observation and communication with operation management team, the mass creeping in this region starts as the soil gets saturated. So, to mimic this scenario phreatic line was drawn along the surface to model fully saturated condition and elevated pore-water pressure.

***Table 2**: Summary of soil properties used in SLOPE/W*

Name	Unit Weight (kN/m^3)	Eff Cohesion, c' (kPa)	Eff. Friction Angle (ϕ')
Compacted Fill	18	3	27
Gabion Wall	22	0	45
Concrete Wall		115000	
Natural Ground (Upper Strata)	18	2	23
Natural Ground (Lower Strata)	18	10	30

The soil sloughing has been taking place as the soil saturates with the monsoon rainfall. The soil sloughing depth is generally 1 to 1.5 meters in depth, which is shallow. No any sign of deep seated failure has been observed in the site. The gabion walls on the toe of the hill were observed and no any sign of bulging and movement were seen in the gabion. The main objective of this SLOPE/W analysis was to analyze the slope for deep seated failure condition and evaluate the factor of safety. So, two stability analysis was ran, one with the slip surface at shallow depth and another with slip surface at deeper depth.

Morgenstern and Price (1965) method was used to evaluate the factor of safety for the slope. The output of numerical modeling for the two cases are shown in the Figure 3-3 and Figure 3-4. The calculated factor of safety for the stability model:

- For failure plane at shallow depth is 1.0
- For failure plane at deeper depth (deep seated failure) is 1.2

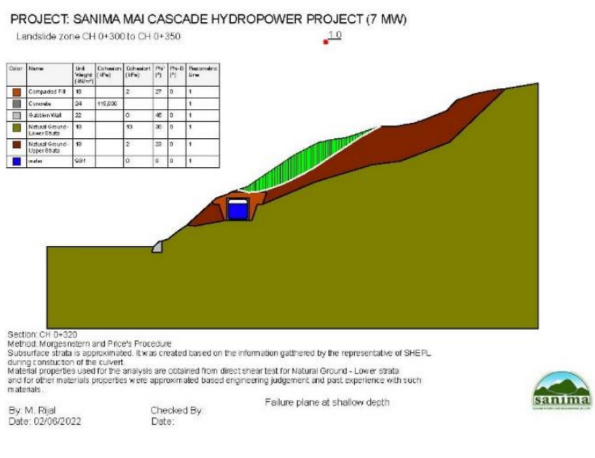

Figure 5: Stability Analysis when slip surface passes through shallow depth

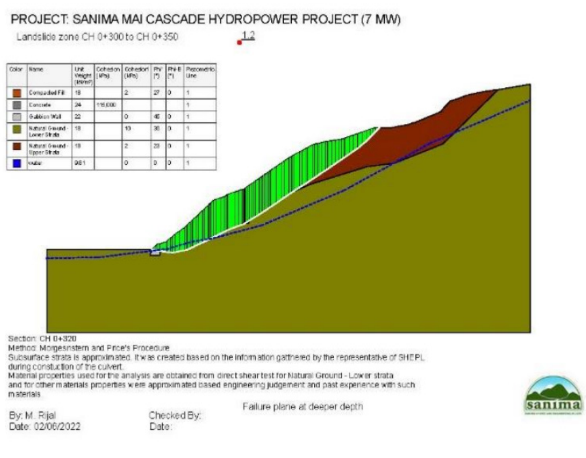

Figure 6: Stability Analysis for deep seated failure

4. Conclusion

The stability assessment signifies the slope is still vulnerable for the shallow seated failure (Soil Sloughing). This failure has been observed in the site after every monsoon. Shallow seated failure is not as critical as deep seated failure as it does not compromise the stability of the culvert directly. But, if this failure propagates further and expose the good foundation soil strata to the monsoon water it will aid and help to propagate deep seated failure. So, this study recommends the requirement of shallow failure. Following measured are proposed to control the soil sloughing along the slope:

- It is highly recommended that the existing slope should be flattened not stepper than 2.5H:1V.

- Two to three intermediate retaining structures with drain canal at the toe of the retaining structures is used to further stabilize the slope, manage surface water runoff and check the soil sloughing.
- Provision of horizontal subsurface drain can be made to catch the subsurface water and check internal erosion. The subsurface drain will also help to lower the phreatic line and hence improve the stability.
- The concept of head-off should also be considered. This is improve the stability of the slope by reducing the driving force.

Factor of safety for deep seated failure is 1.2. It signifies that the slope is stable from the point of view of deep seated failure. This failure case is also a critical one as this compromise the stability of the culvert running along the slope. As the soil strength parameters for the natural ground has been approximated being on the conservative side this result can also be taken as conservative. And authors are of the opinion that the slope is stable from the point of view of deep seated failure.

5. Limitation

The assumptions were made while setting up the slope stability model in GeoStudio. All the soil strength parameters required were not obtained from the lab test. Engineering judgment and experience were used to approximate the soil strength parameters for the foundation soil. As no ground water level information was available, the pezometric line required in the model was also approximated to represent the field conditions. Due to above assumption made in the model, it may not represent the actual in-situ field condition. However this analysis can be used for the preliminary discussion and to plan the future subsurface investigation. As shallow seated soil sloughing has been observed every year and the model also predicted the same, the authors are of the opinion that the model is representing the complex in-situ condition.

6. References

Duncan, M.J., Wright, S.J. & Brandon, L.T (2014). Soil Strength and Slope Stability, 2 nd Edition, John Wiley and Son Inc.

Terzaghi, K., Peck, B.R., & Mesri, G. (1996). Soil Mechanics Engineering Practice, 3rd Edition, John Wiley and Son Inc.

Dhital, M.R. (2015). Geology of the Nepal Himalaya, U.K. Springer.

Naval Facilities Engineering Command (1986). Soil Mechanics Design Manual 7.01.

GEO-SLOPE International Limited (2012). Stability Modeling with SLOPE/W.

Sanima Hydro and Engineering Pvt. Ltd (2017). Mai Hydropower Project (22MW) and Mai Cascade Hydropower Project (7MW) – Audit Summary Report #1.

Sanima Hydro and Engineering Pvt. Ltd (2018). Mai Hydropower Project (22MW) and Mai Cascade Hydropower Project (7MW) –Technical Audit Summary Report #1 (For Civil Works/Components).

Sanima Hydro and Engineering Pvt. Ltd (2021). Mai Hydropower Project (22MW) and Mai Cascade Hydropower Project (7MW) – Audit Summary Report Level#2 (For Civil Structures Works).

Sanima Hydro and Engineering Pvt. Ltd (2022). Mai Hydropower Project (22MW) and Mai Cascade Hydropower Project (7MW) – Audit Summary Report Level#2 (For Civil Structures Works).

Title: Effect of Rock Pillar Width on the Stability of Rock Mass Around Large Cavern in the Himalayas

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Keywords: *2D FEM, Parallel Caverns, Numerical Analysis*

Abstract

The utilization of underground space for infrastructure development is a well-established practice in the hydropower sector in Nepal. Construction of large underground caverns in the fragile geology of Nepal often faces many problems. In recent times, large underground excavations are common in the hydropower sector for various components like settling basins, surge shafts and powerhouses. In this study, the effect of excavation of two large parallel caverns housing the sedimentation basins of a hydropower project have been studied.

