Stability of cut slopes on either abutment of Koteshwar dam, Uttarakhand

Bahuguna, Harish
Head, Environment and Geology Cell
UJVNL, Dehradun, India

Abstract

The Koteshwar dam site located in Tehri Garhwal district of Uttarakhand and it is approximately 22 Km downstream of the famous Tehri dam. The project was commissioned in 2011 and presently it is generating 400MW of hydropower. Components of the project consists of a 97.50m high concrete gravity dam consisting of 14 blocks, a 575m long and 8m diameter diversion tunnel on the left bank having a design discharge of 670 Cumecs, four vertical penstock shafts, four lower horizontal penstocks, a surface powerhouse, a main access tunnel and a stilling basin at the toe of the dam. The project area lies within the Main Himalayan Belt (MHB), in the midlands of Lesser Himalaya which is bounded in the north by a regional tectonic discontinuity i.e. main Central Thrust (MCT) and in the south by another tectonic discontinuity i.e. Main Boundary Thrust (MBT). The project site is located in Zone IV of the Seismic Zoning Map of India wherein seismic shaking may result in damage reaching intensity rating of VIII on MM scale.

The project area exposes an uninterrupted sequence of folded meta-sedimentary rocks of Chandpur phyllites (Pt 3 Proterozoic-III) which have variable proportions of argillaceous and arenaceous constituents. The change in the attitude of bedding on either bank clearly indicates an antiformal closure at the dam site. The physical and geo-mechanical properties the rock masses were determined by employing geophysical methods and performing geo-mechanical tests at the site and laboratory. The lab testing of the samples and determination of geomechanical properties was aimed at deciphering the role of the compositional variation in the rock masses on these properties. Slope stability analysis for either abutment helped in deciding the stripping limit on the abutment slopes, in designing the cut angles of the slopes, and in devising their support measures.

1. Introduction:

The Koteshwar dam site located in Devprayag tehsil of Tehri Garhwal district of Uttarakhand and it is approximately 22 Km downstream of the famous Tehri dam. The dam site is located at the location of erstwhile Pindars village and it is well connected by a metal road with the district headquarter i.e. New Tehri.

The natural valley profile at the site of construction of dam and its appurtenances is altered with the progressive excavation for reaching the designed foundation level for different structures. The fundamental rule followed during the excavation is that it should be started from the top level of designed slope otherwise starting the excavation from lower level would create unstable overhangs which is an undesirable condition both for the safety of the work force during construction and also for the stability of the foundation.

However, even before making the first cut on the up slope the stability of the slopes vis-a- vis the physical and geo-mechanical characters of the rock masses is analysed in detail and the design of cut slopes with support measures is devised.

2. Regional Geological Framework:

The Koteshwar dam project area lies within the MHB, in the midlands of Lesser Himalaya. A generalized stratigraphy of the Main Himalayan Belt in Garhwal-Kumaun region indicates that the rocks of Lesser Himalaya exposed around Koteshwar dam area are divisible into two major tectono-stratigraphic units- i.e. i) the Krol Super Group (Proterozoic III to Eocene) and ii) the Garhwal Group (Proterozoic II). The rocks of the Garhwal Group directly come in contact with the rocks of relatively younger Krol Super Group, which also includes Jaunsar Group of rocks (Chandpur phyllites and Nagthat quartzites) along a major high angle reverse fault known as Srinagar Thrust. In between these two major groups of rocks, a narrow linear wedge exposes the rocks of Shimla Group (Proterozoic-III), at few locales. The project site is located in Zone IV of the Seismic Zoning Map of India.

2.1 Site Geology:

The Koteshwar dam site exposes an uninterrupted sequence of folded meta-sedimentary rocks of Chandpur phyllites (Pt 3 Proterozoic-III) which have variable proportions of argillaceous and arenaceous constituents. Based on the rhythmicity of intercalated bands of arenaceous and argillaceous material and varied degree of tectonic effects in them, the phyllites at the dam site can be classified into mainly four lithological variants.

i. Phyllitic quartzites (PQ)
ii. Phyllites Thinly Bedded (PT)
iii. Quartzitic phyllite (QP)
iv. Sheared/schistose phyllite (SP)

