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RESEARCH ARTICLE

STABILITY EVALUATION OF SLOPES AROUND RIM OF RESEVOIR OF DIVERSION DAM OF VISHNUGAD–PIPALKOTI HYDROELECTRIC PROJECT, GARHWAL HIMALAYA, INDIA

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ABSTRACT

The Vishnugad–Pipalkoti Hydroelectric Project, a run-of-the river (ROR) scheme is located on Alaknanda River, a major tributary of river Ganga, in Chamoli District in the state of Uttarakhand. The project constitutes a 65m high diversion dam near village Helong (79°29'30" E and 30°30'50" N), a 13.4 km long Power tunnel (PT) and an underground power house to the south of village Hat (79°24'56" E and 30°25'31"N) to produce 444 MW of electric power. The reservoir area of Vishnugad–Pipalkoti project spread to an extent of 2.5km and is essentially constituted of quartzite rocks in and around the dam site and extent for a distance of 1.5km, where MCT separates the gneissic rocks on north. During drawdown conditions of the reservoir between MRL and DSL, the reservoir slopes may be subjected to alternate dry and water charged conditions, which may lead to reduction in the shear strength of slope forming materials. A detailed Engineering Geological evaluation of reservoir rim region has been carried out to assess the nature of instability of reservoir slopes during draw-down condition.

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INTRODUCTION

Many major hydroelectric projects like Tehri Dam have been constructed in Himalaya, in addition to a number of projects under construction are being planned. The Vishnugad–Pipalkoti Hydroelectric Project is one of the major run of the river (ROR) projects under construction in Alaknanda Valley. A 65m high diversion dam is being planned to be constructed near village Helong (79°29'30" E and 30°30'50" N), which causes a 2.5km long reservoir behind the dam. This water will be carried through a 13.4km long tunnel in order to produce 444 MW of power. The maximum reservoir level (MRL) is at EL± 1267m and the dead storage level (DSL) is at EL± 1252m with water fluctuation of about 15m during reservoir operations. The water fluctuations leading to alternate wetting and drying of hill slopes may result in instability of hill slopes. The already unstable or potentially unstable slopes around the rim of reservoir need to be identified and studied in detail with particular reference to its stability under draw-down conditions. The geological investigations indicate that the region comprises of Garhwal Group of rocks belonging to the Proterozoic age (GSI, 2012). These rocks are separated in the north from Central Crystalline Group of rocks by the Main Central Thrust. The project area lies within the Zone V of the Seismic Zoning map of India (IS1893 Part I, 2002). The location of the project reservoir is shown in Fig.1.

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Regional Geological Setting

The Garhwal and Kumaun Himalaya (Fig 2), forming the central part of the Himalayan folded belt, exposes rock types of varying age from Proterozoic to Late Tertiary period and are disposed in four major tectonic belts, designated as Foothill Siwalik belt, Lesser Himalayan belt, Central Crystalline and Tethyan belt. The geology of this area was studied by many pioneering researchers since nineteenth century (Middlemiss, 1885; Holland, 1908; Auden, 1935; Heim and Gansser, 1939; Rupke, 1974; G Fuchs and Anush K. Sinha 1978; Valdiya, 1980; Valdiya, 1995; Richards *et al*., 2005). Further based on the collective field evidences and studies, Himalayan mountain ranges have been categorized into six tectonic sheets (Fig 2) from north to south extending in series of parallel belts - (i) the Trans-Himalayan batholith; (ii) the Indus-Tsangpo suture zone; (iii) the Tethyan (Tibetan) Himalaya; (iv) the Higher (Greater)

Himalaya; (v) the Lesser (Lower) Himalaya; and (vi) the Outer (Sub) Himalaya, (Gansser, 1964; Le Fort, 1975; Thakur, 1992) The project area, forming a part of Alaknanda valley, is mainly constituted of rocks belonging to Garhwal Group in the Lesser Himalaya. These rocks are truncated by MCT (Fig 3) towards north about 1.5km from the dam site. Further north of MCT, Central Himalayan Crystalline rocks are exposed.

