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Stability Analysis of the Portal and the Headrace Tunnel Section of Khimti–II Hydroelectric Project.

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Abstract

Construction of underground structures in the Himalaya region is difficult due to the region's complicated geological and topographical conditions. Geological elements such as orientation of foliation, joint sets, joint characteristics affect the stability of the underground construction. Efficient and cost effective hydropower projects need careful design, placement and alignment of underground structures in relation to geological, topographical and stress concentrations. This paper deals with the case study of the Khimti-II Hydropower Project where planning and placement of Portal and Headrace Tunnel with respect to different factors affecting its stability was studied. The Orientations of joint sets are analysed using Dips to find out the possible mode of failure. Factor of safety of possible wedge failures is calculated using UnWedge and is increased to safety after applying support. The distribution of principal stresses and maximum displacement around the tunnel contour is shown using Phase2 software.

Keywords

Stability Analysis, Dips, Unwedge, Phase2

1. Introduction

Nepal, a landlocked country with mountains, has more than 6000 rivers (including small streams and branches) that flow from the high Himalayas to the Indian ocean. These rivers offer great potential for hydroelectric projects in Nepal's private and public sectors. Nepal spans about 890 kilometres from East to West and has a width of 150 to 250 kilometres. The country's altitude changes dramatically from about 100 meters above sea level in the South to 8,848.86 meters above sea level (the Mount Everest) in the North, creating rough and steep terrain and mountainous landscape. Nepal Himalaya has five different geotectonic zones from north to south: Tibetan Tethys, Higher Himalaya, Lesser Himalaya, Siwalik Range, Terai Plain. The steep topography in Nepal's Northern region provides good head for producing high capacity projects with low river discharge [1].

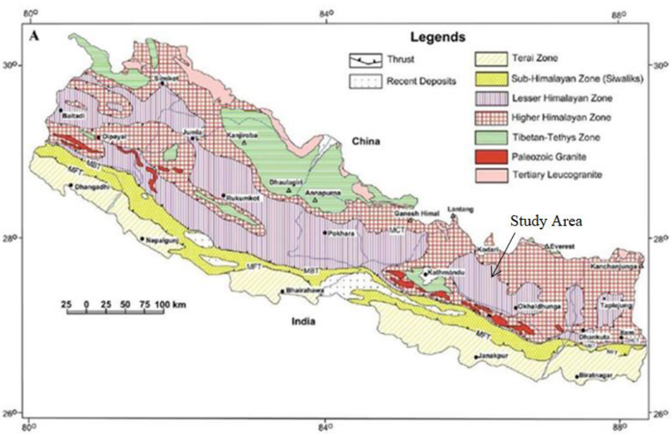

Figure 1: Project Location Map modified from [2]

Nepal has a lot of potential for underground structures for water, power, transport, storage and shelter. But, it faces stability issues due to the weak regional geology and mountainous landscape. New methods and techniques of tunneling have been invented over time. These techniques helped in achieving the desired progress in the shortest time with less accidents and problems. The rock masses in Nepal and the Himalayan region are highly affected by tectonic movement. They are folded, faulted, sheared, fractured and weathered deeply. This causes many stability problems in the complex geological setting during tunneling. This is the main challenge that needs to be solved and addressed scientifically to make the tunnel option cheaper, feasible and safer [3].

1.1 Project Description

Khimti-2 Hydroelectric Project is a RoR power generation project with an installed capacity of 48.8MW. It is situated in Jiri Municipality and Tamakoshi Rural Municipality of Dolakha District and Gokulganga Rural Municipality of Ramechhap District. The project has a gross head of 355m and a net head of 342.92m. It is located in the border of Dolakha and Ramechhap District of Nepal. The project uses the Khimti River (a major branch of Tamakoshi River). The Khimti River starts at EL. 4500m and joins the Tamakoshi River at EL. 600m. The river section from the dam to powerhouse is about 7 km long. The riverbed at the dam is at EL. 1,627m; the riverbed at the powerhouse tailrace is at EL. 1,278m. [4]

The project site is situated in the Lesser Himalayan Midland zone of Central Nepal. The main rocks in this region are metasandstone, phyllitic schist, banded gneiss and augen gneiss. Specifically, the project area is mostly composed of augen gneiss, schist and banded gneiss. This region is near Midland thrust fault, with the orientation generally NE-SW and dipping towards northwest [4]

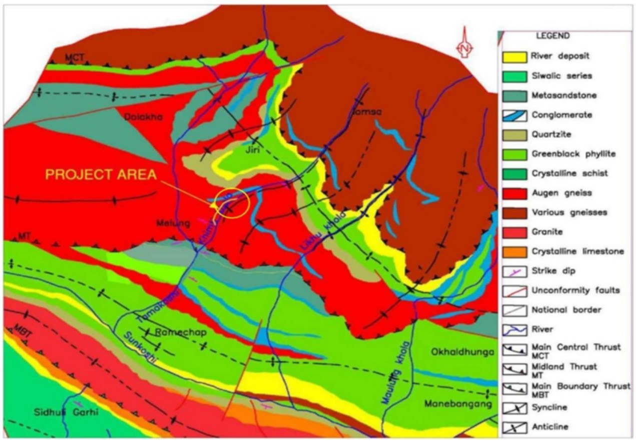

Figure 2: Regional Geological map of project area.(Source: Basic Design Report) [4]