The phyllitic quartzites (PQ) and phyllite thinly bedded (PT) are rich in quartzitic (arenaceous) content with occasional micaceous content and they are somewhat coarser in grain size. The rhythmicity of compositional bands is characteristic of them and the rhythmic bands are more than 10cm apart in PQ rocks whereas, they are closely spaced (0.5cm to 10cm) in PQT rocks. Quartzitic Phyllite is more argillaceous, fine grained and dark colored. It is characterized by profuse silicification along joint planes. Bedding is obscured but foliation planes are well developed and easily discernible. Sheared Phyllite is not categorized purely on lithological basis but has been recognized as the sheared and tectonised variant of PQ, PQT and QP rocks and they are developed mostly in the vicinity of major shear zones.

2.2 Structural Features:

During the course of geological mapping it has been noticed that the bedding surface dips steeply towards NNW quadrant on either bank at higher elevations i.e. on left bank above El 590m and on the right bank above El 630m (at dam axis), however the right bank the same dip azimuth is recorded at El 580m towards the downstream 120m (from the dam axis). On the left bank below El 590m and on the right bank also below El 580m the dip of bedding is towards SE quadrant. It thus clearly indicates an antiformal closure at the

dam site. The foliation at the project site is uniform on either abutment and at all levels with an average strike of N30^{0}W-S30^{0}E and dip of 25^0 to 35^0 in the N.E. direction.

Table 1
Details of Joint Sets at Koteshwar dam Site

Joint set	Orientation		Spacing (cm)	Continuity (m)	Roughness	Opening	Remark
	Dip Amount	Dip Azimuth					
J-1	55^0-80^0	N330^0-010^0 (Above El± 590m)	5-150 cm	0.50 - 5m	Smooth	Tight	Primary bedding (S_0) joint
	45^0-80^0	N140^0-170^0 (Below El± 590m)					
J-2	25^0-35^0	N030^0-080^0	0.5-50cm	1 - 4m	Smooth Planar	Tight	Foliation (S_1) joint
J-3	50^0-80^0	N050^0-085^0	2-100cm	0.30m-1m broken +5m	Moderately smooth planar	2-3cm	Filled with silica vein
J-4	20^0-65^0	N220^0-260^0	5-100cm	0.50m-2m broken +2m	Moderately smooth planar some are rough Undulatory	Tight	High frequency joint, forming wedges with bedding and foliation joints
J-6	45^0-70^0	N280^0-290^0	30-100cm	2-8m broken +2m	Rough Undulatory	Tight	Random joint

3. Assessment of Stability of Abutment Slopes:

The assessment of the designed cut slopes above the dam, stilling basin, powerhouse and other structures is of paramount importance as it helps in finalizing not only the design of the cut angle, height of the individual slope and in deciding the width of the bench in between but also in evolving the suitable and appropriate support measures for them. The stability of the cut slopes located above the important structures is ensured keeping in view the operational phase of the hydropower projects because any problem of instability on the cut slopes arising in the post construction phase has serious implications as it has direct bearing on the generation losses.

The detailed geological mapping of the cut slopes was carried out for the most important sections for the Koteshwar dam project located between dam axis and d/s 150m as they covered most of the important structures like dam, stilling basin, power intakes, penstock shafts, main access tunnel, power house and tail race channel.

In order to make the analysis easy and simple to understand the slope stretches have primarily been categorized under two viz. i) slopes above dam top i.e. El±618.50m and ii) slopes between dam top and river bed i.e. El±533m. The stability analysis of the slopes was carried out by following the different approaches as given under:

i. Detailed Geological Mapping and Geotechnical assessment
ii. Observing Geological Profiles
iii. Performing Kinematic Checks
iv. Determining and working out the basic rock mass characters and parameters
v. Two and Three Dimensional Analysis

3.1 Geological Profiling:

Geological profiles were observed on 1:500 scale at different sections at 30m interval from u/s 30m to d/s 150m. These profiles formed the basis for deciding the height, angle and width of intervening benches in the slopes on either bank. The important profiles have been briefly discussed below.