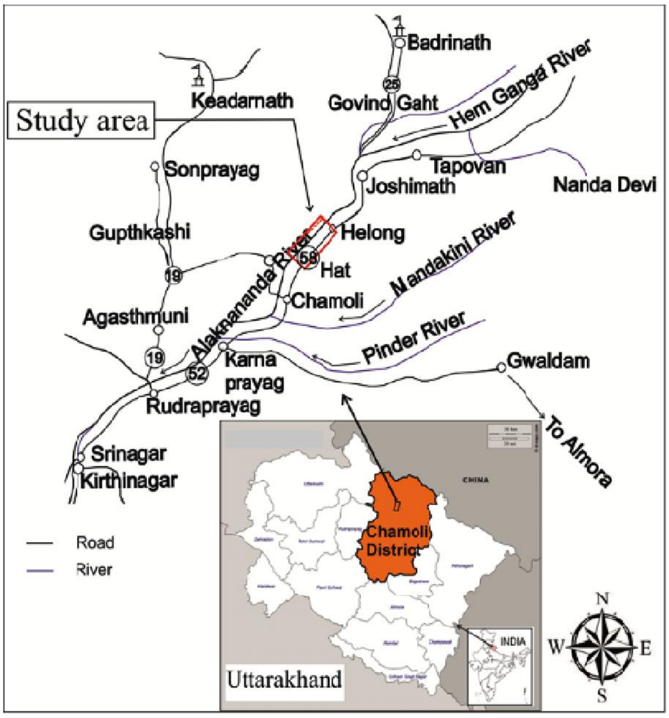

Fig. 1. Location Map of the study area

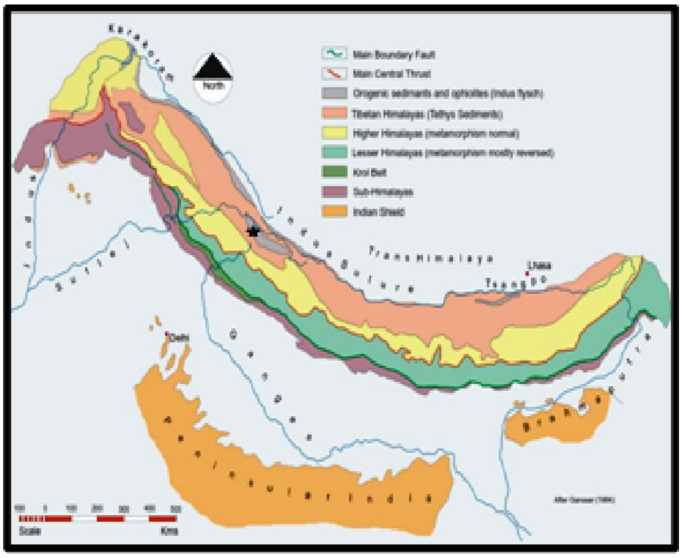

Fig. 2. The Regional Geological map of Himalayan range (After Ganesser, 1964)

3. Reservoir Geology

The 65m high dam will have a water spread that will extend to about 2.5 km upstream of the dam. The entire reservoir area was mapped on 1:10000 scale. During mapping, the unstable slopes as well as potentially unstable slope are identified for further detailed studies. Quartzite rocks are exposed in and around the dam site and extend well into the reservoir on the upstream side up to Main Central Thrust (MCT), which is present about 1.5km upstream of the dam. Further upstream of MCT, Granitic gneisses are present till the end of the reservoir. Debris and river borne materials (RBM) are seen often as isolated pockets in many locations on the left bank (Fig 3). Many of the potentially unstable locations fall within these zones. Structurally, foliation is the major geological discontinuity and two sets of well-developed joints (J1 and J2) can also be seen in the area (Table 1 and Table 2). In order carry out the stability analysis it was essential to delineate the rock slope and debris slope in the reservoir area (Table 3 and Table 4).