The site has schist and augen gneiss as rock types. The left bank of the project area has thin to thick overburden material of colluvium deposit covering the bed rock. The right bank of the Khimti River has augen gneiss well exposed along the headrace tunnel alignment, while the left bank hillslope has thick colluvium and landslide deposit. The schist alternates with gneiss with different interval parallel to the bed rock foliation plane. The augen gneiss is slightly to highly weathered, foliated to massive, blocky and seamy jointed. Three plus random joint sets are common in the whole project area. Two of them are nearly vertical joint sets that are local in different areas. Roughness is rough, undulating to planer, with silt as infilling material. Schist is highly weathered,highly/thinly foliated, deformed and squeezed. Joint set roughness is rough irregular to undulating, or rough planar in some part. [4]

The main objective of the study is to perform stability analysis of the portal area and section of headrace tunnel of Khimti 2 hydropower project.

The broad objective will be obtained from integration of the following objectives.

- To assess the structurally controlled failures in the portal area and in the headrace tunnel.
- To assess the stress induced problems in the headrace tunnel through numerical modelling.

2. Literature review

2.1 Literature Review Related to Structurally Controlled Failure.

Rock masses are mostly made up of intact rock blocks separated by a system of cracks. These cracks can be random or regular features of a crack set. This system of cracks is often called the structural fabric of the rock mass and can include layers, joints, foliation, or any other natural break in the rock. In most cases, the engineering properties of cracked rock masses, such as strength, permeability, and deformability, depend more on the nature of the structural fabric than on the properties of the intact rock. For this reason, rock mechanics experts have developed the following parameters to describe the nature of the cracks that form the structural fabric:

Orientation: The orientation of a crack is best described by two angles: dip and dip direction.

Persistence: Persistence means the continuity or area of a crack and is very important because it defines the possible volume of the failure mass. Persistence is hard to measure; the only reliable way is mapping of rock exposures.

Spacing: The spacing is the distance between two cracks of the same set measured perpendicular to the crack surfaces. Persistence and spacing of cracks define the size of blocks.

Surface properties: The shape and roughness of the crack are its surface properties, which affect the shear strength directly. Infillings: Infillings are minerals or other materials that occur between the intact rock walls of cracks. They can affect the permeability and shear strength of a crack [5].

Types of Slope Failures: Most rock slope failures can be classified into one of four categories depending on the type and degree of structural control:

- Planar failures: These are governed by a single discontinuity surface dipping out of a slope face.
- Wedge failures: It involves a failure mass defined by two discontinuities with a line of intersection that is inclined out of the slope face.
- Toppling failures: It involves slabs or columns of rock defined by discontinuities that dip steeply into the slope face.

When tunnels are dug in cracked rock masses at relatively shallow depth, the most common failures are wedges falling from the roof or sliding from the sidewalls of the openings. These wedges are made by crossing structural features, such as layers and joints, that separate the rock mass into different but locked pieces. When an opening is made by digging the tunnel, the support from the surrounding rock is gone. One or more of these wedges can fall or slide from the surface if the planes are continuous or rock bridges along the cracks are broken.

If these loose wedges are not supported, the stability of the back and walls of the opening may get worse quickly. Each wedge, that is allowed to fall or slide, will reduce the support and the locking of the rock mass and this, in turn, will let other wedges fall. This failure process will go on until natural arching in the rock mass stops more falling or until the opening is full of fallen material.

The steps which are required to deal with this problem are:

1. Determination of average dip and dip direction of significant discontinuity sets.
2. Identification of potential wedges which can slide or fall from the back or walls.
3. Calculation of the factor of safety of these wedges, depending upon the mode of failure.
4. Calculation of the amount of reinforcement required to bring the factor of safety of individual wedges up to an acceptable level.

Finding possible wedges: The potential wedges in the rock mass around an opening depend on the opening's size, shape and orientation and also on the orientation of the important discontinuity sets. The problem's three-dimensional geometry requires a set of complex calculations. These can be done by hand, but it is much easier to use one of the computer programs that are available. One such program, called UNWEDGE, was made specifically for underground hard rock mining and is used in the following discussion.

2.2 Literature Review Related to Stress induced instability.

Rock stresses can cause instability when they are higher than the rock mass strength. There are two main types of instability caused by rock stresses; 1) rock burst / rock spalling, and 2) tunnel squeezing or deformation. Rock spalling is cracking parallel to the tunnel edge happening usually in strong and brittle rock masses. If the cracking is with loud sounds and vibrations, this is called rock burst. The risk of rock burst or spalling in a tunnel is usually based on the ratio between maximum tangential stress and the rock mass strength (about 50 percent of the uniaxial compressive strength). Some authors, like Hoek and Brown (1980), Broch and Sørheim (1984) and Grimstad and Barton (1993), have suggested criteria for assessing rock burst and spalling in tunnels.