3.2 Section at Dam Axis:

The general slope profile on the left abutment was moderate to gentle from El. ±700m to El. ±570m and it was sub vertical between El. ±570m and river bed El. ±533m (Figure 1). The upper part of the slope was controlled by foliation joint whereas the lower steeper slope was controlled by steeper foliation joint and the bedding joint dipping at 55^0 - 75^0 towards N150^0 - N160^0. The slope profile above El. ±630m was occupied by overburden which was largely defined by the slope wash material. The slope wash material, understandably, had low values of cohesion and friction and at times it behaves like cohesion less material. The average lateral depths of intensive weathering, fresh rock and de-stressing were at 13m, 17m and 30m respectively. The excavation required removal of overburden mass and rock for which the angle of cut and height of the slope has to be designed differently.

The profile on the right abutment slope was moderate to gentle between the river level i.e. El. ±533m and El. ±650m and in the rocky part it was controlled by the joints dipping 45^0-75^0/N210^0-270^0 and 45^0-62^0/N280^0-320^0. The slope above El. ±650m becomes gentler and was occupied by river borne material (RBM) and slope wash material. Predominantly PQ with bands of PT was exposed along the dam axis which was traversed by five prominent set of joints including the bedding and foliation.

The joints dipping towards NNW and WSW were having unfavorable orientations as they were dipping towards the valley side and are making unstable structural wedges with the bedding and foliation joints. The excavation on the left abutment was suggested from El±670m in order to avoid unstable negative slope being created just above the dam top at El±618.50m and it will also take care of the overburden material in the up slope.

On the right abutment there was huge thickness (50-60m) of river borne materials (RBM), which created a problem in deciding the excavation line *vis a vis* stability of the structure. It was mandatory to remove the entire RBM above the top of the dam body and this required deep excavation (up to 50m at setting out axis) on the right abutment. Although the dam top is at El. ±618.50m but the existence of RBM and slope wash materials on the up slope necessitated gentle cut slopes. Considering this limitation, excavation was suggested right from El.±670m.

