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RESEARCH ARTICLE

EMPIRICAL APPROACH IN GEOTECHNICAL ANALYSIS OF FAILURES OF CUTSLOPES: A CASE STUDY

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Abstract

Cutslope failure of surface powerhouse are analysed with Slope mass rating (SMR) and Limit equilibrium method (LEM). The significance of these parameters is to decipher the probability of failure of slope as well as to determine the factor of safety (FS). Combined use of both parameters helps makes more confident to understand the behaviour of cutslope and its probability to fail during construction stage so that failure in any form could be avoided. The results obtained from the SMR and LEM provide a useful and confident solution while dealing with slopes coherently with the start of construction of civil structure in Himalayan terrain. If cutslopes are properly evaluated/interpreted along with requisite rock support then the time and cost overrun can be minimized.

1. INTRODUCTION

Hydropower projects involves huge capital along with number of impediments like difficult reaches of rugged topography with high mountains, steep gradient of slope, thick vegetal cover, complex geology and thick cover of overburden materials and climate. Managing the stability of slopes has always been a challenging task and posed concern precisely during execution of civil structures in Himalayan terrain. Proper investigation, design, smooth construction is necessary for its stability with time. Proper evaluation of geological investigation leads to evolve effective design which further helps to check any failures as well as cost overrun and delay while implementation of hydropower projects. In the analyses of slope stability of power house cutslope, Slope mass rating (SMR) and Limit equilibrium method (LEM) is used.

SMR is based on Bieniawski rock mass rating RMR_{basic} which depend upon addition of five parameters i) Strength of Rock ii) RQD iii) Spacing of discontinuities iv) Condition of discontinuities and v) water inflow through discontinuities to arrive at different classes of rock.

The (LEM) basically consists in devising an arbitrary mechanism of collapse and the collapse load is found from equilibrium of forces that act at boundaries and do not violate the failure criterion. It is the relationship between the capacity of the system (i.e. strength or resisting force) and the demand on it (stress or disturbing force) is considered. The balance is usually given in terms of factor of safety (FS). The analysis of stability of slopes consisting of well defined joints and bedding planes can be well performed using the (LEM) technique. In this the geology of the rock mass plays an important role. The presence of three or more than three joint sets leads to the formation of wedges. Analyses are undertaken to evolve the factor of safety (FS) of slopes.

2. STABILIZATION OF CUT SLOPES IN HIMALAYAS

Geotechnical analysis of stabilization and failures of powerhouse cutslopes was analysed using Slope mass rating (SMR) and Limit equilibrium method (LEM) of Pārbati Project Stage II. The information about failure and stabilization of cut slope failure were extracted from the papers of Bhatnagar and Das (2013).

3. THE PROJECT

The Parbati Project Stage II (800MW) is located in District Kullu, Himachal Pradesh, India. The Project envisages construction of a 85m high concrete gravity dam on river Parbati, a left bank tributary of Beas river, near village Barsaini in Himachal Pradesh. The water is diverted through a 31.5km long HRT. The surface powerhouse (123m x 47m x 44m) is located on right bank of Sainj river near village Suind, (Fig. 1) housing four generating units of 200 MW each. The water from the powerhouse shall be discharged in river Sainj, another tributary of river Beas, through four small tail race channels. The other components of the powerhouse complex comprises of 124m high, 17m diameter underground surge shaft along with surge galleries of length 225m and 175m at lower and upper level respectively, two 3.5m dia steel lined circular inclined pressure shafts (inclined length 1546m at an angle of 30^0) and four 2.5m dia penstocks.

4. GEOLOGY OF THE POWERHOUSE AREA

The surface powerhouse is located longitudinally at EL ±1330M (Fig.2) on the right bank of Sainj river on a 40m wide multistage riverine terrace. The back slope of the powerhouse raises steeply (45º-50º) upto a height of 175m (EL ±1505M) with curve at the downstream portion.
The rock mass exposed is low grade metamorphic rocks of the Green bed member of Banjar Formation. The main rock types in this area are metabasics, chlorite phyllite/schist and quartz chlorite schist. Surface geological mapping of the proposed location of surface powerhouse has been carried out and the discontinuities observed during geological mapping are tabulated in Table 1.
The foliation joint S1 (Table 1) dips towards upstream direction and at curved portion, foliation joint S1 is almost parallel to the orientation of back slope. Other joint set S2 are valley dipping and S3 and S4 dips inside the hill.

Exploratory drifts:

To explore the geotechnical parameters of the rock mass in the powerhouse area, two drifts were excavated at El ±1350 M and ±1346 M respectively. The drift at El ±1350M, is located in the d/s portion of the powerhouse and is 100m straight with two X-cuts of 20m each in u/s and d/s direction. The other drifts are at El ±1346M is in the u/s portion of the powerhouse. On the basis of geo-mechanical classification, the prevailing rock mass conditions as inferred from drift are tabulated in Table 2.

Exploratory drilling:

One drill hole was drilled for 50m depth at right bank terrace. The bedrock encountered at depth of 29m which fairly corroborates the geophysical surveys. The bedrock comprises of moderately jointed quartz chlorite schist. Core recovery ranged between 40 % to 80% in bedrock and RQD is nil to 35%. Permeability value ranged between 2.4 lugeon and 9.9 lugeon.

Another drill hole of 60m depth was drilled at El. 1681.22m, on slope, along Pressure shaft alignment. The bedrock encountered at depth of 40m.The rock type encountered is medium strong, slightly to moderately weathered, medium to fine grained metabasics with quartz chlorite schist.