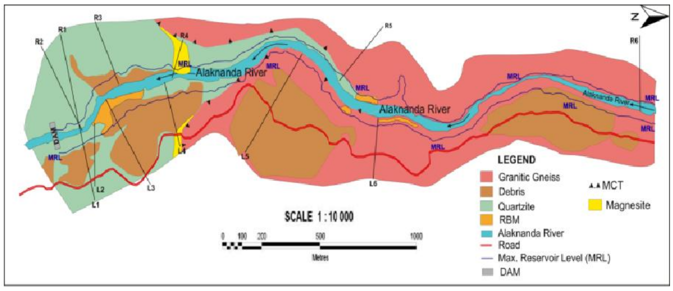

Fig. 3. Geological map of Vishnugad–Pipalkoti reservoir area with section lines and MRL

Table 1. General discontinuity attitude (Right Bank)

S.No.	Nature of discontinuity	Strike	Dip/Dip direction
1	Foliation	N300^0	400^o /N0300^0
2	Joint J1	N010^0	75^o /N280^0
3	Joint J2	N310^0	60^o /220^0

Table 2. General discontinuity attitude (Left Bank)

S.No.	Nature of discontinuity	Strike	Dip/Dip direction
1	Foliation	N300^0	400^o /N0300^0
2	Joint J1	N080^0	65^o /N170^0
3	Joint J2	N320^0	65^o/230^0

Table 3. Summary of slope sections on right bank

S. No.	Section	Location and distance form dam axis	Type of slope
1	R1	Near dam axis, 00 m	Rock slope
2	R2	Near intake structure, 40m	Rock slope
3	R3	U/S of Nall, 270 m	Rock slope
4	R4	Opposite of LSH-2, 490 m	Rock slope
5	R5	Along Urgam bridge, 1330m	Rock slope
6	R6	Near Kalpaganga, 2860m	Rock slope

Table 4. Summary of slope sections on left bank

S. No.	Section	Location and distance form dam axis	Type of slope
1	L1	Near dam, axis	Mainly rock with some debris talus at higher level
2	L2	Along intake of diversion tunnel, 110m	Mainly rock with some debris at base and higher levels
3	L3	Near intake of diversion tunnel, 180m	Debris at lower level rock, slope at mid and again debris at higher level
4	L4	Near LSH-2, 450m	Debris slope
5	L5	Near D3, 1120m	Debris
6	L6	Near D5, 1610m	Debris and river borne material below road level and rock above road level

4. Stability Evaluation of Hill Slopes in Reservoir Rim Area

The most important problems encountered during the operation of the reservoir are the seepage and hill slope instability around the rim of reservoir. The reservoir area of Vishnugad–Pipalkoti project is essentially constituted of quartzite rocks in and around the dam site and extent for a distance of 1.5km, where MCT separates the gneissic rocks on north. During drawdown conditions of the reservoir between MRL and DSL, the reservoir slopes may be subjected to alternate dry and water charged conditions, which may lead to reduction in the shear strength of slope forming material. This process may eventually cause instability of the hill slopes. As a consequence of this, the strategically important NH-58 highway, which is located just above MRL, may also face instability problems. On the basis of field investigations, 12 important potentially unstable locations, 6 each on both right and left banks have been chosen. Geological cross sections (Table 4.1) and kinematic analysis (Table 4.2) to identify the mode failure likely to occur at all selected slopes was done.

4.1. Right Bank

i) Section R1 (Fig 4.1): Comprises of quartzite rocks, which is fairly steep and extending for a height of 170m above the river bed. In fact, this slope section is in continuation with the slope section L1 on the left bank, It is located just upstream of the dam axis. It mainly consists of rock slope with thin debris cover at places above the reservoir level. The debris starts from EL± 1310m and further above, while MRL is limited to EL± 1269m. As such the drawdown will have no impact on the debris materials exposed above MRL. The major structural discontinuity foliation plane dips 35º towards north easterly into the hill on the right bank at an oblique angle forming stable wedges with the joint discontinuities J1 & J2.

ii) Section R2 (Fig 4.2): It is a rock slope with slope angles of more than 45□. The geological discontinuities were plotted in a stereo net and kinematic analysis carried out. The analysis shows that the slope has potential plane failure instability. In view of that, protection measures as indicated for R1 section is justified in this area.