2.3 Literature Review Related to Failure Behavior in Tunnel.

Rock mass failure can happen without discontinuities. There are cases for tunnel where the rock metrics is weaker than the stress and hence it can fail. Making an opening in a rock mass changes the stress distribution in the ground, some stresses would go up and some would go down. The stress increase could cause failure. For opening, failure usually happens near the excavation wall.

Hoek and Brown (1980) described four main causes of underground instability.

1. High rock stress failure related to hard rock. This kind of failure can happen e.g. when mining at deep levels or for big excavation at shallow levels. Stress conditions for tunneling in high mountain regions or in weak rock conditions can also cause stress-induced instability problems
2. Structurally controlled failure often happens in jointed and faulted hard rock, especially when several joint sets are steeply sloped.
3. Weathered or swelling rock failure usually related to relatively poor rock. This kind of failure may also occur in isolated seams within as otherwise solid hard rock.
4. Groundwater pressure or flow induced failure, which can happen in almost any rock mass. If the failure is combined with any of the other types of instability mentioned above, it could be very serious. [6]

2.4 Literature Review Related to Numerical Modelling.

Various researchers have used FEM (Finite Element method) showing versatility of the method towards successful implementation in various rock engineering problems.

Many researchers have applied FEM (Finite Element method) to show the flexibility of the method in solving various rock engineering problems. In Phase2, field stress can be fixed or gravity stress. The gravity field stress option is used to define a gravity stress field that changes linearly with depth from a user given ground surface elevation. Gravity field stress is usually used for surface or near surface at shallow depth elevations and the areas where the topography affects stress magnitudes and directions. Stress ratio is calculated with Poisson's ratio. Also, the material parameters such as unconfined compressive strength of intact rock(sigmaci), Hoek-Brown constant (mi), Geological Strength Index (GSI), Young's Modulus of Intact Rock (Ei), Poisson's ratio (v), density of rock mass are the inputs to the material property.

The principle stress can be shown and the results can be seen. The stress level could be checked in specific location of the analysis. The major and minor principle stress and angle between stresses with horizontal can be used to find the vertical and horizontal stress at that point and the result can be compared with the gravity and tectonic stress.

The strength factor of the rock mass around the tunnel can be shown with contours. With the elastic analysis if the strength factor is more than 1 everywhere around the tunnel, the result will be the same even if the plastic analysis is done. So there is need of plastic analysis if the strength factor is less than one around the tunnel with elastic analysis.

The value of vertical stress, horizontal stress, and total displacement can be shown with the contour around the tunnel. The value can be compared with the result from analytical, semi-analytical method and also with the value of measured convergence.

3. Methodology

The purpose of this project work is to evaluate the structurally controlled failure in the portal and headrace tunnel section and to learn about the rock mass properties that affect the stability of a section of the headrace tunnel of the Khimti-2 hydroelectric. To achieve these objectives, the research methodology as shown in the flow diagram (Figure 3) is used.

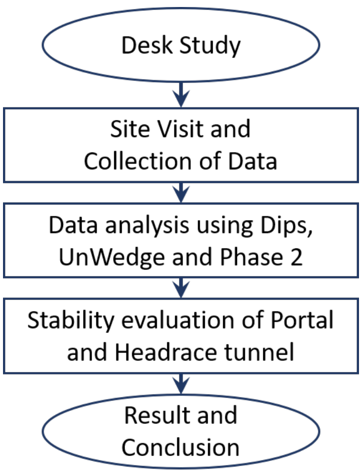

Figure 3: Methodology

3.1 Description of Methodology

Research methodology is the specific methods or techniques used to collect, process, and analyze data about a topic. In a research, the methodology section allows the reader to critically assess a study's overall quality and reliability. The methodology of this thesis is summarized in Figure 3. The methodology of the research is described individually in this section. The research starts with the literature reviews which help to identify the research question and selection of case study in that basis. The collection of data from the field visit, observation, previous study, previously published research for the calculation of rockmass properties. This is followed by the numerical analysis from rocscience software Phase2.

Desk Study: Desk study consists of literature review, study of report, journals, papers, articles and old thesis documents. All the available previous study reports, data/information including maps and drawings and other information related to the study area was studied and analyzed in depth in the context of the objectives of the study.

Data Collection: It includes the project details and engineering geological information on the rock mass condition. Geological properties of rock masses will be referred as provided. General data such as unit weight, modulus of elasticity, Poisson's ratio, uniaxial compressive strength and other properties of rock mass were obtained from empirical relations. Whereas the specific data of the tunnel i.e. dimension, plan and ground profile of tunnel alignment, rock type and rock mass classification were obtained from detail project report of Khimti-II HEP and face maps present at the site.

Study site visit was conducted several times to collect various geological data, design data and their field verification. Site visit was focused on various activities discussed below:

- For Geological mapping at the various chainage.
- To study surface geology of the study area.
- To study various support system.
- Rock identification and verification as provided by the project report.
- Water table of the study area.
- Rock mass classification in tunnel.