The parallel settling basin caverns of Super Madi Hydroelectric Project have been selected for analysis. This project lies just above the Main Central Thrust in the southern parts of the Higher Himalayas. The rock mass has been characterized using Geological Strength Index and consequently the rock mass parameters were calculated. The rock mass is modelled to follow the Generalized Hoek Brown failure criterion. The analysis of the cavern was carried out using 2D FEM program RS2. In this paper, we have discussed the effect of geometry of excavation on the rock mass. Sharp corners in edges lead to accumulation of stresses. This is mitigated by improving the geometry by rounding off the corners. Additionally, support systems for the caverns have also been evaluated.

1. Introduction

The country, Nepal, has also recently begun adapting tunnels for development of transportation infrastructure where parallelly running tunnels are common. This paper explores the effect of excavation of two large parallel caverns housing the sedimentation basins of a hydropower project. Super Madi Hydro-Electric Project lies in the southern part of the Higher Himalayan Zone, just above the active main central thrust (MCT) . The settling basin caverns, which are the focus of our study, are located inside the massive rock mass, which have slightly deformed foliated micaceous and banded gneiss with a thin layer of schist. Such configurations of parallel excavations are not new in Nepal. Similar studies on twin excavations have been conducted by various authors such as Usmani et. al. 2014 [1] and Imran Ahmad Khan et. al 2021 [2].

2. Methodology

2-dimensional finite element method (FEM) has been used to perform the analysis in this paper. RS2 software developed by Rocscience has been used. The rock mass is considered as a continuum and characterized it according to the Geological Strength Index(GSI). The rock mass has been modeled for the Generalized Hoek Brown failure criterion. The post failure behavior of the rock mass is defined according to Khadka[3]. For extremely weak rocks with GSI values less than 30, the elastic-plastic model is used where no residual parameters are required. For weak to good quality rocks, strain softening model is used where the residual parameter is obtained by reducing the peak GSI between 60 and 70% for very poor to poor rock (30<GSI<50) while it is reduced between 40 and 50% for fair and good rocks (50<GSI<65). [4]

During construction, there always exists a time gap between excavation and support installation during which the rock mass relaxes and deforms. The relaxation has to be incorporated in the numerical model. For simulating this behavior, the approach taken by Usmani et al, 2014 has been adopted. We replace the bench material which is to be excavated with a material whose deformation modulus is 30% of the actual rock mass' and initial loading is set "none". Supports are then installed in the following stage.

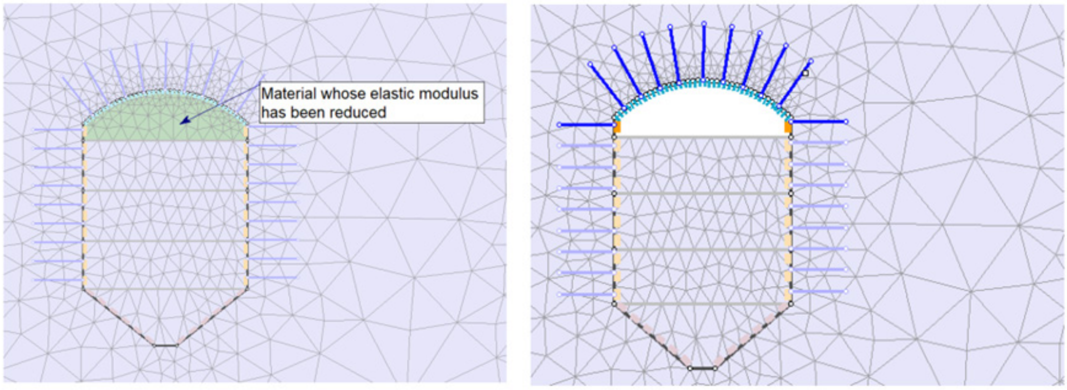

Fig. 1. Modeling process where core material is replaced with one whose elastic modulus is reduced (left) and support installation in the following stage (right)

3. Conclusion

The original geometry of the caverns leads to high accumulation of stress at the corners which results in having to install thick linings. This can be mitigated by rounding off the corners. The analysis done as part of this study neglects the effect of joints and water. The model can be calibrated to even higher degree of accuracy if comparisons to in-site measurements could have been done.

4. References

[1] Usmani, A., Nanda, A., Mandal, A., & Jain, S. K. (2015). Interaction Mechanism between Two Large Rock Caverns. International Journal of Geomechanics, 15(1), 06014014. https://doi.org/10.1061/(ASCE)GM.1943-5622.0000386

[2] Imran Ahmad Khan, K. Venkatesh and R.K. Srivastava, Elasto-plastic finite element analysis of twin tunnels – A comparison of excavation in intact and jointed rock mass, Materials Today: Proceedings, https://doi.org/10.1016/j.matpr.2021.05.122

[3] Khakda S 2019, January Tunnel Closure Anlaysis of Hydropower Tunnels in Lesser Himalayan Region of Nepal through Case Studies. Doctoral Thesis, Kathmandu University Retrieved from Rock Mechanics and Underground Structure Lab: http://rmlab.ku.edu.np/research/academic-project/

[4] Shyam Sundar Khadka et al 2019 J. Phys.: Conf. Ser. 1266 012015

Title: The influence of soil nail inclination on slope stability - A review paper

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Keywords: *Slope stability, soil nailing, inclination*

Abstract

Due to the rapid construction of highways and major road expansion in the mountainous region, slope stability has become a challenging issue in Nepal. In addition, owing to the rugged mountain topography, steep terrain, fragile geological conditions resulting from tectonic movement, and intensive rainfall during the monsoon season, serious landslides and soil erosion are recorded every year. These represent major constraints on development, causing high levels of economic losses and claiming many human lives, demonstrating the necessity to analyze the stability of these soil slopes.

Soil nailing is one of the most extensively used methods for slope stabilization. It is considered to be a major preventive and economic method to overcome slope failure due to its ease of construction, technical suitability, and low maintenance cost. It is an emerging soil stabilization technique that involves strengthening the soil mass using reinforcing bars called soil nails into natural or excavated slopes to prevent its ground movement. These soil nails are passive bars that withstand tensile forces, shearing forces, and bending moments to retain soil mass from sliding. Due to ground deformation, soil nails generate reinforcing action through soil nail interaction, which results in an increase in tensile pressures in the soil nail. The development of axial force, which is basically a tension force, accounts for the majority of resistances. Consequently, Shear and Bending are considered to provide limited resistance.