iii) Section R3 (Fig 4.3): It is located about 270m from the dam site. It is steep rock slope with slope angles of more than 55□. The geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). The analysis indicates that no wedges, either plane or wedge are formed and hence stable in nature.

iv) Section R4: This section is located on a steep rock slope (>65°) about 490m upstream of dam axis. Dolomitic limestones intercalated with magnesite are exposed at the site (Fig 4.4). The observed geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). Since the foliation dips into the hill and other joints are not favourably aligned, no adverse wedges are formed at this site.

v) Section R5 (Fig. 4.5): This section is located just near the axis of the Urgam bridge on the right bank. It is located on a steep rock slope of more than 65°. Quartzites are exposed at the site. The observed geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). The study indicated that unstable wedges were likely to form at this site. However, since the slope is at the tail reaches of the reservoir, no measures are actually required at the site as the water will be present very close to the river bed level and hence may hardly has any impact on the stability of the slope due to drawdown.

vi) Section R6 (Fig. 4.6): Is located near Kalpaganga, 2860m upstream of dam axis. It is located on a steep rock slope of more than 75°. Gneissic rocks are exposed at the site. The slope section R6 is located away from the dam axis. The MRL is located just less than 10m above the river bed and hence not likely to induce instability of rock slopes.

On the basis of stability analysis, it is recommended to adopt some slope treatments for slopes R1, R2, R3 and R4 (Table 4.3).

4.2 Left Bank

On the left bank, six important slopes have been chosen for detailed stability studies. More detailed analysis has been done, since they are potentially unstable in nature.

i) Section L1 (Fig 4.1): It is located just upstream of the dam axis. It mainly consists of rock slope with thin debris cover above, which is seen above the reservoir level. The debris starts from EL± 1310m and further above, while the MRL is limited to EL± 1269m. The geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). The rock slope is found to be stable under static and dynamic conditions with large factors of safety against wedge failure. However, under extreme conditions i.e. dynamic condition with tension crack filled with water, the failure is likely to occur due to over toppling. Here, the site is located just adjoining the dam axis hence, the slope has to be protected adopting similar measures suggested for R1 section. The thin debris above the reservoir level shall be removed as the quantum is very less. The protection measures should continue for 100m distance on each side of the dam axis.

ii) Section L2 (Fig4.7): The section basically shows a rock slope with thin debris cover seen between El ±1295m and El ±1356m. The geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). The rock slope is found to be safe against wedge failure, though some overtopping may occur in extreme conditions. However, the debris slope is not stable under the worst conditions. Due to close proximity of the section to the dam axis, slope protection work of rock and debris being adopted at section L1 should be extended to this section also.

iii) Section L3 (Fig 4.8): The section is located across the approach roads, which give access to the main dam from NH-58. The slopes are mainly characterized by thick debris extending from river bed to about El ± 1290m. The rock stability analysis indicates that they are stable (Table 4.2). However, the debris slope may fail under dynamic and saturated conditions. The alternate draw-down conditions of water level may induce instability in the bottom portion of debris. Since most parts of the debris mass (about 80%) lie below MRL, the failure of the debris, may not adversely affect the overall reservoir capacity.

Table 4.1 Geological cross-section details for Right and Left Bank

Slope Section	Height (m)	Reservoir water level (m)	Attitude of Slope Face (°)	Attitudes of Discontinuities (°) Dip/dip direction
R1	172	55	60/105	35/030, 75/260, 90/220
R2	273	55	60/105	35/030, 60/135, 90/220
R2	72	55	70/105	35/030, 60/135, 90/220
R3	217	50	49/110	60/035, 75/200, 85/150
R4	80	50	76/153	50/030, 65/270, 85/150
R5	197	30	71/180	35/055, 77.5/180, 67.5/270
R6	139	5	89/135	45/010, 40/180, 80/280
L1	172	55	70/299	35/030, 75/260, 90/220
L2	65	55	72/313	40/030, 60/135, 90/220
L3	82	50	60/282	40/020, 75/280, 90/220
L4	140	50	34/298	40/030, 65/225, 85/115
L5	40	40	42/350	37.5/030, 50/200, 90/110
L6	112	20	74/321	40/020, 75/070, 40/180