Determination of Rock mass Parameter:

The modulus of deformation of rock mass (Em) is the ratio of stress to strain during loading of rock mass, including elastic and inelastic behavior while the modulus of elasticity of intact rock (Ei) is the ratio of applied stress and strain within the elasticity limit. The jointed rock mass is not elastic. So, the term modulus of deformation is used instead of modulus of elasticity. The deformation modulus of jointed rock mass is very low compared to the elasticity modulus of intact rock. To design an underground excavation, rock mass characteristics need to be estimated. Methods like the generalized Hoek-Brown criterion and MohrCoulomb failure criterion can be used to describe the rock mass behavior like strength and deformations. Data from core samples are often used to estimate the properties of intact rock (no weakness planes) and from that point through empirical approach to estimate the overall characteristics of the rock mass around an underground opening. Strength of intact rock sample is usually higher than the overall strength of the rock mass and methods are therefore needed to convert data from core samples to the rock mass[6].

Effects of rockmass properties on stability:

For study about the effect of various rock mass properties on the stability of the portal and headrace tunnel, Dips, UnWedge, Phase 2 software is used. The joint sets were determined and kinematic analyses were performed using dips. The geometry and stability of underground wedges defined by joint sets were determined by UnWedge. The principal stresses, strength factor, total displacement around the tunnel excavation were determined by Phase 2 software.

4. Results and Discussions

The orientation of foliation and joint sets were measured in the project site or were collected from the face mappings of the tunnel. After collecting joint information, Best Tunnel alignments were found from the rosette plot and possibility of plain and wedge failure is verified using DIPS software developed by Rocscience. Three representative major joint sets are identified in the adit-2 section of the project site from where the data were collected. These representative joint planes are as shown in Table.

Table 1: Orientation of Joint sets and Hill slope

Discontinuity	Dip	Dip Direction
Foliation (J1)	21°	344°
Joint sets (J2)	70°	142°
Joint sets (J3)	70°	198°
Hill Slope (HS)	75°	170°

The favorable alignment for the Head Race Tunnel was found to be 170° and the alternative alignment to be 80° from the rosette plot as shown in figure 4. The current trend of the tunnel is 162°. So, the tunnel is constructed in its favourable alignment:

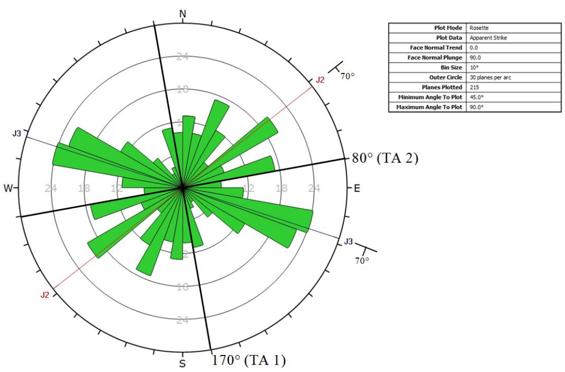

Figure 4: Rosette plot showing tunnel alignment.

4.1 Kinematic Analysis

The kinematic analysis of the above three representative joint sets were done for the Plane failure condition.

As shown in Figure 5, planar sliding is not likely to happen along foliation plane. The joint sets J2 and J3 cut the hill slope and fulfill all other conditions of plane failure except that their strike is not parallel to the hill slope (i.e. hill slope +/- 20°). However, multiple joints can form wedges for any cut slope which can slide along the line of intersection between two planes or can slide along one joint plane while the other joint plane forming the wedge can work as a releasing plane.

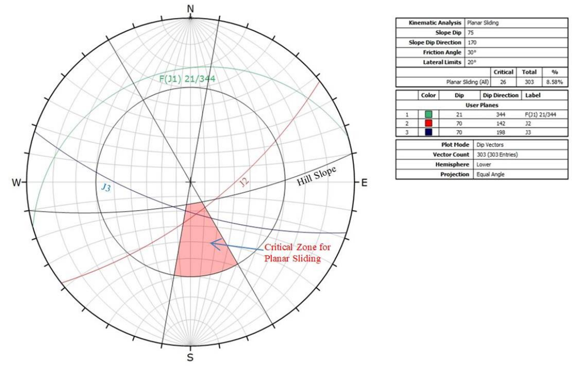

Figure 5: Plane failure analysis using dips. (critical: 8.58%)

Wedge failure analysis is carried out in dips for the representative joint sets as shown in figure below.