This paper presents a comprehensive review of the soil nailing system in terms of inclination to determine the optimum angle for the effective stabilization of soil slope. The aim of this paper is to provide an overview of the research work which has been carried out through numerical modeling, experimental studies, and laboratory tests for different orientations of soil nails to determine the maximum value of factor of safety (FOS). Based on the observations, slope stability is analyzed using the limit equilibrium method, finite element method, and finite discrete element method. The review results showed that the soil nail inclination has a significant effect on the stability of the soil slope. For slopes with specific soil parameters such as cohesion, angle of internal friction, and unit weight of soil, the optimum nail angle lies within a certain range, however, it also relates to the angle of the slope.

SURUng-I
First International Conference on
Space Utilization and Research in Underground Structures
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Title: Review Paper on Sesmic Response Analysis of Underground Structures

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Keywords: *Seismic, tunnel, caverns*

Abstract

Earthquake can cause a huge amount of loss in terms of economy and human life. To achieve safety against natural disasters like Earthquake, it is necessary to construct the underground caverns considering geotechnical problems like site effects. Evaluating the role in local site conditions on ground shaking is essential part in earthquake ground motion-prediction.

Seismic ground response analysis is carried out to estimate the stratified soil response (in term of acceleration, PGA profile, stress and strain histories, and response spectrum) subjected to a considered bed rock motion. The commonly applied analysis methods are: Linear, Equivalent linear and Non-linear. Linear analysis methodology is based on the assumption that shear modulus and damping is strain independent and constant for each layer. The equivalent linear method of ground response analysis was developed to analyze the nonlinear response of soil using frequency domain analysis with the aid of linear transfer functions. In nonlinear method, the intricate but realistic stress strain behavior of soil is modelled for accurate measurement of soil behavior using direct numerical integration in the time domain.

Owen and Scholl (1981) correlated seismically-induced tunnel damage with surface peak ground acceleration using data from 127 case histories and concluded that slight damage occurred in rock tunnels for Peak Ground Accelerations (PGA) below 0.4 g. According to Hashash et al. (2001), the evaluation of underground structure seismic response requires an understanding of the anticipated ground shaking as well as an evaluation of the response of the ground and the structure to such shaking. Zhang et al. (2016) examined the damage of the Tawarayama tunnel caused by the 2016 Kumamoto Earthquake, reporting that ring cracks were found on the tunnel with a spacing of 10 m in around 20% of the spans of the tunnel. This observation was attributed to the interaction between the seismic wave propagation and the geological conditions at site. Recently, Callisto and Ricci (2019) carried out a back analysis of the damage suffered by the San Benedetto tunnel during the 2016 Norcia earthquake in Italy, aiming at evaluating the ability of available methods of analysis to predict its seismic performance. Simplified methods, where the seismic loading is introduced in an equivalent static manner, were found to provide reasonable predictions, while more accurate responses were provided by dynamic analyses of the case study.

This study will utilize the equivalent-linear (EQL) and non-linear (NL) methods of analyses to perform ground response analysis. Ground response analysis is required to carry out for predicting ground surface motion and to evaluate properties of soil during earthquake. It has been a great challenge to design underground structures in highly seismic active zone with a loose soil like Kathmandu. Most of the underground structures in these places are designed with Q-system classification system which do not consider seismic effect. This study aims to develop specific guideline considering seismic forces for underground structures in these places

1. Introduction

Comparing the entire natural disasters, earthquake is the worst. The earthquake in Nepal is result of movement of tectonic plate in the north-northeast direction. The Indian plate is moving north at around 45mm a year and pushing under the Eurasian plate. Over time that is how the Himalayas are created. It is impossible to stop the occurrence of earthquake however it's disastrous consequences can be controlled by understanding its nature, causes, frequency, magnitude and areas of influence. On reference to the chronological study of earthquake, Nepal lies in one the most seismologically active zone in world. A narrow belt along the topographic front of higher Himalaya that runs from west is east is seen more active. Nepal has recently faced a huge Gorkha earthquake 2015 with a magnitude M 7.8.

Damage of many structures was seen in that earthquake. It's a very big challenge in Nepal to build civil engineering structures against such strong seismic waves. Nepal is a land locked country having steep topography but rich in water resources. For the sustainable development of Nepal, short and efficient roads in mountainous region, inter-basin irrigation and mega hydro projects are inevitable. These infrastructures are only effective and economical by the construction the underground tunnels and cavern.

In 1917, the first ever 500 m long road tunnel was built by army engineer Dilli Jung Thapa with local resources available at that time in Churia range of the of the Himalaya. It was supposed to reduce the time taken by horse-drawn carriages and lorries to travel from the Indian border to Bhimphedi. Some parts of the tunnel collapsed during the Gorkha earthquake 2015, but the tunnel entrance is still intact.

Nepal extends from east to west for about 890 kilometers and has a width ranging from 150 to 250 kilometers towards north-south direction. Within this very short width, the altitude of the country varies greatly from about 100 meters above sea level at its southern border to its maximum up to 8,848 meters above sea level (the Mount Everest) at its north giving very rough terrain and steep mountainous topography. From north to south, Nepal has four distinct geographical / geological regions such as higher Himalaya, the mid-hills (lesser Himalaya), the Siwaliks (Churia) and the Gangetic plane (Tarai). The Siwaliks, mid-hills and higher Himalayas covers almost 83 percent of its land representing Nepal as mountainous country. Because of large elevation difference in very short distance, the climatic condition of the country varies greatly.

Many stability problems have been seen as big challenge during construction and after operation of the tunnel. The impact of earthquakes on tunnels can be severe due to ground failures such as liquefaction, strong ground shaking, and fault crossing. [1]

Stability problems during construction have been already studied in Nepal, however stability during such seismic waves need to be studied during operation of tunnel. Experience shows that underground structures, especially deep ones, are far less vulnerable to earthquakes than superficial ones. The latter are endangered by earthquakes due to the fact that the motion of the ground can be amplified by the response of the structure to such an extent that the induced strains damage the structure. The earthquake waves can also be amplified within soft superficial strata. In addition, saturated soil may lose its shear strength (liquefaction), and this can lead to landslides or failure of foundations and retaining walls. In contrast, deep buried structures, especially flexible ones, are not expected to oscillate independently of the surrounding ground, i.e., amplification of the ground motion can be excluded. This is manifested by the relatively low earthquake damage of tunnels. Of course, the portals may be damaged by earthquake-induced landslides [2].

Study of tunnels in areas affected by strong earthquakes in the last decades revealed that at least three cases of tunnels damaged by earthquake shaking or offset by seismic faulting, including the Bolu (Turkey) twin tunnels, which collapsed during the 1999 Düzce earthquake. During the 1999 Chi-Chi earthquake in Taiwan, a large number of mountain tunnels suffered significant damage. In particular, 26% of the 50 tunnels located within 25 km of the earthquake fault were severely damaged, while over 20% of the tunnels were moderately damaged. [3]

Various types of damage were observed, including: lining cracks, portal failures, spalling of the concrete lining, groundwater inrush, exposed and buckled reinforcement, displaced lining, rockfalls in unlined sections, lining collapses caused by slope failures, pavement cracks and lining shear-off. Severe damage was observed close to surface slopes or portal openings, while deeper buried sections behaved generally better.