Laboratory Rock mechanic test:
The rock mechanic properties of meta-basics were tested in laboratory and the results are tabulated in Table 3.

5. GEOLOGY ENCOUNTERED DURING EXCAVATION

The excavation plan of the powerhouse cut slopes (Fig.2) shows the highest and lowest level is 1432M and 1331M respectively. From upstream to downstream, the entire cutslope was divided in Sections (-) 6 to (+) 36 each separated by 5m. These Sections are represented as -6, -5...... to -1, 0, +1, +2,.....+36 . The portion between Section +8 and Section +24 is straight and in downstream portion between Sections +24 to Section +36, the cut faces are in curved portion.

During excavation the rock mass exposed are metavolcanics with bands (1m to 7m) of chlorite schist. The foliation joint shows wide variations due to local warping. Joint openings of the order of 1-50cm were noticed along the S1 & S2 joints. The straight portion between Section +24m and u/s (+8) has cut sub-perpendicular to the foliation whereas between Section +24 to Section +30, i.e. in the d/s curved portion, the cut faces are sub parallel to the foliation planes. The rock mass condition in the curved portion comprises mainly moderately to highly weathered chlorite schist. At many places 5-20cm thick bands of chlorite schist with clay were also encountered. The seepage conditions were mostly dry to moist. The weathering index was increasing towards the downstream end so much so that it was difficult to decide whether to put this area in overburden or rock. Though the rock mass structures were visible but for civil engineering purposes it was similar to overburden. Nevertheless, the rock conditions in the central portion were comparatively better.

6. POWERHOUSE EXCAVATION AND ROCK SUPPORT/REINFORCEMENT METHODOLOGY:

The excavation of powerhouse cut slope was designed in steps of 15 m with berm width of 4m. The excavation was proposed at EL.1417M with cut line at El. 1423M. The support was 9m long, 36 mm Ø rock anchors at 3m c/c on top row and 6m long 25 mm Ø rock anchors at 3m c/c below that along with wire mesh and shotcrete. During initial stage of rock cutting, no sound rock mass along the cut line were encountered, therefore ,the rock cutting started from higher elevation around El 1450m(sec +4 to -5). Still due to non availability of sound and firm rock, loose boulders were removed and remaining surface was covered with wire mesh and shotcrete with 4m long 25 mm Ø rock anchors.

Due to some villages in nearby area, the excavation of powerhouse cut slope was done by jackhammer drilling near the cutline and by hydraulic drill (TAMROCK & ROCKDRILL (Atlas copco)) in the area beyond 10m from the cut slope line. By jack hammer, usually 32-40 mm dia hole at spacing of 300-500mm and 1.5m to 2.5m depth were carried out whereas by TAMROCK & ROCKDRILL usually 89/64 mm dia holes, spaced at 2 to 3 m and of 3m depth were drilled. After blasting, the exposed area was cleaned with air jet and the area was supported with welded wiremesh & 100mm shotcrete. In few areas where welded wire mesh could not be placed, chain link wire mesh was placed due to the presence of uneven surface.

6.1 Problems encountered during the excavation of the Power House Cutslope:

As the excavation and treatment of powerhouse cut slope was in full swing, failures were experienced. Wide variation in foliation joint in combination with other joint sets caused small failures of the bench at El.1387 and resulted in overhanging of rock mass at EL. 1395M. Concrete cladding /back filling were provided to support the overhanging portion between sec +4 to +8 at El 1387 to 1305 and sec+3 to +1. A wedge failure took place between Sec +19 to +21 between El. 1417 to 1432.The metavolcanics were blocky and traversed by 03 set of joints sets namely S1($062^0/65^0$) and valley dipping joint S2 ($140^0/65^0$) combines to form wedges and hill dipping joint $331^0/80^0$ caused the release of wedge.

Later, the development of 1-50mm wide cracks in the Power House Cut slope was observed. These cracks proved fatal and a slide (1st slide, Fig.3) took place in the service bay area between section +9 to -7 starting from El ±1440 to ±1368 M. This slide later extended upto El ±1480 M. It appears that all the features i.e. wedge failure, valley dipping joints which are vulnerable due to their orientation viz-a viz the cut slope and heavy precipitation culminated to form slip circle failure and caused major slide in the service bay area.

In light of the above slide, the entire slope was inspected in detail. A number of hair-line to few cm wide cracks was noticed at various levels throughout the powerhouse cut slopes. A number of 6-15m long Single/Multi-point borehole extensometers were installed in the powerhouse cut slopes. Apart from monitoring of slope movement by inclinometer, restoration of the slope was carried out.

Again a major jump in the instruments readings and increase in width of the cracks (2cm -10cm) was noticed. After this incident, all the blasting and other activities were suspended in this portion and the excavation work was taken from the upstream side. When the excavation work reached near the desired level in the unit-3 & 4, the cracks started showing widening and the instrumentation reading shot up to alarming values. Subsequently the area of cut slopes in the d/s of Section +18 from El ±1372 (apparently) to Section +36 at El 1440-1450m started sliding. The estimated amount of slided muck was around 30,000 to 35,000 m^3 (Second slide, Fig.4).

The (Third slide, Fig.5) of the powerhouse slope took place in the downstream portion of cutslope. This slide extended up to EL±1495m above unit 3 and 4. The slided muck along with rock chunks with rock anchors covered upto unit 2 portion. The estimated amount of slided muck was 30000-32000 cum. Shotcrete at various berms was also got detached. Rock swelling above berm EL±1346m was also observed. More than 75% of the already treated portion falls down between sections +18 to +32. (Bhatnagar and Das, 2013).
In totality, wedge failure and three slides took place. For analysis, wedge failure named as Location L1, First slide as Location L2 and Second and Third Slide as Location L3 respectively (Fig.2).