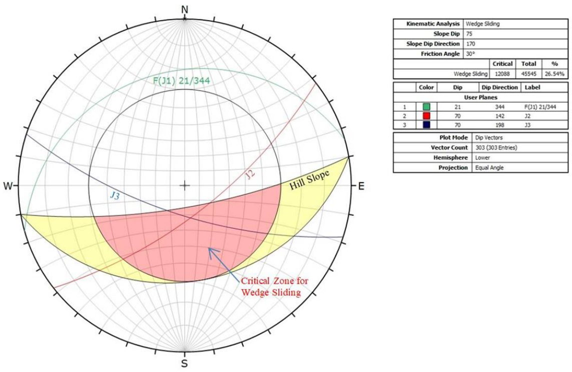

Figure 6: Wedge failure analysis using dips. (critical: 26.54%)

As shown in Fig 6, Wedge is formed by the joint sets J2 and J3 which falls under the critical zone for wedge sliding so there is a possibility of wedge failure. For finding the size, position and the factor of safety of the wedges formed in the portal we can use SWedge, another software by Rocscience.

4.2 Block Stability Analysis

The Trend and Plunge of the Headrace Tunnel is 162° and 0.573° respectively. The three representative joint sets at each chainage as obtained from the face mapping are used for the analysis. The unit wt. of rock mass is assumed to be 2.7t/m3. The friction angle of the joint is assumed to be 30° while the cohesion to be 10t/m2. The analysis was performed on five faces at a distance of about 25m for a section of about 100m in the headrace tunnel.

Tunnel Axis Orientation		Shear Strength		Rock Bolt Properties	
Trend:	162°	Model Used:	Mohr-Coulomb	Bolt Type:	Fully Grouted
Plunge:	0.573°	Φr:	30°	Tensile Capacity:	10 t
Unit Weight		C:	10 MPa	Plate capacity:	10 t
Rock:	2.7 t/m³	Assume pore water pressure to be zero.		Bond Strength:	5 t/m
Water:	0.981 t/m³			Bolt of length 2m is used.	

Figure 7: Table of Input Parameters for Wedge Analysis.

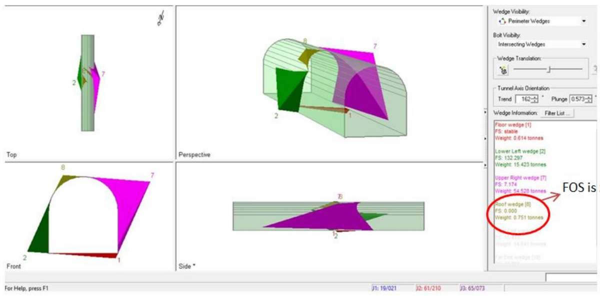

Figure 8: Wedges formed at chainage 6+001.80m.

The weight of the roof wedge formed at chainage 6+001.80 as shown in figure 8 is 0.751 tons while its factor of safety is zero. Therefore, spot bolting is used which increased its factor of safety to 10.356 as shown in figure 9.

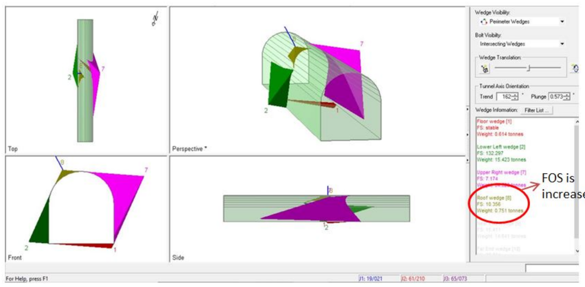

Figure 9: Wedges formed at chainage 6+001.80m with spot bolting.

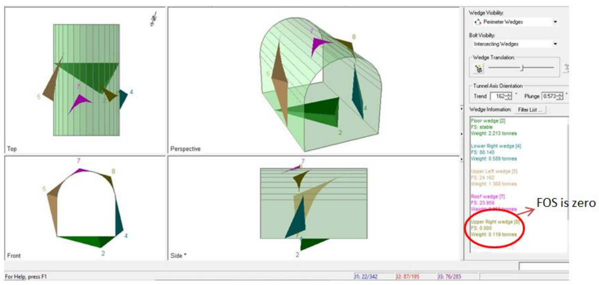

Figure 10: Wedges formed at chainage 6+025.40m.

The weight of the wedge formed in the roof is 0.119 tones at chainage 6+025.45, the factor of safety is increased from zero to 50.37 by applying the spot bolting as shown in figure 10 and figure 11.

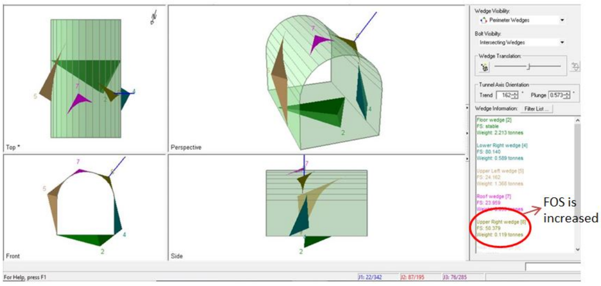

Figure 11: Wedges formed at chainage 6+025.40m with spot bolting.

At chainage 6+049.90.

The roof wedge is not formed at this chainage and other wedges formed at the sidewalls have sufficient factor of safety. So, it is not necessary to use spot bolting.