These data indicate that tunnels cannot be considered as structures invulnerable to earthquakes. Furthermore, the tectonic offset of tunnels shows that certain observed seismic surface ruptures are not necessarily indicative of tectonic faulting and represent only secondary local ground instability effects.

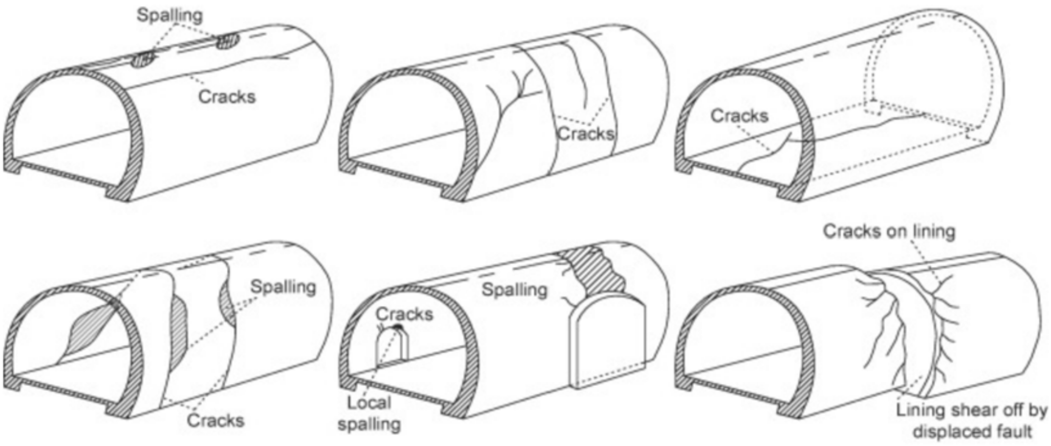

Fig 1: *Various types of damage in tunnel due to earthquake [4]*

The Main differences of the seismic response of underground structures from those of the surface structures are the following:

- The seismic effect is controlled by the deformation imposed on the structure by the ground, not by the forces or stresses.
- The inertia of the surrounding soil is much larger relative to the inertia of the structure for most underground facilities.
- Presently, the existing road (without tunnel) has to go up and down the Nagdhunga Pass, thus the alignment has various problems as follows: Continuous sharp curves and hairpin curves: 19 locations.
- Steep gradient: aggregate 1.6 km section exceed a vertical gradient of 7%.

Due to sub-standard alignment, vehicles can travel only with less than 20km/hour and traffic congestion is experienced daily. There are also dangerous/unstable slopes beside the road, thus risks of road closure due to slope failure is high. Solution to avoid above problems is to construct a tunnel for a safe, smooth, and less costly travel.

2. OBJECTIVE:

I. Primary Objective:

- To develop specific guideline, considering seismic forces, for tunnel structures located in Himalayan Region.
- To study PGA in case of underground caverns and develop the response spectrum curve.

II. Secondary Objective:

- To study the seismic response of Himalayan Underground structures based on real ground motion and National codes.
- To highlight the necessity of ground response analysis of underground structures in Himalayan region.

3. NEED OF STUDY

- Analysis of underground structures under seismic loading is not available for Himalayan region in Nepal.
- No specific design guideline for seismic consideration for tunnel structures located in Himalayan region.
- Study of impact of earthquake on underground structures w.r.t. liquefaction, strong ground shaking and fault crossing have not been carried out for Himalayan region in Nepal.

4. METHODOLOGY:

Seismic ground response analysis is carried out to estimate the stratified soil response (in term of acceleration, PGA profile, stress and strain histories, and response spectrum) subjected to a considered bed rock motion. The commonly applied analysis methods are: Linear, Equivalent linear and Non-linear (Kramer, 1996). Linear analysis methodology is based on the assumption that shear modulus and damping is strain independent and constant for each layer. The equivalent linear method of ground response analysis was developed to analyse the non-linear response of soil using frequency domain analysis with the aid of linear transfer functions. Equivalent linear method is an approximation method in which the non-linear behaviour of the soil (i.e., shear modulus and damping is strain dependent) is modelled in terms of equivalent linear properties (secant shear modulus and damping which is strain-independent for a range of strain) corresponding to effective shear strain using iterative procedure. The iterative procedure is governed by the target of finding a compatible shear modulus and damping for a particular effective shear strain. Generally, the effective shear strain is considered to be 65% of the maximum shear strain developed in the layer [5]. Though, this method is computationally convenient and provides reasonable results, it is incapable to represent the change in soil stiffness that actually occurs [5]. In nonlinear method, the intricate but realistic stress-strain behaviour of soil is modelled for accurate measurement of soil behaviour using direct numerical integration in the time domain. The present study utilizes the equivalent-linear (EQL) and non-linear (NL) methods of analyses through DEEPSOIL, commercially available software, to perform ground response analyses. The computer program DEEPSOIL has been developed to perform one dimensional (1-D) ground response analyses [6]. DEEPSOIL facilitates linear and EQL analyses in the frequency domain; and linear and non-linear analyses in the time domain. Equivalent linear method in the frequency domain and nonlinear in the time domain has been chosen in this study to observe the soil response under earthquake loading and make a comparative study of the outcome

One-dimensional (1D) simulation of shear waves propagating vertically through soil layers, also known as ground response analysis (GRA). Ground response analysis is required to carry out for predicting ground surface motion and to evaluate properties of soil during earthquake. Our research utilizes equivalent linear and non-linear method through computer software DEEPSOIL. The computer program DEEPSOIL has been developed to perform 1D ground response analysis. Few passed earthquake motions will be used for further results. Then comparison will be done with the known PGA with seismic zoning factor (PGA of mentioned site from NBC 105:2020). After comparing it with NBC, modeling of the various tunnel structures will be done with ETABS and RS2. Then time history analysis of all those tunnels will be done with the data obtained from NBC and with the data obtained from DEEPSOIL separately.