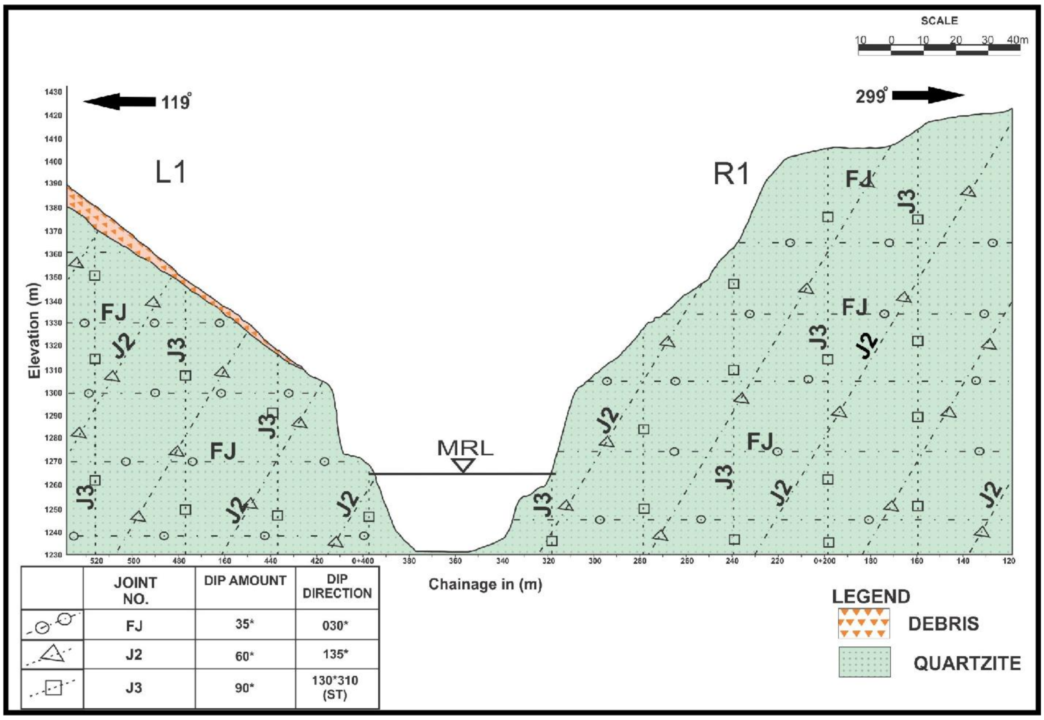

Fig. 4.1. Geological cross section of R1and L1

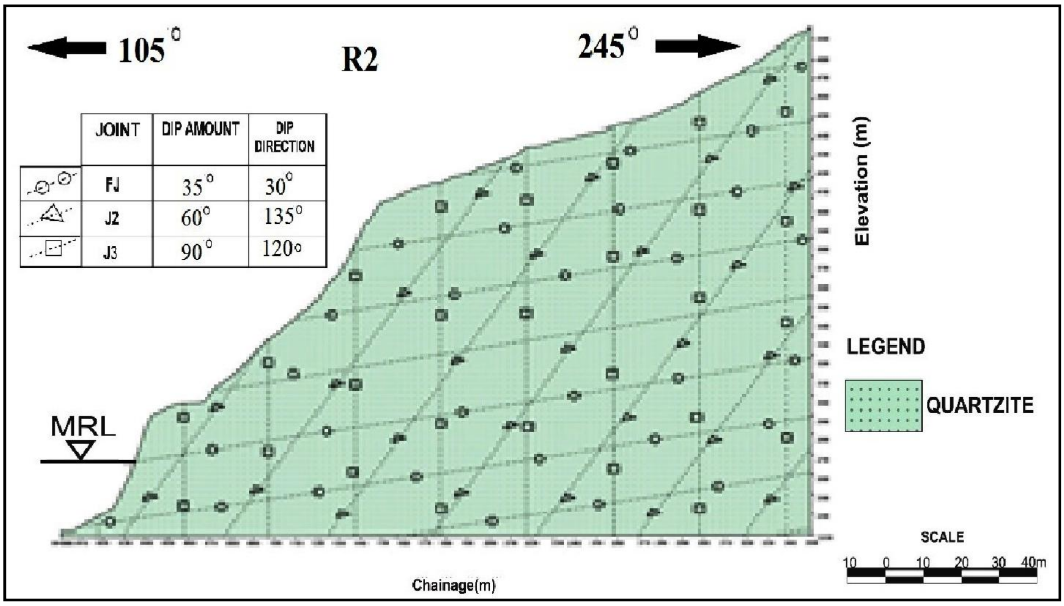

Fig. 4.2. Geological cross section of R2

Table 4.2 Kinematically possible failure modes in rocks: Right and Left bank

Slope Section	Planar failure		Wedge failure		Toppling failure	
	Yes/No	Along	Yes/No	Along	Yes/no	Along
R1	-	-	-	-	-	-
R2	Yes	J1	Yes	FJ& J1	-	-
R3	-	-	Yes	FJ& J2	-	-
R4	Yes	J2	-	-	Yes	J2
R5	Yes	J2	-	-	-	-
R6	-	-	Yes	J1& J2	-	-
L1	-	-	Yes	J1&J2	-	-
L2	-	-	Yes	J1& J2	Yes	J1
L3	Yes	J1	Yes	FJ& J1	-	-
L4	-	-	Yes	FJ& J2	Yes	J2
L5	-	-	Yes	FJ&J2		-
L6	-	-	Yes	FJ&J2	-	-