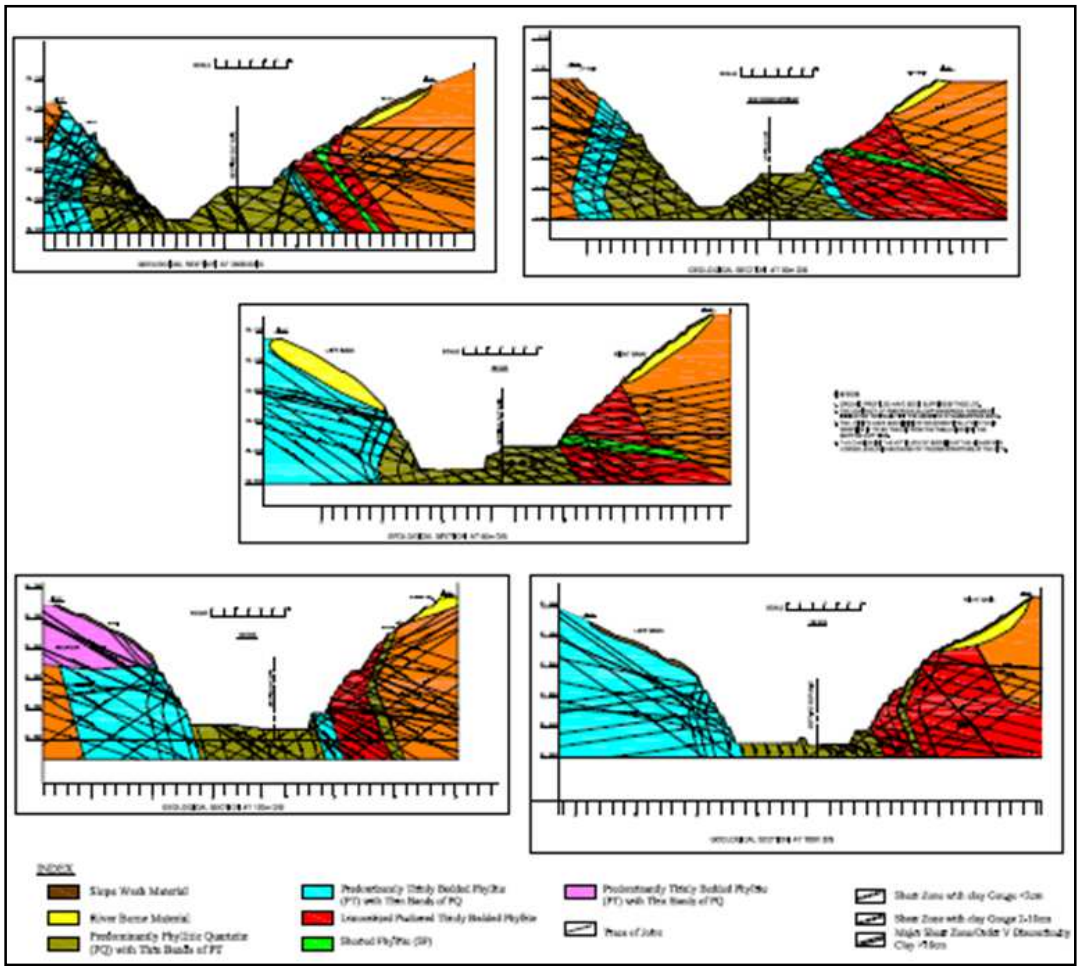

Figure 1 Geological Sections of Koteshwar Dam

3.3 Section downstream 30m:

The profile on the left bank at this section was moderate between EL. ±570m to El. ±670m with a convex hump at EL. ±630m (Figure 1). The slope between EL. ±575m to El. ±550m was steep slope (75^0) which was controlled by the bedding joint after which the slope was again moderate up to the river bed level. The slope above EL. ±630m was occupied by overburden mass/slope wash material. The average lateral depths of intensive weathering, distressed zone and fresh rock are 10 m, 15 m and 30 m

respectively. The top level of dam in this section is at EL. ±598m, therefore the stripping/excavation has been suggested from EL. ±620m. The excavation line was nearly parallel to joint set J2 and it will take care of the convex hump at El. ±630m. The average depth of excavation in this section was 12m.

3.4 Section downstream 60m:

The slope on the left bank above El. ±620m was occupied by slope wash/overburden material whereas the profile below was nearly uniform with a minor convexity between EL. ±585m and EL. ±570m. Major part of the slope was controlled by bedding joint set and some part by foliation joint. Both the bedding and foliation joints were considered unfavorable for the stability of the slopes and intersection of these two brought down huge unstable blocks during the excavation. The average lateral depths of intensive weathering, fresh rock and distressing are at 10m, 16m and 28m respectively. The average depth of excavation along this section was around 15m. Since the top of dam body in this section is at EL. ±570m the excavation was suggested from El. ±592m so as to stabilize the entire slope.

3.5 Section downstream 90m:

Ground profile on this section on the left bank was gentler up to El. ±620m, after which the slope was convex. The slope above EL. ±630m was covered by slump mass/slope wash/overburden material which continued towards the downstream chainages at lower levels (Figure 1). The slope above EL. ±586m was controlled by foliation joint whereas the slope below was controlled by bedding joint. The limits of intensive weathering, fresh rock and distressing are at 12m, 12.5m and 28m respectively and the average depth of excavation was around 12m. The top of dam body in this section is at El±570m whereas the excavation was suggested from EL. ±590m so as to stabilize the entire slope above the dam body by achieving the desired cut slope.

3.6 Section downstream 120m:

Ground profile in the section on the left bank indicated break in slope at E1±620m and EL.±553m and the entire slope was controlled by bedding joint particularly at lower levels (Figure 1). The slope between EL. ±620m and EL. ±650m was occupied by slump mass/slope wash/overburden material (Figure 1). The limits of intensive weathering fresh rock and de-stressing are at 7m, 12.5m and 30m respectively.

Since this section was d/s of the dam body no excavation surface has been suggested, however, fresh rock line has been indicated for deciding the stable cut slope.

On the right bank the profile showed hummocky topography on this section and the slope was being controlled by joint set J3. The rock and RBM contract was at El±568m and above El±650m there was slope wash material (Figure 1).

The thickness of RBM was anticipated to be in the range of 35m- 40m. Intersection of foliation joint and the joint dipping towards WSW were making perfect rhombic wedges, and hence wedge failures were anticipated during excavation.

The limits of intensive weathering, fresh rock and de-stressing were suggested. Since the designed foundation of the power house was at El±515m it obviously meant removal of the RBM and the foundation was in the fresh rock.