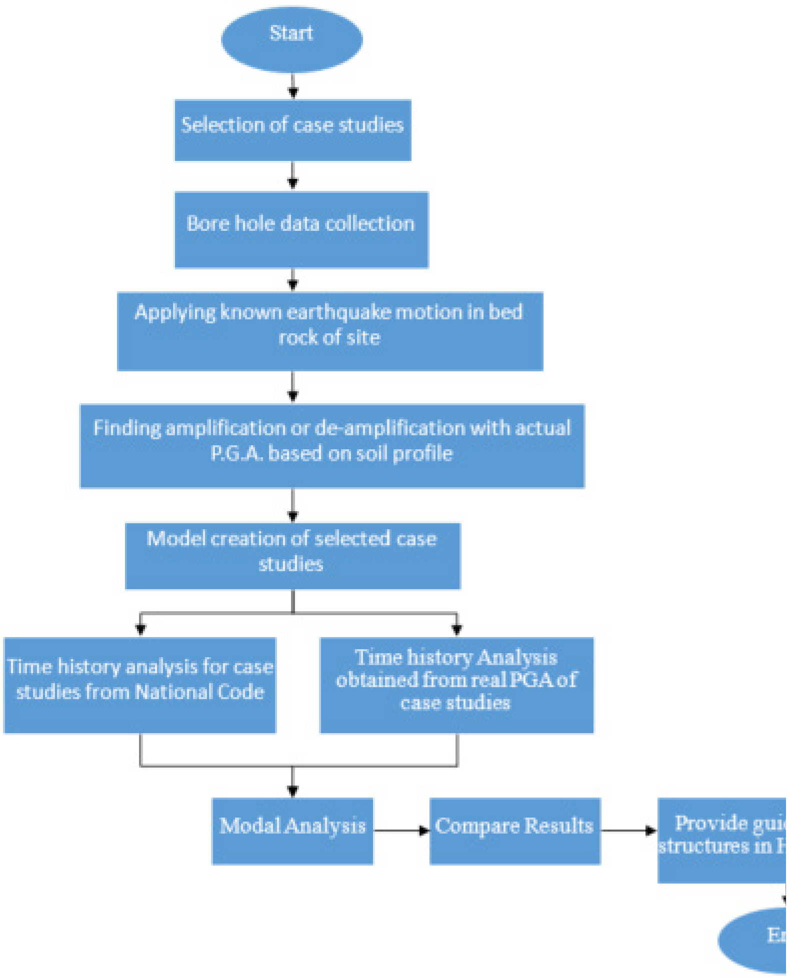

***Fig 2**: Flowchart of Methodology*

5. LITERATURE REVIEW:

Several studies have documented earthquake damage to underground facilities.

According to [7] the evaluation of underground structure seismic response requires an understanding of the anticipated ground shaking as well as an evaluation of the response of the ground and the structure to such shaking. Evaluation of the seismic response and subsequent design of buried structures can be summarized in three major steps:

1. Definition of the seismic environment and development of the seismic parameters for analysis.
2. Evaluation of the ground response to shaking, which includes ground failure and ground deformations.
3. Assessment of the structural behavior due to seismic shaking including;
    a. development of seismic design loading criteria,
    b. underground structure response to ground deformations, and
    c. special seismic design issues.

For most underground structures, the inertia of the surrounding soil is large relative to the inertia of the structure. Measurements made by [8] of the 5 seismic response of an immersed tube tunnel during several earthquakes show that the response of a tunnel is dominated by the surrounding ground response and not the inertial properties of the tunnel structure itself. Therefore, the main point of underground seismic design is on the free-field deformation of the ground and its interaction with the structure. The emphasis on displacement is totally in contrast to the seismic design of surface structures, in which the focus is on inertial effects of the structure itself. This difference requires development of the alternative design methods in which the seismically induced deformations of the ground is the controlling factor. Historically, there exist simplified approaches for evaluating the response of a buried structure:

- Dynamic earth pressure approach
- Free field deformation approaches the dynamic earth pressure method have been suggested for the underground box structures and used widely for not only underground structures but also for the surface structures such as the retaining walls. This method supplies designer a good estimate for the loading mechanism if the structure is situated at relatively shallow depths and having a rectangular cross section. For a buried rectangular structural frame, the ground and the structure would move together, making it unlikely that a yielding active wedge could form. Therefore, its applicability in the seismic design of underground structures has been the subject of controversy [9]. In the free field deformation approach, the ground is subjected to seismic wave propagation without existence of the structure. Hence, this approach ignores the existence of the structure and the cavity. The estimated deformations occurring at the ground is applied to the structure and the response of the structure is calculated. [10] and [11] suggest a simplified approach which is based on the theory of wave propagation in homogeneous, isotropic, elastic media. The ground strains are calculated by assuming a harmonic wave of any wave type propagating at an angle (angle of incidence) with respect to the axis of a planned structure. They represent free-field ground deformations along a tunnel axis due to a harmonic wave propagating at a given angle of incidence (Figure 3). Because of the uncertainty involved in the angle of incidence for the predominant seismic waves, a conservative path is followed by using the most critical angle of incidence yielding the maximum strain.

According to [7] the evaluation of underground structure seismic response requires an understanding of the anticipated ground shaking as well as an evaluation of the response of the ground and the structure to such shaking. ;[12] correlated seismically-induced tunnel damage with surface peak ground acceleration using data from 70 case histories and employing relevant attenuation relationships.

Extending the above database to 127 cases, [13] concluded that slight damage occurred in rock tunnels for Peak Ground Accelerations (PGA) below 0.4 g.

[14] attempted to develop correlations between observed damage and salient parameters affecting seismic behavior, namely lining geometrical properties, geotechnical conditions and earthquake characteristics. They concluded that deeper tunnels or rock tunnels were generally safer, while damage was more extensive with increasing earthquake magnitude and decreasing epicentral distance.

[15,16] reported the following parameters as the most critical affecting the response of mountain tunnels: earthquake magnitude, depth and epicentral distance of the seismic source, geometrical properties of the lining, burial depth and sudden changes of tunnel dimensions.

[17] proposed a damage classification based on 254 damage reports from the Chi-Chi earthquake, the 2004 Mid Niigata Prefecture earthquake and the 2008 Wenchuan earthquake, providing a short damage description for each proposed damage level. [18] examined the damage of the Tawarayama tunnel caused by the 2016 Kumamoto Earthquake, reporting that ring cracks were found on the tunnel with a spacing of 10 m in around 20% of the spans of the tunnel. This observation was attributed to the interaction between the seismic wave propagation and the geological conditions at site. Recently, [19] carried out a back analysis of the damage suffered by the San Benedetto tunnel during the 2016 Norcia earthquake in Italy, aiming at evaluating the ability of available methods of analysis to predict its seismic performance. Simplified methods, where the seismic loading is introduced in an equivalent static manner, were found to provide reasonable

predictions, while more accurate responses were provided by dynamic analyses of the case study.

6. REFERENCES

[1] A.Jaramillo, C. (2017). Impact of seismic design on tunnesls in rock-case histories. Impact of seismic design on tunnesls in rock-case histories.

[2] Kolymbas, D. (2005). Tunnelling and Tunnel Mechanics.

[3] Kazuhide YASHIRO, Yoshiyuki KOJIMA, & Mitsuru SHIMIZU. (2007). Historical Earthquake Damage to Tunnels in Japan and Case Studies of Railway Tunnels in the 2004 Niigataken-Chuetsu Earthquake.

[4] Nidhi Gautam, Rishi Bhalavada, & Dr. J.K.Sharma. (2018). Damage assessment of tunnels during earthquake.

[5] Kramer. (1996). "Geotechnical Earthquake Engineering". Prentice Hall, Upper Saddle River, New Jersey.

[6] Hashash Y.M.A, Groholski D.R., Phillips C.A., Park D., & Musgrove M. (2011). DEEPSOIL version 4.0, Tutorial and user Manual.

[7] Hashash, Y.M., et al., Seismic design and analysis of underground structures. Tunnelling and underground space technology, 2001.