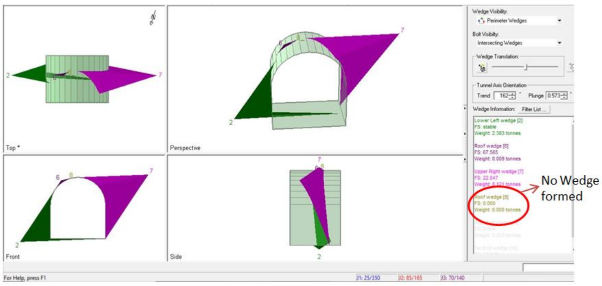

Figure 12: Wedges formed at chainage 6+049.90m.

The weight of the roof wedge formed is 0.013 tones at chainage 6+074.75, the factor of safety is increased from zero to 776.84 by applying the spot bolting as shown in figure 13.

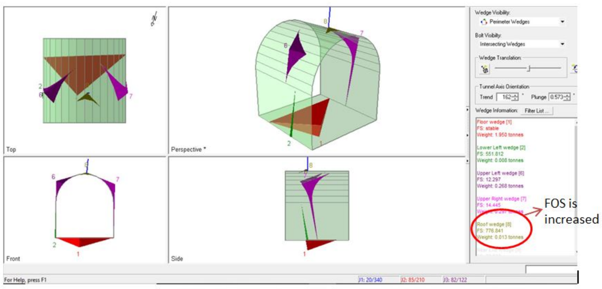

Figure 13: Wedges formed at chainage 6+074.75 with spot bolting.

The weight of the roof wedge formed is 0.417 tones at chainage 6+095.85, the factor of safety is increased from zero to 18.65 by applying the spot bolting as shown in figure 14.

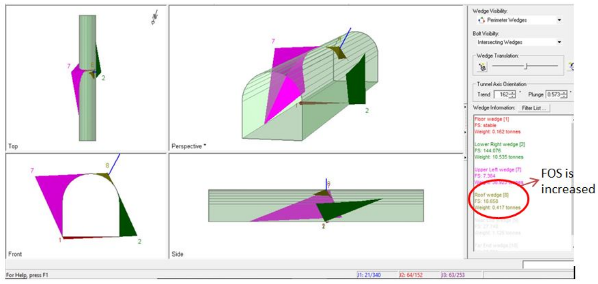

Figure 14: Wedges formed at chainage 6+095.85m with spot bolting.

As Shown in Figures above, Large wedges are formed at the upper left and upper right section of the tunnel but their factor of safety is sufficient enough so they are stable. However, the wedges formed at the roof of the tunnel are prone to danger; their factor of safety is less so they must be addressed while designing the rockbolt support or additionally they can be treated with spotbolt to increase their factor of safety. The weight and the factors of safety of the wedges formed are shown on the right sections of the respective figures.

4.3 Numerical Modelling

Rock mass modeling is a very hard task because of the discontinuities, anisotropic, heterogeneous, and nonelastic nature of rock mass, using empirical and numerical methods. Rock masses are difficult materials for empirical and numerical modeling because of their complex nature and different formation. The empirical methods do not measure the performance of support systems, stress redistribution, and deformation around the tunnel. These parameters are very important to consider in designing of optimum underground structure and support systems. The numerical methods solve the problem of empirical method. Numerical methods give the exact mathematical solution for the problem based on the engineering judgment and input parameters.

For Khimti-II HEP, Major rock types found in the site are augen gneiss and phyllitic schist intercalation. The rock mass is classified into rock class III and rock class IV through out the section considered and input properties assigned based on the same classification as shown in figure 15 and 16. For this type of rock, value of Mi is taken as 18 for further analysis as an input parameter. The disturbance factor is taken as 0.6 for about 1.5m around the excavated section for the poor blasting method. The Laboratory test for various mechanical properties like poisson's ratio, bulk density and modulus of elasticity are not carried out. Therefore, those parameters have been taken from consultation with the supervisors working on the same project.

Material: Gneiss Class III		Model Used: Generalized Hoek-Brown		
Unit Wt.:	0.027 MN/m³	D	0	0.6
Young's Modulus:	20000 MPa	mb parameter:	1.966	0.761
Poisson's Ratio:	0.25	s parameter:	0.00101	0.00018
Intact UCS:	75 MPa	a parameter:	0.5130	0.5130
GSI is calculated from Q-value obtained from facemaps and mb, s & a calculated from Hoek-Brown equations.		mi parameter:	18	

Figure 15: Table of input parameter for Gneiss (rock class III).

Material: Gneiss Class IV		Model Used: Generalized Hoek-Brown		
Unit Wt.:	0.027 MN/m³	D	0	0.6
Young's Modulus:	20000 MPa	mb parameter:	1.327	0.434
Poisson's Ratio:	0.25	s parameter:	0.0003	0.000039
Intact UCS:	60 MPa	a parameter:	0.5273	0.5273
GSI is calculated from Q-value obtained from facemaps and mb, s & a calculated from Hoek-Brown equations.		mi parameter:	18	

Figure 16: Table of input parameter for Gneiss (rock class IV).