3.7 Section Downstream 150m:

Profile on this section on the left bank above EL. ±605m was occupied by overburden mass/slope wash material and it also indicated a convex hump between EL. ±595m and EL. ±565m (Figure 1). Above EL. ±592m the rock exhibits W_2 grade of weathering.

Wedge failures due to intersection of bedding and foliation joints were anticipated during the excavation for making approach roads. Fresh rock line was indicated which on an average was 17m deep, for designing the stable cut slope.

4. Outcomes of Geological Profiling:

The profiling helped in deciphering the topographic and geomorphic changes at different section lines. It also helped in delineating the extent and limits of river borne material, slope wash material, slump mass and the rock mass at different locations vis-à-vis the limits of dam body at different sections.

The slopes on the left bank above EL. ±620m were occupied by overburden material/slump mass/slope wash material). The contact between slump mass and the rock descends down at lower levels towards the downstream chainages particularly downstream of 60m.

The slopes in rocks are largely controlled by bedding joints which are dipping steeply towards the valley, particularly below EL. ±590m. Wedge failures were anticipated due to the intersection of bedding and foliation joints which were dipping gently towards downstream. Planar failures along the shear zones parallel to bedding and foliation joints were also anticipated during excavation for reaching the desired foundation grades and also for constructing the approach roads.

On the right bank the slope profile above EL. ±590m, on most of the sections, was occupied by huge thickness of river borne material (RBM) of the fossil channel. The weathered and puckered phyllitic rocks were observed at the dam axis above El±620m which again was overlain by overburden material above EL. ±690m.

At the downstream sections the slopes at the higher stretches (above EL. ±675m) were also occupied by RBM which finally merges with the slope wash material. The excavation for the power house pit would involve removal of the huge thickness of RBM and also hard rock. The bedding joint was favorably oriented as it was dipping into the

hill whereas planar failures were anticipated along foliation and WSW dipping joints. Wedge failures were anticipated due to the intersection of foliation and WSW dipping joints.

4.1 Kinematic Checks:

To find out the common modes of failures in the rock mass kinematic checks (Richards and Atherton 1987) for either bank were performed following the standard practice of stereo plotting of the joint planes and their poles, analyzing them on the slope faces with respect to the friction. The day light and toppling envelops (Richards and Atherton 1987, Priest and Brown, 1983) were drawn for the individual slopes and potential modes of failures along prominent discontinuities were identified and results were corroborated with the findings of geological profiling. For Kinematic checks the slopes on the left bank were divided in two categories i.e. above EL.±590m and below EL.±590m because of the change in the attitude of bedding at that level.

The slopes on the right abutment were also divided in two segments i.e. above EL. ±575m and below EL. ±575m because it marked the location of power dam blocks and also the location of inlet portal of main access tunnel. The kinematic checks for phyllitic quartzite and phyllite thinly bedded for different angles of friction and different angles of cut slope as given in Table 2.

Table 2
The Inputs for the Kinematic Checks

Abutment	Slope Azimuth	Cut angles	Rock	Friction (Degrees)
Left	N 136^0	75^0 63^0 45^0	PQ (W0)	34
			PT (W0)	26
Right	N 316^0	75^0 63^0 45^0	PQ (W0)	36
			PT (W0)	33