6.2 Reasons of failure of the Power House Cut Slope:

Failures in a rock slope often depend on the orientation of the slope and the discontinuities in the rock mass. The main parameters governing this type of failure are the shear strength of the discontinuity. Reasons of multiple failure of the Power House slope are summarized below:-

(i) **Design aspects:** The design of the cut slope was not compatible to the site condition as per geotechnical parameters. This could be due to limited land available due to site constraints and inadequate support provided in the initial stages was also proved.

(ii) **Weak Rock mass:** The powerhouse slope is mainly constituted of medium strong metavolcanics with thick bands of weathered chlorite schist/phyllite.

(iii) **Heavy Precipitation:** The area receives heavy precipitation. The open joints present in the rock mass provided easy path for percolation, resulting in lubrication of joint planes to facilitate failure.

(iv) **Concrete claddings**: Concrete cladding provided on ±1402M berm from section +20 to section +36 and on 1387 M berm from section +24 to section +36, proved to be extra load.

7. EMPIRICAL APPROACH METHOD

The empirical approach method involves identification of causative factors and their influence in inducing instabilities. This method involves well known technique like Rock mass rating (RMR_{basic}), Slope Mass Rating (SMR). (Anbalagan et al 2007).

7.1 Rock mass rating (RMR_{basic})

Rock mass rating **(RMR_{basic})** is estimated according to adding the Bieniawsky rating of the five parameters (Bieniawsky 1989), (Table 4) to arrive at different classes of rock. The rock mass rating **(RMR_{basic})** evaluated indicates only the quality of rock i.e Class I, Class II to Class V, very good to very poor rock without taking consideration of joint orientation.
The rating arrived after adjustment of discontinuity orientation for slopes: very favourable 0, favourable -5, fair -25, unfavourable -50, and very unfavourable -60 does not have any definition and expression about failure of slope.

7.2 Limit equilibrium method

The stability of slopes on an inclined plane is explained by limit equilibrium i.e the condition at which the forces tending to induce sliding are exactly balanced by those resisting sliding. Hoek and Bray (1981), Anbalagan et al (2007). The factor of safety (FS) is an index/ratio required to compare the stability of slopes under limit equilibrium.

$$\text{Factor of Safety (FS)} = \frac{\text{Total resisting force along plane of separation}}{\text{Total mobilising force available to induce failure}}$$

According to Hoek and Bray (1981), the slope for the saturated and dry slope will be stable if the FS should be less than 2.0 considering variable slope angle. The (FS) for the plane failure is calculated by the equation given below:

$$FS = \frac{cA + (W.\cos \Psi p - U - V.\sin \Psi p) \tan Ø}{W.\sin \Psi p + V.\cos \Psi p} \quad (1)$$

Where c is cohesion along failure plane, Ψp is the dip amount of failure plane; A is the area of the failure plane, W is weight of sliding block, U is the uplift force due to water pressure on the sliding surface and V indicates force due to water pressure in the tension crack. Similarly the (FS) calculated for the wedge failure is as follows:

$$\mathrm{FS} = \frac{3}{\gamma_r H}(c_A X + c_B Y) + \left(A - \frac{\gamma_w}{2\gamma_r}X\right)\tan\phi_A + \left(B - \frac{\gamma_w}{2\gamma_r}Y\right)\tan\phi_B$$

$$\left.\begin{aligned} X &= \frac{\sin\theta_{24}}{\sin\theta_{45}\cos\theta_{2.na}} \\ Y &= \frac{\sin\theta_{13}}{\sin\theta_{35}\cos\theta_{1.nb}} \\ A &= \frac{\cos\psi_a - \cos\psi_b\cos\theta_{na.nb}}{\sin\psi_5\sin^2\theta_{na.nb}} \\ B &= \frac{\cos\psi_b - \cos\psi_a\cos\theta_{na.nb}}{\sin\psi_5\sin^2\theta_{na.nb}} \end{aligned}\right\} \quad (2)$$

Where C_A and C_B are the cohesive strength of plane A and B, $Ø_A$ and $Ø_B$ is the angle of friction of Plane A and B. γ is unit weight of rock, γ_w is unit weight of water. H is the total height of wedge. X, Y, A and B are dimension less factor depend upon the wedge geometry calculated with the help of stereoplot. The values of θ's for calculating X Y A and B are obtained from the stereoplot (Hoek and Bray (1981), Anbalagan et al (2007). Ψ_a and Ψ_b are the dips of Plane A and B and Ψ_5 is the plunge of line of intersection.

7.3 Slope Mass Rating (SMR)

The Slope mass rating (SMR) proposed by Romana et al, (1993,2003), Singh and Goel,(1999), Umrao et al.(2011) is explained in Fig.6 and its parameters are tabulated in Table 5, 6 & 7.