Tunnel is excavated in inverted D shape with 3.80m width and 4.20m height. Disturbed zone of 1.5m around excavation is considered and a box type of external boundary with an expansion factor of 5 is created for modelling as shown in figure 17.

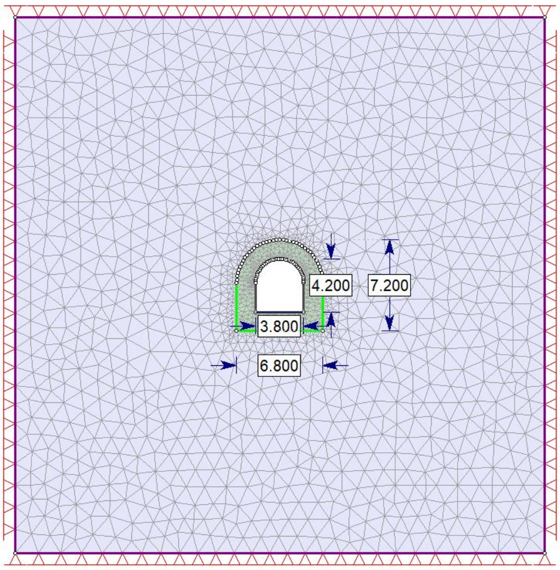

Figure 17: Phase2 model of excavated tunnel with external boundary and disturbed zone.

The stresses are the result of the vertical stress from gravity, tectonic stresses, topographic and residual stresses. These vertical stresses are changed after the excavation. Vertical stress is caused by the overlying strata. If we assume the rock mass is homogenous and isotropic, the vertical stress is from the overlying strata,

In situ measurement is not carried out for this project, therefore gravity model is used in modelling. Hoek and Brown (1980) have discovered that the ratio (K) between horizontal and vertical in-situ stresses changes a lot and that the average horizontal stress near the surface is in most cases higher than the vertical stress.

The ratio k is more than 1 at shallow depths, but it is less than 1 and becomes a constant value at great depths (McCutchen, 1982), this means that the plate tectonic movements have a big effect on the average horizontal stress. The horizontal stress to vertical stress ratio (K) is assumed to be 1.5 for this case as per previous experience.

Rockbolt pattern and Shotcrete is adopted as the support for Rock class III and IV. The properties of rockbolt and shotcrete used are listed in the figure 18 below.

Rock Bolt Type: Fully Bonded		Liner Type : Standard Beam	
Bolt Length & Dia:	2.5m & 20mm	Material Type:	Plastic
Bolt Modulus (E):	20000 MPa	Thickness:	0.1m
Tensile & Residual Tensile Capacity:	0.1 & 0.01 MN	E & ύ	30000 MPa & 0.20
In & out plane Spacing:	1.2m & 2m	Compressive Strength:	Peak: 25 MPa Residual: 5MPa
Orientation:	Normal to Boundary	Tensile Strength:	Peak: 5 MPa Residual: 0 Mpa

Figure 18: Table of input parameter for Support properties.

The excavated rock mass consists of highly foliated, weathered, highly jointed, light gray to white Gneiss intercalated with highly foliated, weathered, coarse grained, light greenish gray to dark gray schist. The rock mass consists of three sets plus random joint sets. J1(foliation) is high persistence having open to moderate separation and slickensided, undulating, J2 of high to medium persistence open to tight separation, J3 and JR have low to medium persistence. The ground water condition is minor inflow.

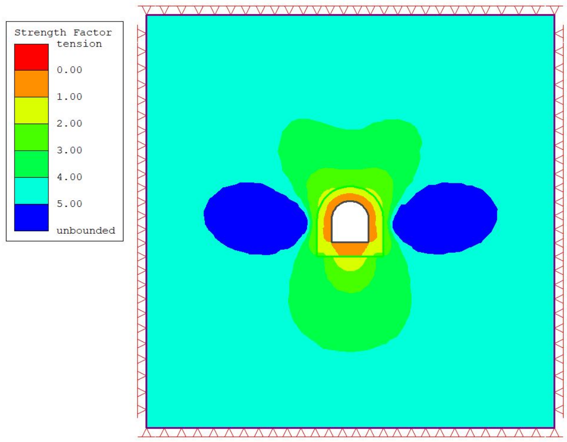

Figure 19: Phase2 model showing Strength factor.

In elastic analysis, the material type is considered as elastic that means rock mass behaves elastically. The major concern of this analysis is to find the strength factor around tunnel periphery that was shown in the Figure 19. The strength factor is less than one around the tunnel in both cases with and without support. If the strength factor is less than one in elastic analysis, there will be failure of the material and for more additional information plastic analysis would be necessary [6]. Strength factor is less than one for both rock classes considered. Hence, Plastic analysis is done.

Figure 20 below shows the displacement contours around the tunnel cross section after the application of rockbolts and shotcrete in rockclass III, the maximum displacement occuring before and after the application of supports is 0.00120m and 0.00112m respectively at the side wall of tunnel.