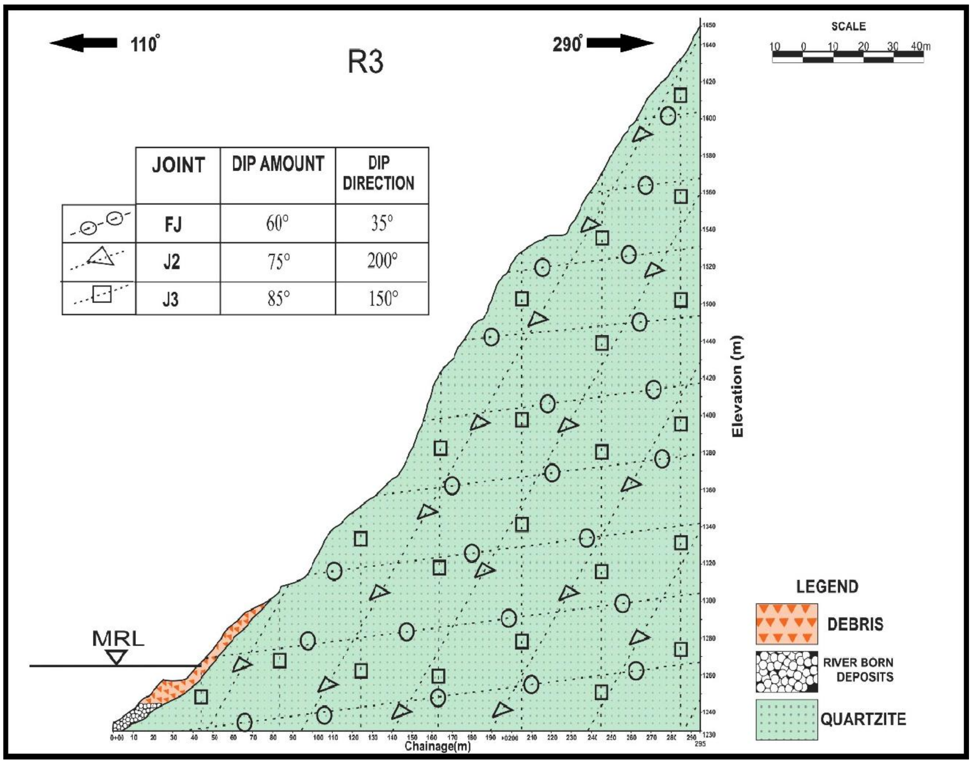

Fig.4.3. Geological cross section of R3

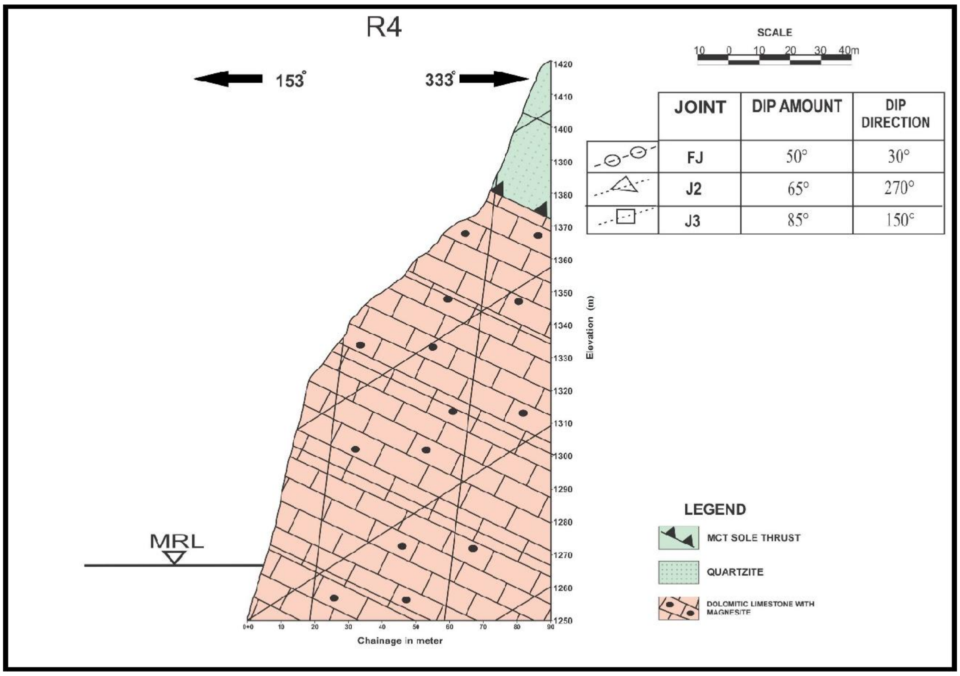

Fig 4.4. Geological cross section of R4

Table 4.3. Right bank reservoir slope stability analysis results and recommendations

Case	Section	Joint Parameters	Tension Crack	Joints	Static FoS	Recommendation	Dynamic FoS	Recommendation
1	R1	Dry	Dry	No Wedge	-	-	-	-
2		wet	Dry	No Wedge	-	-	-	-
3		wet	Filled up	No Wedge	-	-	-	-
1	R2	Dry	Dry	FJ & J2	1.96	Reinforcement not required	1.32	Spacing of Anchor - .4299m
2	Uphill	Wet	Dry	FJ & J2	1.85	Reinforcement not required	1.24	Spacing of Anchor - .3456m
3	slope	Wet	Filled up	FJ & J2	Over Topple	Spacing of Anchor - .1137m		Spacing of Anchor - .1137m
1	R2	Dry	Dry	FJ & J2	3.11	Reinforcement not required	1.82	Reinforcement not required
	Downhill			J2 & J3	4.27	Reinforcement not required	2.39	Reinforcement not required