SMR is obtained from summation of RMR_{basic} and the adjustment factor F1, F2, F3 and F4 depending upon joint slope relationship. It is represented by

$$SMR = RMR_{basic} + (F_1 \times F_2 \times F_3) + F_4 \quad \text{Where}$$

1. F_1 depends upon parallelism between joints and slope face ranges between 1.00 to 0.15. $F_1 = (1\text{-}\sin A)^2$ where A is the angle between dip direction of slope face and joint in case of planar and wedge failure. In wedge failure A is the angle between plunge direction of line of intersection formed by discontinuities and slope dip direction.
2. F_2 is joint dip angle in the planar mode of failure. It is a probability of joint shear strength. Its value range between 1.00 to 0.15. $F_2 = \tan^2 \beta$ where β is the dip angle of joint for planar failure. In case of wedge failure, B is the dip angle of plunge of line of intersection.
3. F_3 indicates the relationship between the slope face and joint dip. In planar failure, it refers the probability of joints daylighted in the slope face. In case of wedge failure, F_3 indicate the relationship between the slope face and dip of plunge of intersection of two joints.
4. F_4 has empirical values depending upon the method of excavation.

Table 7 indicates different classes of SMR rating indicating the stability of rock mass and its probability of failures. During evaluation of probability of slope to fail, the reverse of the value is used i.e 0.9 indicate 90% stable and probability of failure is 10%.0.6 indicate 60% stable and probability of failure is 40%. 0.4 indicates 40% stable and probability of failure is 60%.Similarly 0 indicates 100% unstable and probability of failure is 100%**.**

8. JOINT PARAMETERS AND EVALUATION OF RMR

Continuous geological mapping of the powerhouse cutslope was carried out after the failure of slopes as well as with excavation. Face log of each berm prepared measuring the discontinuities and other geotechnical parameters present in the rock. The geotechnical parameters for the location L1, L2 and L3 are collected and tabulated at Table 8 for evaluation of RMR_{basic} given in Table 9.

9. ANALYSIS OF FAILURES

9.1 Wedge Analysis along section 19-21 (Location L1)

The Stereographic projection of the discontinuities using Dips shows formation of wedge that is dipping out of the slope face (Fig.7). The line of intersection formed by S1 and S2 is daylight on slope face on which the formed wedge had been slide down. The wedge failure model was developed using S- Wedge. The values of θ's are obtained from the stereoplot (Fig.7) (Hoek and Bray (1981), Anbalagan et al (2007). The FS calculated is tabulated using Eqn.(2) Table10.

9.2 First Slide (Location L2)

The Stereographic projection of the discontinuities using Dips and the plane failure model was developed using S-Wedge (Fig.8), shows that there would be planar failure along the foliation plane $110^0/75^0$. The wedge failure is not likely to occur. The calculated FS using limiting equilibrium method is 0.527 under maximum cohesive strength of 305KPa but it was not likely the case as the cohesion between two joints was much less than 100 KPa due to soil filling between the joints and the FS calculated is 0.290 and tabulated using Eqn. (1) Table 11.

9.3 Second and Third Slides (Location L3)

S1: $067^0/50^0$, S2: $150^0/60^0$, Slope face: $110^0/70^0$, Intersection S1^S2: $104^0/58^0$

The Stereographic projection of the discontinuities using Dips and the plane failure model was developed using S-Wedge (Fig.9) shows that there would be planar failure along the foliation plane $067^0/50^0$. The wedge failure is not likely to occur. The calculated FS using limiting equilibrium method is 1.419 under maximum cohesive strength of 305KPa but as the cohesive strength decrease due to opening of joints and at 160KPa the block is just supported by the resistive force with FS as 1.00. If the cohesion between the foliation planes gets reduced to further, the block fails naturally. The FS has been calculated using Eqn. (1) is given in Table 12.

10. RESULT AND DISCUSSION:

The geotechnical parameters for all the three location L1, L2 & L3 are tabulated in Table 8 and evaluation of RMR_{basic} is given in Table 9. Slope mass (SMR) rating is evaluated using Table 5 and 6 and tabulated in Table 13 which were further correlated with Table 7 which finally reflects the probability of failures. The SMR rating (Table 13) for Locations L1, L2 and L3 are -15, -8 and -24 respectively which are below Class V rating (0-20) indicates big planar or soil like failure and probability of failure is 100%. All the three locations are completely unstable and anchored total wall excavation is required (Romana1985, Anbalagan et al. 2007).