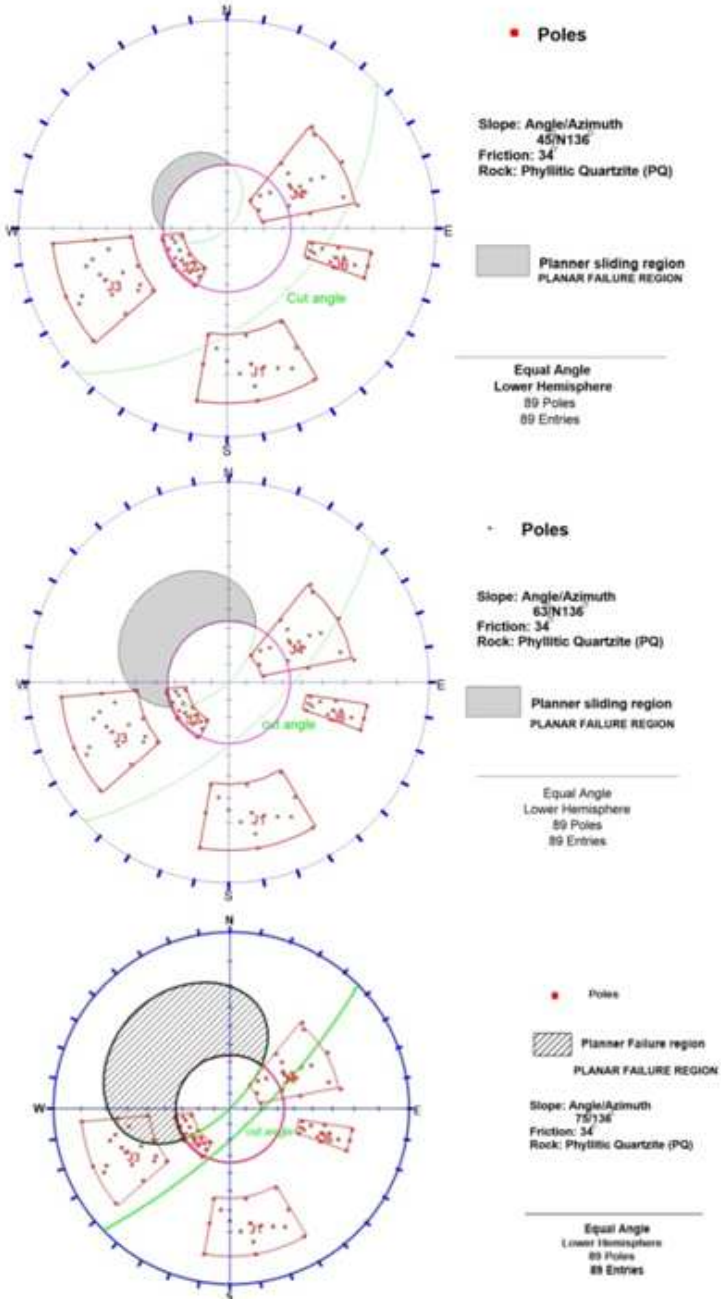

Figure 2 Stereo plots of Joints for Left Bank

The results of kinematic checks have been tabulated in Table 3.

Table 3
Results of Kinematic checks at Koteshwar dam project

Abutment	Elevation	Rock	Angle	Planar failure	Wedge failure	Toppling
Left	Above El±590m	PQ	45^0	None	None	J1
			63^0	Some of J2	None	J1
			75^0	J2 and J3	None	J1 & J6
		PT	45^0	J2	None	J1
			63^0	J2 & J3	Intersection of J1&J2	J1
			75^0	J2 & J3	Intersection of J1&J2	J1 & J6
	BelowEl±590m	PQ	45^0	J1	None	None
			63^0	J1,J2&J3	Intersection of J1&J3	None
			75^0	J1,J2 and J3	Intersection of J1 & J3 Intersection of J1 & J4 lies close to failure region	J5
		PT	45^0	J1&J2	Intersection of J1 & J2	None
			63^0	J1,J2 & J3	Intersection of J1 & J2 Intersection of J1 & J3	J5
			75^0	J1,J2 & J3	Intersection of J1 & J2 Intersection of J1 & J3 Intersection of J1 & J4 lies close to failure region	J5
Right Bank	Above El±575m	PQ	45^0	J1	None	None
			63^0	J1&J3	Intersection of J4&J5	None
			75^0	J1,J3 and J5	Intersection of J1&J5 Intersection of J4&J5	none
		PT	45^0	J3	None	None
			63^0	J3	Intersection of J4&J5	None
			75^0	J3 & J5	Intersection of J4&J5 Intersection of J1&J3 lies close to failure region	None
	BelowEl±575m	PQ	45^0	None	None	J1
			63^0	J3	Intersection of J4&J5	J1
			75^0	J3 &J5	Intersection of J4&J5 Intersection of J1&J3 lies close to failure region	J1
		PT	45^0	J3	None	J1
			63^0	J3	Intersection of J4&J5	J1
			75^0	J2,J3 & J5	Intersection of J4&J5 Intersection of J1&J3 lies close to failure region	J1