2	slope	Wet	Dry	FJ & J2	2.62	Reinforcement not required	1.48	Spacing of Anchor - 3.9401m
				J2 & J3	3.01	Reinforcement not required	1.73	Reinforcement not required
3		Wet	Filled up	FJ & J2	Detached	Spacing of Anchor - .3650m	Detached	Spacing of Anchor - .3650m
				J2 & J3	1.69	Reinforcement not required	2.07	Reinforcement not required
1	R3	Dry	Dry	No Wedge	-	-	-	-
2		Wet	Dry	No Wedge	-	-	-	-
3		Wet	Filled up	No Wedge	-	-	-	-
1	R4	Dry	Dry	No Wedge	-	-	-	-
2		Wet	Dry	No Wedge	-	-	-	-
3		Wet	Filled up	No Wedge	-	-	-	-
1	R5	Dry	Dry	No Wedge	-	-	-	-
2		Wet	Dry	No Wedge	-	-	-	-
3		Wet	Filled up	No Wedge	-	-	-	-
1	R6	Dry	Dry	J2 & J3	1.33	Spacing of Anchor - .2832m	.96	Spacing of Anchor - .1197m
2		Wet	Dry	J2 & J3	1.22	Spacing of Anchor - .2172m		Spacing of Anchor - .0900m
3		Wet	Filled up	J2 & J3	Over Topple	Reinforcement not required	Over Topple	Reinforcement not required