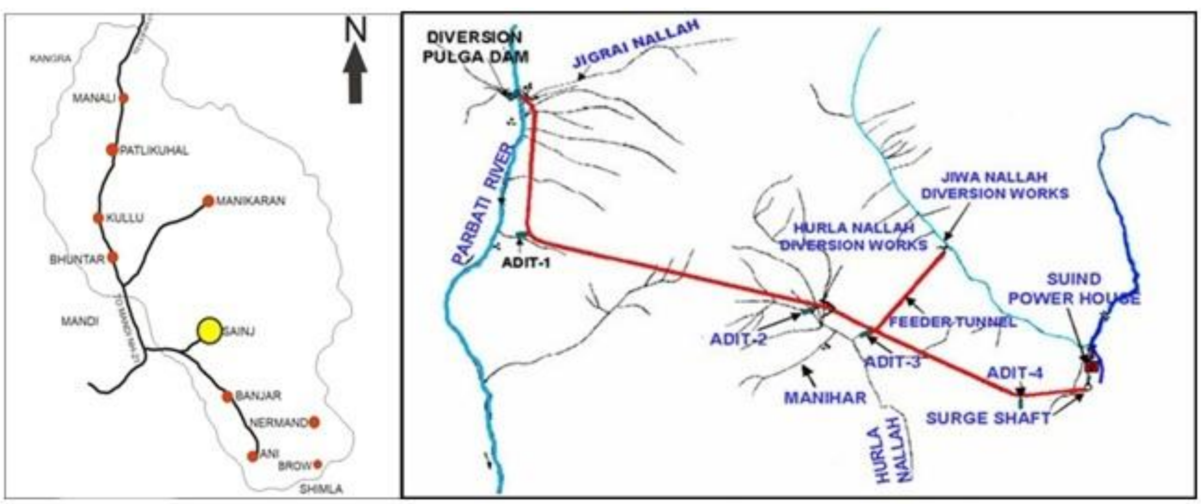


Fig. 1. Location Map and Layout Plan of Parbati Project, Stage-II

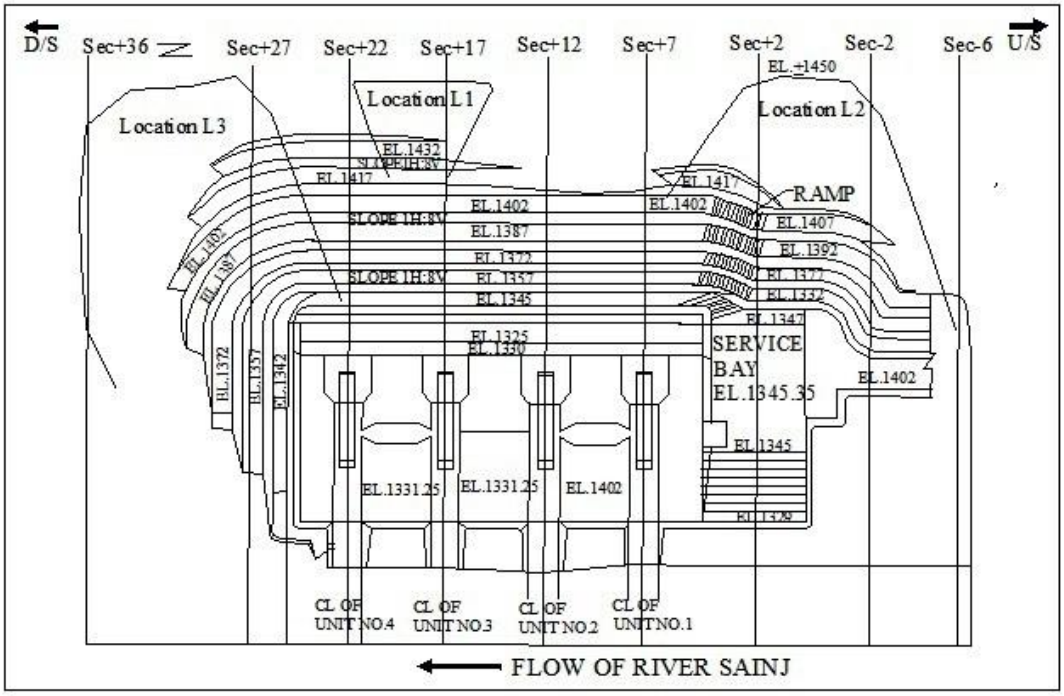


Fig.2 Powerhouse Excavation Plan and Sketch showing Location of Failures L1, L2 and L3

Fig.3 First Slide

Fig.4 Second Slide

Fig.5 Third Slide

Table 1 Discontinuity details in Powerhouse Area

Average orientation	Spacing (mm)	Persistence (m)	Aperture filling	Condition	Remarks
S1- 070 ^{o}/50^{o}	10 - 50	5 - 6	Tight to sheared	Smooth undulatory	Variation in orientation was noticed
S2- 150^{o}/60 ^{o}	50 – 2000	5	Tight to open 20 to 50 mm	Rough undulatory	Valley dipping joint governing the slope
S3- 020 ^{o}/40^{o}	500- 1000	6	Tight to open(2mm)	Smooth planar	
S4- 285 ^{o}/70^{o}	400-1000	5 - 6	Tight	Smooth planar	

Table 2 Geomechanical Classification of Rock Mass from drift

Drift	Length	Class II	ClassIII	Class IV	Class V
EL.1346M	50 m	16%	62%	18%	4%
EL.1350M	100m main drift with 20m left and right side drift	13.5%	35%	50%	1.5%

Table 3 Rock mechanic Properties of Rockmass constituting the slope

Parameters	Metabasics	Chlorite schist-phyllite
Friction Angle, (^{o})	33^o	24^o
Cohesion, MPa	3.0	0.5

Table 4 Rock mass Classification (after Bieniawsky, 1989)

Parameter		Range of Values						
UCS	Values	>250 MPa	100-200 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating	15	12	7	4	2	1	0
RQD	Values	90-100%	75-90%	50-45%	25-50%	25%		
	Rating	20	17	13	8	3		
Joint Spacing	Values	>2m	0.6-2.0m	200-600mm	60-200mm	<60mm		
	Rating	20	15	10	8	5		
Joint Condition	Values	Very rough surfaces No continuous No separation Un weathered wall	Slightly rough surfaces Separation <1mm Slightly weathered wall	Slightly rough surfaces Seperation <1mm Highly weathered wall	Slickensided surfaces or Gouge<5mm thick or separation 1-5mm	Soft Gouge > 5mm or Separation >5m Continuous		
	Rating	30	25	25	25	0		
Ground water	Values	Completely dry 15	Damp 10	Wet 7	Dripping 4	Flowing 0		
RMR_{basic} Rating		100-81	61-80	41-60	21-40	<21		
Class number		I	II	III	IV	V		
Description		Very good	Good	Fair	Poor	Very Poor		

Table 5 Adjustment rating of joints for SMR (Romana 1993, 2003)