[8] Okamoto, S., Tamura, C., Kato, K., Hamada, M., 1973. Behaviors of submerged tunnels during earthquakes. Proceedings of the Fifth World Conference on Earthquake Engineering, vol. 1. Rome, Italy.

[9] Wang, J.-N., 1993. Seismic Design of Tunnels: A State-of-the-Art Approach, Monograph, monograph 7. Parsons, Brinckerhoff, Quade and Douglas Inc, New York.

[10] Newmark, N.M., 1968. Problems in wave propagation in soil and rock. Proceedings of the International Symposium on Wave Propagation and Dynamic Properties of Earth Materials.

[11] Kuesel, T.R., 1969. Earthquake Design Criteria for Subways. J. Struct. Div., ASCE ST6, 1213 1231.

[12] Dowding, C.H., Rozen, A., 1978. Damage to rock tunnels from earthquake shaking. J. Geotech. Eng. Div., ASCE 104 ŽGT2.,175 191.

[13] Owen, G.N., Scholl, R.E., 1981. Earthquake engineering of largeunderground structures. Report no. FHWA RD-80 195. Federal Highway Administration and National Science Foundation.

[14] Sharma, S., Judd, W.R., 1991. Underground opening damage from earthquakes. Eng. Geol. 30, 263 276.

[15] Chen, J., Luzhen, J., Jie, L., Xiaojun, S., 2012a. Numerical simulation of shaking table test on utility tunnel under non-uniform earthquake excitation. Tunn. Undergr. Space Technol. 30, 205–216.

[16] Chen, Z., Shi, C., Li, T., Yuan, Y., 2012b. Damage characteristics and influence factors of mountain tunnels under strong earthquakes. Nat. Hazards 61 (2), 387–401.

[17] Wang, H.F., Lou, M.L., Chen, X., Zhai, Y.M., 2013. Structure-soil-structure interaction between underground structure and ground structure. Soil Dyn. Earthquake Eng. 54, 31–38.

[18] Zhang, X., Jiang, Y., Sugimoto, S., 2016. Seismic damage assessment of mountain tunnel. Tawarayama tunnel due to the 2016 Kumamoto Earthquake. Tunn. Undergr. Space Technol. 71, 138–148.

[19] Calisto, L., Ricci, C., 2019. Interpretation and back-analysis of the damage observed in a deep tunnel after the 2016 Norcia earthquake in Italy. Tunn. Undergr. Space Technol. 89, 238–248.

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Title: Study on the Implementation of Direct Stiffness Matrix Method in Underground Framed Structures

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Keywords: *Framed Structures, Direct Stiffness Matrix, Joint displacements. Local and Global coordinate system.*

Abstract

This paper presents a brief analysis of underground framed structures using the most common implementation of the finite element method i.e., the Direct Stiffness Matrix Method (DSM). As per the literature review, it was found that limited research has been done in the area of underground frame structure using the stiffness method. While previous research has focused on modeling plane truss, shell type, and tall buildings. This research can develop sustainable research lines and help researchers significantly increase their research output and enhance its quality. The theoretical foundation for matrix methods of structural analysis was laid by James C. Maxwell, who introduced the method of consistent deformations in 1864; and George A. Maney, who developed the slope-deflection method in 1915. These classical methods are considered to be the precursors of the matrix flexibility and stiffness methods, respectively. The matrix stiffness method was developed by R. K. Livesley in 1954. In the same year, J. H. Argyris and S. Kelsey presented a formulation of matrix methods based on energy principles. In 1956, M. T. Turner, R. W. Clough, H. C. Martin, and L. J. Topp derived stiffness matrices for the members of trusses and frames using the finite-element approach and introduced the now popular direct stiffness method for generating the structure stiffness matrix. Underground structures are constructed beneath the surface of the Earth. It is an integral constituent of the infrastructure and is utilized for a variety of applications including railways, subways, highways, sewage, water transport, and material storage. Earth, rock pressure, and water pressure, which are considered as the most important potential loads acting on underground structures are calculated initially. Varying overburden pressure is also determined which is later converted into the uniformly distributed load which acts at the top slab whereas, lateral and water pressure, as well as the horizontal component of the overburden pressure, acts at the support i.e., vertical members of the framed structure. For determining the height and length of the frame structure, the Indian Road Congress (IRC) was referred. Analytical and Codal method using MATLAB R2016a for the applied load case has been carried out for the determination of Structure stiffness matrix(S), Joint displacements (d), Fixed end forces (Qf), Member end displacement (u), and End forces (Q), and Support reactions (R) for both Local and Global Coordinate system using Direct Stiffness Matrix method which is the basic theory underlying Finite Element Analysis for analysis of tunnel structures. The stiffness method which is also known as the displacement method is the primary method used in the matrix analysis of structure which is conducive to computer programming. Once the analytical model of a structure has been defined, no further engineering decisions are required in the stiffness method to carry out the analysis. The basis of the method is that the structure to be analyzed is discretized into a number of small elements with a framed jacket structure. In applying the method, the system must be modeled as a set of simpler, idealized elements interconnected at the nodes. The material stiffness properties of these elements are then, through matrix mathematics, compiled into a single matrix equation that governs the behavior of the entire idealized structure. The structure's unknown displacements and forces can then be determined by solving this equation. Finally, the structure is checked for the permissible responses and the results are compared for the Analytical and Codal method and MATLAB R2016a was found more efficient.

Title: Uncertainty analysis of rock mass quality for a tunnel case from Nepal

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Keywords: *Tunnel; Headrace; Tunnel squeezing; Uncertainty*

Abstract

Tunneling in weak and tectonically disturbed rock mass usually faces complex and unpredicted instability problems. Stability is the major concern particularly, when tunnel pass through weak, schistose, sheared, deformed and thinly foliated rock mass. Many hydropower tunnels with weak rock mass quality and high overburden have squeezing problems (Panthi, 2006). When induced stress level exceeds the strength of rock mass, tunnel fails. Stress plays a crucial role in developing brittle fractures, rock strength reduction and rock mass instabilities. This paper uses probabilistic approach of uncertainty analysis focusing on accessing rock mass quality index using Q-method and plastic deformation (tunnel squeezing) along headrace tunnel of Kulekhani-III Hydroelectric Project in Nepal. Qsystem is used as the foundation for evaluating rock mass quality and tunnel squeezing analysis is based on a semianalytical method suggested by Hoek and Marinos (2000), an equation proposed by Panthi (2006) to evaluate rock mass strength and GUI Octave 5.1 by defining the uncertainties of variable input parameters. Results from KulekhaniIII Hydroelectric Project highlights that prediction of rock mass quality along headrace tunnel shows fair correlation between predicted and actually measured Q-values. The same principle has been used for assessing plastic deformation along the headrace tunnel. It is concluded that the uncertainly analysis method proposed by Panthi (2006) is very relevant and viable method to predict instability conditions given that the selected equations are applicable and representative to the assigned type of instability.