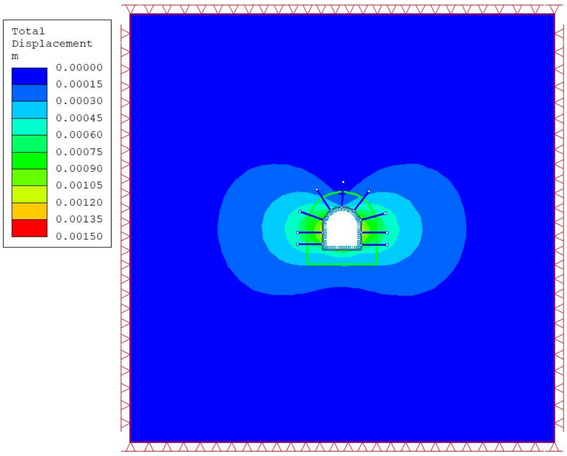

Figure 20: Phase2 model showing displacement contour around the tunnel section in rock class III.

The number of yielded elements decreased from 319 elements to 246 elements after the application of supports in rockclass III as shown in Figure 21 below.

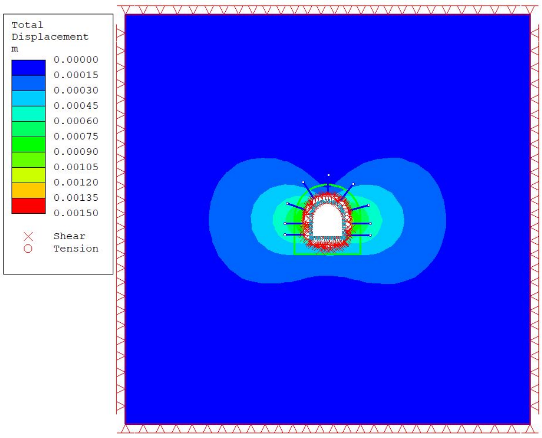

Figure 21: Phase2 model showing yielded elements around the tunnel section.

Figure 22 below shows the displacement contours around the tunnel cross section after the application of rockbolts and shotcrete in rockclass IV, the maximum displacement occuring before and after the application of supports is 0.00125m and 0.00115m respectively at the side wall of tunnel.

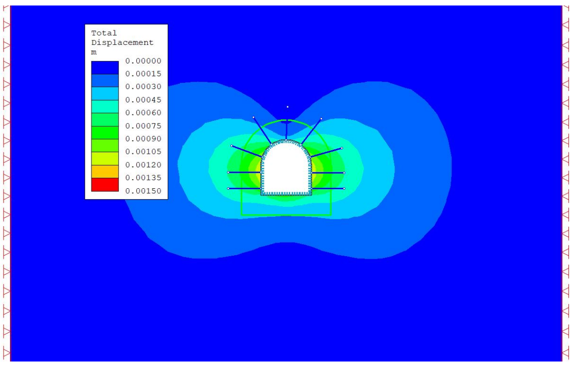

Figure 22: Phase2 model showing displacement contour around the tunnel section in rock class IV.

The number of yielded elements decreased from 473 elements to 431 elements after the application of supports as shown in Figure 23 below.

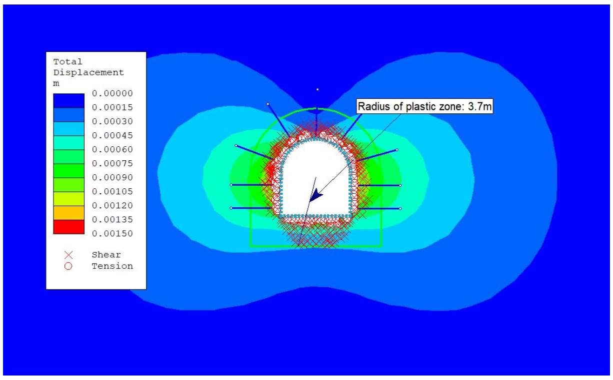

Figure 23: Phase2 model showing radius of plastic zone after applying Support.

5. Conclusions

In this study, the orientation data of the joint sets were collected from the field visit and Kinematic analysis is carried out using dips for the stability analysis of portal. The geometry and stability of underground wedges defined by joint sets were determined by UnWedge. The principal stresses, strength factor, total displacement, yielded elements around the tunnel excavation were determined by Phase 2 software. These models were made based on the assumptions: Elastoplastic behavior model using generalized Hoek– Brown criterion is used to make the models and Tunnel model is 2D considering plane strain problem. From all the result and discussion of that, this project work concludes as following:

- The orientation of the joint sets can be used for cut slope design at the portal to provide maximum stability economically.
- The estimated supports can further be analyzed for the wedge failure using UNWEDGE programming. This must be helpful for the verification of the support that it can withstand possible wedge failure.
- The distribution of principal stresses around the tunnel contour, the maximum displacement at the specific location of the cross-section, the strength factor obtained using Phase2 modelling can be used to evaluate the rock mass condition and tunnel stability.

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