Case		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
P	$I\alpha_J-\alpha_S I$	$>30^0$	$30-20^0$	$20-10^0$	$10-5^0$	5^0
T	$I(\alpha_J-\alpha_S)-180^0 I$					
P/T	F_1	0.15	0.40	0.70	0.85	1.0
P	β_J	$<20^0$	$20-30^0$	$30-35^0$	$35-45^0$	45^0

P	F_2	0.15	0.40	0.70	0.85	1.0
T	F_2	1	1	1	1	1
P	β_J-β_S	$>10^0$	$10-0^0$	0^0	0^0 to 10^0	$<-10^0$
T	β_J-β_S	$<110^0$	$110-120^0$	$>120^0$	-	-
P/T	F_3	0	-6	-25	-50	-60
α_j: Joint Dip direction β_J : Joint Dip amount α_s: Slope Dip direction β_s : Slope Dip amount, P= Planar T= Toppling Failure						

Table 6 Adjustment rating for methods of Excavation of Slopes

Method	Natural Slope	Presplitting	Smooth Blasting	Blasting or mechanical	Deficient Blasting
F_4	+15	+10	+8	0	8

Table 7 Stability classes as per SMR Values (Romana.1993, 2003)

Class	I	II	III	IV	V
SMR	81-100	61-80	41-60	21-40	0-20
Rockmass description	Very good	Good	Normal	Bad	Very bad
Stability	Completely Stable	Stable	Partially Stable	Unstable	Completely Unstable
Failures	None	Some block failure	Planar along some joints or many wedge failure	Planar or big wedge failure	Big planar or soil like or circular
Probability of Failures	0.9	0.6	0.4	0.2	0

Table 8 Geotechnical parameters of rockmass in cut slope.

Parameter	Location (L1)	Location (L2)	Location (L3)
Rock Type	Metabasics with chlorite schist		
Strength	R3 (Medium Strong)	R3 (Medium Strong)	R2-R3 (Medium Strong to weak)
RQD	<25%	<25%	25-50%
Joint spacing	<60&60-200mm	<60&60-200mm	<60mm
Joint roughness	Smooth Planar	Smooth Planar	Smooth Planar
Joint Separation	Tight to 5 mm	1 to 50mm	1-5mm
Joint Persistence	3-10m	3-10m	3-10m
Joint infilling	Soft <2mm	Soft <2mm	Soft <2mm
weathering	Slightly weathered	Moderately weathered	Highly weathered
Ground water condition	Damp	Dry	Damp

Table 9 Rock Mass rating (RMR_{basic}) of Locations L1, L2 and L3

Sr. No	Parameter	Location (L 1)	Location (L2)	Location (L3)
1	Strength Rating	4	4	2
2	RQD Rating	3	8	3
3	Joint spacing Rating	8	8	5
4	Joint condition Rating	11(2+1+1+2+5)	8(2+0+1+2+3)	7(2+1+1+2+1)
5	Ground water condition	10	15	10
	RMR_{basic}	**36**	**43**	**27**
	Class	**IV(Poor)**	**III (Fair)**	**IV(Poor)**

Table 10 Wedge failure back analysis along section 19-21

Slope Face Lower	Slope Face Upper	S1	S2	Intersection line	Unit weight of rock	FS	Comment
$110^0/70^0$	$110^0/43^0$	$062^0/65^0$	$140^0/65^0$	$101^0/59^0$	23KN/cum	1.421	The block was partially stable without support

Table 11 Plane failure back analysis

Slope Face	Joint Plane	Phi	C	Unit weight of rock	FS	Comment
$118^0/80^0$	$110^0/75^0$	33^0	100KPa	25KN/cum	0.290	The block was unstable without support

Table 12 Plane failure back analysis

Slope Face	Joint Plane	Phi	C	Unit weight of rock	FS	Comment
$055^0/72^0$	$067^0/50^0$	33^0	305KPa	25KN/cum	1.419	The block was partially stable without support

Table 13 Slope Mass rating (SMR)

Lo. No	**RMR** basic	**Type of Failure**	α_j **or** α_i	α_s	α_j **or** $\alpha_i - \alpha_s$	F_1	β_J or β_i	F_2	β_s	$\{\beta_J$ or $\beta_i\}-\beta_s$	F_3	F_4	**SMR rating**
L1	36	Wedge	$N101^0$	$N110^0$	9^0	0.85	59^0	1	70^0	-11^0	-60^0	0	-15
L2	43	Planar	$N110^0$	$N118^0$	8^0	0.85	75^0	1	80^0	-5^0	-60^0	0	-8
L3	27	Planar	$N104^0$	$N110^0$	6^0	0.85	58^0	1	70^0	-12^0	-60^0	0	-24