Title: Evaluation on unlined/shotcrete lined headrace tunnel for Tamakoshi V hydroelectric project

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Keywords: *Unlined/shotcrete lined pressure tunnel, hydraulic jacking, confinement criteria, Himalayan rock mass*

Abstract

Rock mass in the Himalayan region have undergone severe deformation due to tectonic movement and are fractured, folded, faulted, sheared, and weathered. The complex geological and geotectonic environment prevailing in the Himalaya is a challenge for successful tunneling. It is important to explore solutions that address prevailing geology and geotectonic environment, so that use of concrete lining as permanent support is limited. This article studies on the potentiality of exploiting headrace tunnel (HRT) of Tamakoshi V Hydroelectric Project (HP) as unlined/shotcrete lined pressure tunnel. The article uses Norwegian Confinement Criteria (NCC), Modified NCC and In-situ Stress state assessment for the evaluation of this possibilities. Further, the article makes detail assessment on the leakage possibility and potential use of preinjection grouting to limit the leakage.

Title: Effect of Damage Zone Around an Excavation due to Blasting

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Keywords: *phase2, disturbance factor, disturbance zone*

Abstract

Nepal has officially entered the tunnel age. The geographic condition of Nepal demands tunnels for the development of infrastructure like transportation and energy. Tunneling requires the excavation of underground material. This can be done via manual excavation, tunnel boring machine & blasting. Drilling and blasting is a common method of tunnel excavation in Nepal. Blasting of rock is a careful process of timed explosion. This helps in breaking of rock which is then excavated. However, it also creates a zone of disturbance around the excavation. This paper presents the stability analysis of tunnel with finite element method by means of software Phase2 of underground structure (tunnel) of Tanahu Hydropower. Rock mass data required for the stability analysis is obtained from Tanahu Hydropower project. The rock mass has been modelled to follow the Generalized Hoek Brown failure criterion. A comparative study between two models -one incorporating a damaged zone around the excavation due to blasting and the other without the damage zone have been performed. A thickness of 1 m is assigned around the tunnel where the effect of blasting is seen. The damaged zone has been modelled by increasing the disturbance factor of the zone. Finally, support systems for the tunnel have been suggested for the tunnel section.

Title: Performance of Concrete Lining used in Tunnel Support by Physical Scaled Model and Finite Element Modelling

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Keywords: *Tunnel, Weak rock mass, Physical Scaled Model, Rock Mass Classification, Reinforcement*

Abstract

This study is focused on performance of concrete lining as tunnel support system using physical scaled model and its Finite Element Modelling. Empirical approaches of Rock mass classification such as Q-system and Rock Mass Rating (RMR) are commonly used for estimation and design of tunnel support in the Nepal Himalaya. In the field application in hydropower tunnels, there has been many failure instances of estimated tunnel support in the form of squeezing and collapse of whole opening. To prevent such failures, there is practice of modification to its estimated support elements based on the field conditions. The modification is done with the reinforcement of steel rebar and steel ribs as per requirements.

Generally, Concrete lining has been commonly provided as a final support for the tunnel passing through weak rockmass in-addition to support estimated from rock mass classification. With the objective to study the performance of concrete lining under the overburden of rock and soil mass above the tunnel, physical scaled models of concrete lining have been made at laboratory with scale of 1:10. The vertical stress due to overburden has been applied using vertical point load at the crown of the support and its effect has been studied in terms of displacement. The load has been gradually increased up to the point of the failure. The failure point has been defined as the point in which cracks are developed. The corresponding maximum load and displacement are taken as design load and displacements. Hydraulic jacks, strain gauge, dial gauge and steel loading frame has been used to study the load and corresponding displacements.

Three different support system of concrete lining have been developed. They are i) without no reinforcement, ii) with reinforcement of steel rebar and ii) with reinforcement of steel ribs respectively. In order to validate the results from experimental models, comparison has been made with the FEM models. FEM models have been developing the similar loading and boundary conditions of physical scaled models. It has been found out that concrete lining with steel rib is more flexible and can carry more load in comparison to the concrete lining with and without steel rebar.

Title: Finite Element Analysis of Underground Water Tank Considering Soil Structure Interaction and Water Sloshing Phenomena

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Keywords: *Soil Structure Interaction (SSI); Underground Water Tank (UGWT); Water Sloshing; Time History Analysis; Spring Mass Model*

1. Introduction

This study states that the effects of soil structure interaction (SSI) including sloshingeffect of water on the Underground Rectangular Water Tank (UGWT) having capacity to store 900 m^3 of water located at Banepa, Kavrepalanchok district of Nepal.The main purpose of this research is to studyinfluence of SSI on seismic resistant design of water tank because seismic codes lack recommendations concerning the effect of SSI. It is very important to include SSI in earthquake resistant design of structure considering less-favorable geotechnical condition in Nepal.

Literature shows that structures founded on rock are considered to be fixed-base structures and the same structure would respond differently if supported on a soft soildeposit. First, the inability of the foundation to conform to the deformations of the free- field motion would cause the motion of the base of the structure to deviate from the free-field motion which is also known as kinematic Interaction. Second, the dynamic response of the structure itself would induce deformation of the supporting soil which is also known as inertial (dynamic) interaction (M.A. Hashash et al. , 2001). In this research the effect due to sloshing behavior of water and dynamic behavior of sub soil on UGWT will be studied.

2. Literature Review

For seismic analysis NBC 105: 2020 gives Seismic Zoning Factor (Z) represents peak ground acceleration (PGA) for 475 years return period and the PGA for nearest to Banepa city is 0.35g.

There are two methods for the analysis of SSI,and these are direct and substructure spring method. In substructure spring method (NEHRP-2012) of analysis the foundation input motion is assumed to be the same as free-field motion, *i.e.* the effects of kinematicinteraction are neglected in soil structure interaction analysis.

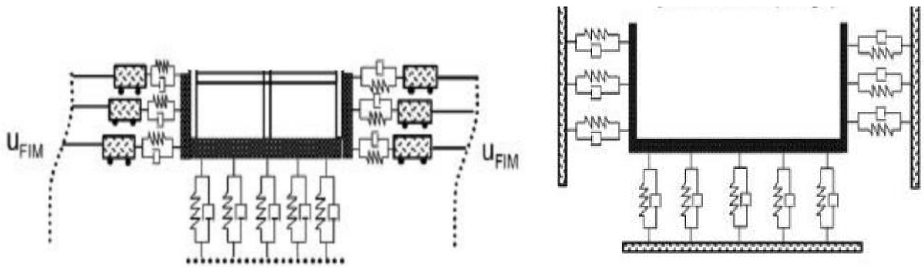

Fig-1: *Soil -Structure Interaction SystemAssumption from NEHRP 2012*

Along with Soil structure interaction for seismic design of Underground water tank, the total forces exerted by water on tank wallare considered. (ACI 350.3-01) recommends forces to consider in underground liquid containing structures. Hydrodynamic forces exerted by liquid on tank wall are consideredin the analysis in addition to hydrostatic forces.