5. Designed Cut Slopes:

The results of kinematic checks which indicated the dominant modes of failures (Table 3) on either abutment for the two dominant types of lithologies noticed at the dam site,

guided the design of cut slopes at the dam site and also helped in devising the support measures. The details of cut slopes designed for either abutment is given in Table 4:

Table 4
Details of the Designed Cut Slopes at Koteshwar Dam Site

Bank	Elevation (m)		Slope Angle	Slope Height (m)	Bench Width (m)
	From	**To**			
Left Bank	618.50	599	53^0	19.50	4 - 5
	599	564	63^0	35	4 - 5
	564	547.50	53^0	16.50	4 - 5
	547.50	534	53^0	13.50	4 - 5
	534	521	45^0	13	River Bed Portion
Right Bank	618.50	602.50	70^0	16	4 - 5
	602.50	576.50	76^0	26	77m wide Platform for Power Dam Blocks
	576.50	559	63^0	17.50	4 - 5
	559	521	63^0	38	River Bed Portion

6. Support Measures:

The largest wedges of 4m – 4.5m depth, having volume of about 10 – 13 cubic meters and weight about 27 tons to 37 tons were expected in the massive phyllitic quartzites whereas smaller wedges of about 1 cubic meter and weight of 2.7 – 3 tons were expected from thinly bedded phyllites .

The tentative support for arresting the planar and wedge failures was considered by means of a combination of rock bolting and reinforced Shotcreting. The support within the limits of dam cut profile consist of 5m long rock bolts of 25mm diameter at a spacing of 3m c/c whereas the spacing beyond dam profile was reduced to 2m c/c on either abutment. The support was augmented by Shotcreting of M-30 to M-40 grade that was pneumatically shot in two layers of 50mm with a layer of chain link fabric in between. To reduce the pore pressure and release the interstitial water from the rock openings and joints, drainage holes 3m – 5m deep in the rock at 3m c/c spacing were provided.

The slopes above the top elevation of dam on either bank and those above the powerhouse pit area above El 575m were occupied by weathered rock mass, RBM and slope wash material. The slopes on the RBM were designed at cut angles varying between 45^0 and 53^0 and those on slope wash material were graded at angle not exceeding 45^0.

It thus allows maintaining the free draining characters of the RBM material and does not allow built up of pore pressure in case of saturation during rains. Lined drains were provided at the bottom of every slope cut to collect the water from the slope.

References:

1. Bahuguna, Harish (2017)**.** Geological and Geotechnical Challenges Faced at Koteshwar Hydroelectric Project District Tehri Garhwal, Uttarakhand, INDOROCK-2017, 7th Indian Rock Conference organized by Indian Society of Rock Mechanics and Tunneling Techniques, ISBN 81 – 86501 – 25 – 1 Pp 368 – 378.
2. Bahuguna, Harish (2016). Application of Block Tectonic Model and Inter block Tectonics at Koteshwar Dam Project International Conference on Recent Advances in Rock Engineering, 16- 8 Nov. 2016, Bengaluru, India, pages 77 - 84.
3. Bahuguna, Harish (2016). Engineering Geological Evaluation of Koteshwar Dam Foundation, Unpublished D-Phil thesis, HNB Garhwal University, Srinagar Garhwal, PP 1 – 328.
4. Barton N., Lien R, Lunde J. (1974). Engineering Classification of Rock Masses for the Design of Tunnel Support. Rock Mech. 6(4): 189 – 236.
5. Barton N. (2002). Some New Q-Value Correlations to Assist in Site Characterization and Tunnel Design, Int. Jour. Of Rock Mechanics and Mining Sciences, , p 189 – 216.
6. Barton, N. And Choubey, V. (1977). The Shear Strength of Rock Joints in Theory and Practice, Rock Mech. 10 (1-2) 1-54.
7. Bieniawski Z. T. (1989). Engineering Rock Mass Classification: A Complete Manual for Engineers and Geologists in Mining, Civil and Petroleum Engineering. New York: Wiley, 251p.
8. Priest S.D. and Brown E.T. (1983). Probabilistic Stability Analysis of Variable Rock Slopes. Trans. Ins. Min. Metall. Sect. A 92.
9. Richards LR and Atherton D., (1987). Stability of Slopes in Rock. Chapter 12 of "Ground Engineer's Reference Book", Butterworth's.