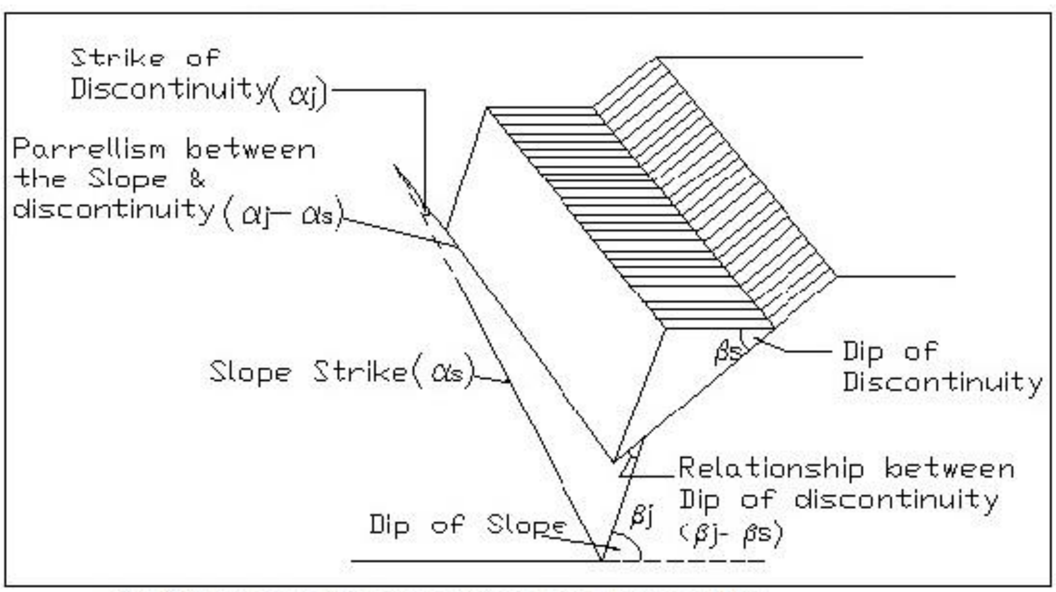


Fig.6 Schematic diagram of Slope Mass Rating(SMR)

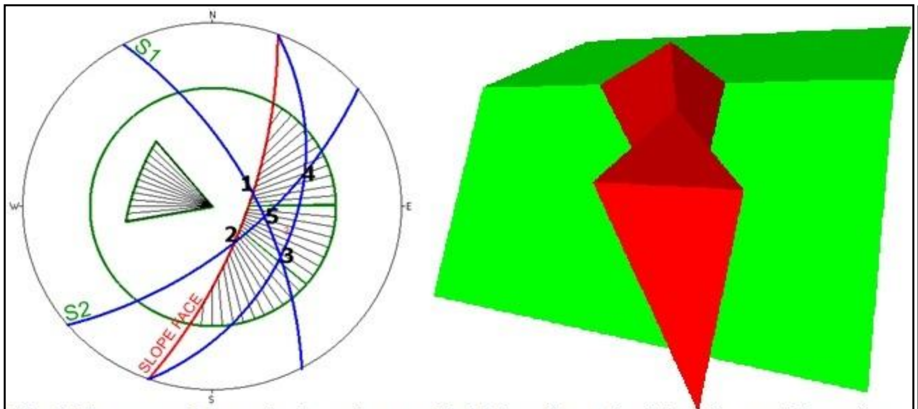


Fig.7 Stereographic projection along with Major discontinuities along with wedge failure model alonf Sec 19-21

$\theta_{nanb} = 80^0$	$X = 4.69$
$\theta_{24} = 48^0$	$Y = 5.42$
$\theta_{13} = 44^0$	$A = 0.42$
$\theta_{35} = 22^0$	$B = 0.42$
$\theta_{45} = 25^0$	$\psi_a = 65^0$
$\theta_{2.na} = 68^0$	$\psi_b = 65^0$
$\theta_{1.nb} = 70^0$	$C_A = C_B = C = 24KPa$
	$\phi_A = \phi_B = \phi = 24^0$

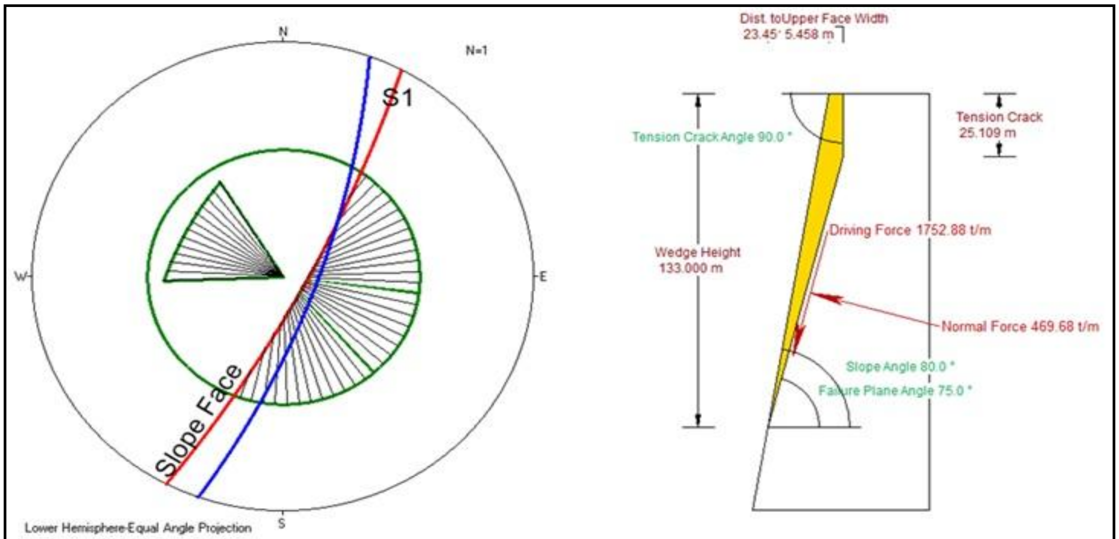
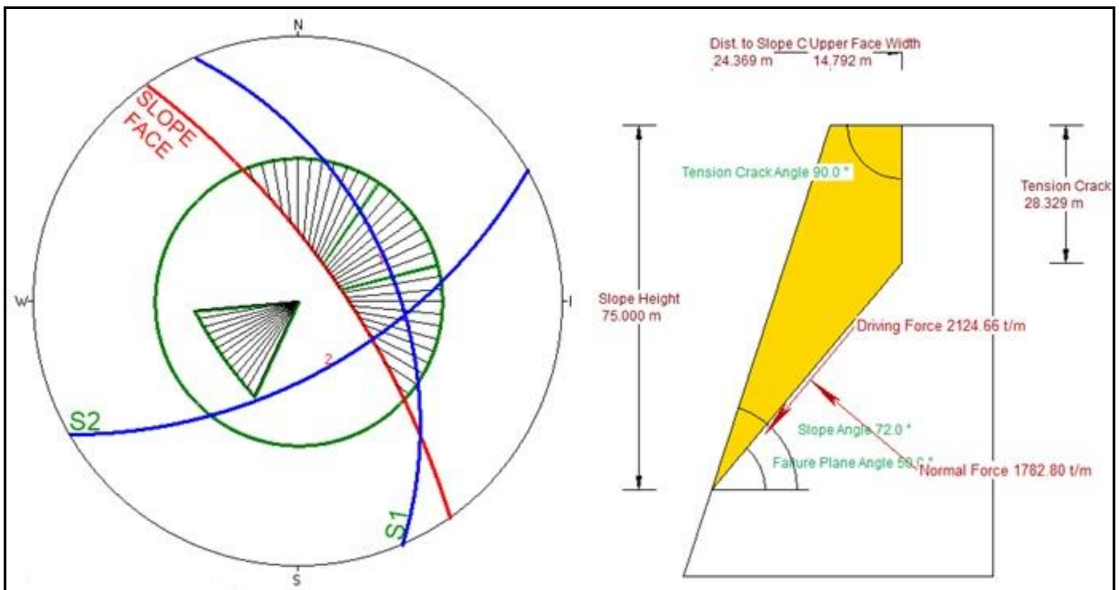