According to (IS 1893 (part-II) :2014) and Housner (1963) to consider water sloshing effect on UGWT, the water tank can be idealized as spring mass model as shown in Fig-2. (IS 1893 (Part-II): 2014) recommends onemass approximation provision for the tank below 1000 Kilo Liter capacity. For one massapproximation (IS 1893 (Part-II): 2014) recommends that the total water mass is assumed in impulsive mode only, here the convective mode is ignored.

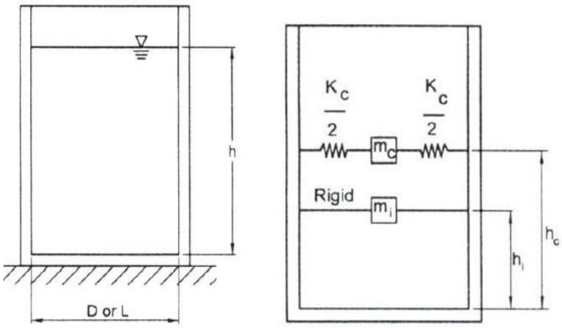

***Fig-2: Spring** Mass Models for Water Tank (IS1893 part (II) : 2014)*

3. Methodology

The Underground water tank is modeled and analyzed in finite element tool SAP2000 for fixed and flexible support conditions. All components of the tank, were modelled as area sections that used linear-homogenous shell elements having isotropic material property. The SSI analysis model consists of a single degree of freedom structure of heighth, mass m, stiffness k, and viscous damping coefficient c. The properties of springs are calculated from different standard penetrationtest (SPT) values, Poisson's ratio and elasticity of soil (*Source: Soil Investigation Report of particular site provided by Clay Engineering Consultancy Pvt.Ltd. , Gangakhel , Kathmandu*). For water sloshingeffect the water mass is modeled as one massmodel *i.e* impulsive mode is taken and the impulsive force is assumed to act at center ofgravity of the whole water mass (IS 1893 (part-II) :2014). Time history analysis is carried out to determine seismic response of water tank with three recorded ground motions including Gorkha Earthquake-2015. The seismic analysis is carried out using timehistory analysis for underground structures.

4. Results

This study finds and compare the different modes of frequencies, corresponding time period, stresses on walls and slabs of tank along with the deformations for fixed and flexible support conditions. Some of the general results obtained are;

- There is difference in response forfixed and flexible support conditions due to soil flexibility.
- There is difference in response of theUGWT while considering water asstatic load and while considering water as dynamic load.
- There is difference in response of theUGWT for different earthquake time history analysis.
- There is difference in stresses values given by Design code (IS 3370-IV: 1967(2008)) in comparison with finiteelement analysis results.

5. Conclusions and Recommendations

Time history analysis is performed to obtain the response of UGWT along with consideration of soil flexibility and water dynamics. It is obtained that there is difference in response while considering water as static and dynamic loading. The study is limited to linear time history analysisfor three ground earthquake in rectangular tank with capacity less than 1000 Kilo Liter.And, further study can be done in different shape and size of water for different seismic zones.

6. References

ACI 350.3, Seismic design of liquid containing concrete structures. (2001). Michigan: American Concrete Institute.

IS : 1893 (Part2): Criteria for Earthquake Resistant Design of Structures. (2014). Bureau of Indian Standards. New Delhi, India:

IS :3370(Part IV)-1967(2008): Code of Practice for Concrete Structures for The Storage of Liquids (2008). Bureau of Indian Standards. New Delhi, India.

SAP2000, Structural Analysis and Design Program. (2019). United State of America: COMPUTERS & STRUCTURES. INC

Housner, G. (1963). The dynamic behaviour of water tanks. Bulletin of the Seismological Society of America, 53, 381-387.

Soil structure interaction for building Structures by NEHRP, NIST GCR 12-917-21, September 2012.

NBC 105: 2020. Seismic Design Builings in Nepal, Ministry of Urban Development

M.A. Hashash et al., 2001. Seismic Design and Analysis of underground structures. Tunnelling and Underground Space Technology 16 (2001) 247-293

Soil Investigation Report of particular site at Banepa (2021) : Clay Engineering Consultancy Pvt.Ltd , Gangakhel , Kathmandu

Title: Review paper on design of shotcrete lining

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Keywords: *Design, Shotcrete Lining, Method*

Abstract

Shotcrete is widely used as a support element in the construction of underground structure like mines and tunnels. The shotcrete is sprayed close to the excavation face to accommodate inward radial displacements prior to the installation of a final, permanent lining. For the design of the support system, various researches have been done and several empirical, analytical methods are developed. These empirical methods provide recommendations for support selection based on rock mass quality. Analytical methods are based on several assumptions that limit their applicability. So the Numerical methods have been used to overcome the limitations fromanalytical and empirical methods. The thickness of the shotcrete lining to be used is determined from the Chart given by Grimstad and Barton (1993). This chart is being used for the support estimation of tunnel in case of Nepal also but the geology of the Nepal does not match the geology considered in the system. So the chart may not be enough and required some numerical modeling for the determination of the shotcrete lining thickness. The purpose of this paper is to give update on the latest design philosophies and methodologies, and to discuss the future development trends for shotcrete lining. Various Research papers related to the design of shotcrete will be studied and the methodology used and conclusion drawn from the research will be reviewed. The paper starts with the brief overview of the design of shotcrete lining that has been used in the past. After that the paper will introduced to some recent developments in the numerical modeling techniques to facilitate the design of the shotcrete concrete lining will be introduced. In conclusion, the paper will discuss the possible design of the shotcrete lining design in the context of Nepal with its potential benefits followed by possible improvements in the numerical modeling in case of weak rock mass.

Title: Stability Analysis and Optimization of Support System for Cavern in Weak Rock Mass

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Keywords: *Stability Analysis, Support System, Weak Rock Mass*

1. Introduction

Weak rock is the one that fails when subjected to the stress. Sedimentary rocks such as bedded sandstones, shales, siltstones, and mudstones are among the rocks that fall into this category. Input parameters for the selection of tunnel support system like rock mass designation, joint sets, roughness, degree of alteration, deformation modulus, etc are constantly varying. So, to quantify these parameters and standardize the optimized way of support system is the purpose of this study. In this study, two-dimensional (2D) elastoplastic analysis shall be performed using the finite-element code Phase 2D to study the rock mass behavior caused by staged excavation of thecavern. We plan on to determining the appropriate support system as per the support pressureand the plastic zone around the cavern.

2. Objectives

- To propose the optimized method of support system for weak rock mass.

3. Scope

- A systematic Design and optimization method of weak rock support system shall be proposed with respect to engineering geological investigation, loading conditions and rock mechanics test.
- The qualification of the rock mass will be assessed by using the Q-system; to assess the stability of rock masses for support design and also to determine the rock mass properties.
- The design variables of minimum shotcrete thickness and the optimal installation position for the rock support shall be sort out.
- The numerical modeling shall be performed to validate the optimized support combination design.
- Potential geotechnical hazards and the ground responses of different support design shall be identified.

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