Fig.8 Stereographic projection of the Major Discontinuities and a plane failure model



Fig,9 Stereographic projection of Major discontinuities and a plane failure model (Second and Third Slide)

Fig.10 Google earth image of Stable treated cut slope & construction of surface powerhouse in progress.

11. REDESIGNED ROCK SUPPORT FOR STABILIZING THE POWERHOUSE CUTSLOPE AFTER FAILURES

Keeping in view of three slides which took place at various section and elevations, the rock support for stabilizing the cutslope was modified as follows:

(i) Rock slope at different berms was reconsidered and more land above El ±1495M was acquired for maintaining proper slope. The excavation of the backslope started from EL ±1510M.A cross drain of 1m width x 0.5m height was constructed at El 1520M to drain out the surface runoff. The slope was cut down to the level of ±1330M. Between EL 1492M to 1402m a 1H: 2.5V slope was maintained whereas between El 1372M to 1330M the slope was kept as 1H: 3.5V. Thus a vertical slope of about 200m height with intermediate benches at approx 15m interval was to be stabilized, which was a challenging task.

(ii) Berms of 5-6m width were provided, covered with shotcrete to prevent seepage of rainy water in benches. In addition vertical drains were provided to drain water during monsoon/heavy precipitation.

(iii) Extensive grouting of slope by drilling 10 m long, 76mm dia (NX) drill holes at distance of 10m at each berm/benches.

(iv) Cable anchors of length 35m (fixed and free length of 9m and 26m) were provided in three rows at spacing of 5m at each bench. A total 842 cable anchor was installed to stabilization.

(v) Rock anchors of 12m length were provided in staggered manner. Shotcrete with wire mesh (100 to150mm) were also provided to stabilize the slope.

(vi) Drainage holes 76mm Ø, 12m long on downward direction were provided on each berm. In addition Pressure relief hole/Pre grout holes were also provided on each berm.

(vii) In addition, 4 no. of drifts of 40m length with 20m long x cuts at El.1432 and 40 m long drift at EL.1417m were excavated. The drift along with crosscuts were reinforced with steel and concrete up to spring level and between spring level to crown with concrete mass to bear rock load/sharing of load.

(viii) Fig. 10 shows Google earth image of Stable treated powerhouse cut slope after failure and construction of surface powerhouse is in progress

(ix) Extensive instrumentation programme for performance monitoring the stability behavior of cut slope by installing an array of Instruments. (Table.14). All the instruments installed are being continuously monitored from the time since installation and are working smoothly and none of them is giving any abrupt results.

Table 14 Instruments installed on Powerhouse cut slope

Instruments	Quantity	Remarks
MPBX (Multiple point borehole extensometer)	30Nos (Three Point and four point), maximum Length ranging from 12m to 40m	Working smoothly and not giving any adrupt results
Load Cell (150 ton and 25 ton)	25 Nos	-do-
Prism Target	8 Nos	-do-
Peizometer	5 Nos	-do-
Inclinometer	3 Nos	-do-
Tiltmeter	5 Nos	-do-

12. CONCLUSION:

The results of SMR rating shows probability of failure of cut slopes are 100% and according to Limit equilibrium method (LEM), the FS are 1.421, 0.290 and 1.419 respectively in dry conditions. The FS will reduce further due to presence to schist bands/clay as well as during rainfall periods. It is interpreted that the combined use of both parameters makes more confident to understand the behaviour of cutslope and its probability to fail during construction stage. Henceforth these techniques should be adopted during interpretation of cut slope in himalayan terrain so that inadequate rock support could be evaluated and time and cost overrun could be minimized.

13. DISCLAIMER AND ACKNOWLEDGEMENT

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