FoS – Factor of Safety

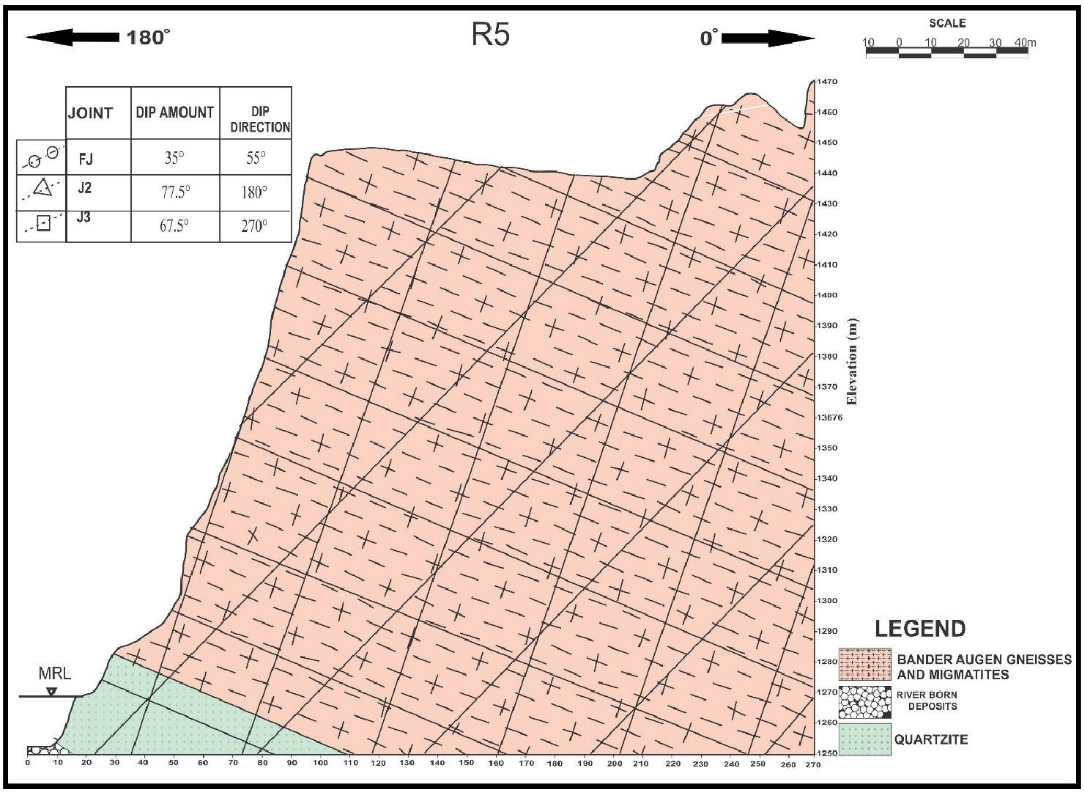

Fig. 4.5. Geological cross section of R5

Table 4.4 Left bank reservoir slope stability analysis results and recommendations

Case	Section	Joint Parameters	Tension Crack	Joints	Static FoS	Recommendation	Dynamic FoS	Recommendation
1	L1	Dry	Dry	J2 & J3	8.18	Reinforcement not required	6.74	Reinforcement not required
2		Wet	Dry	J2 & J3	5.74	Reinforcement not required	4.71	Reinforcement not required
3		Wet	Filled up	J2 & J3	Over Topple	Reinforcement not required	Over Topple	Reinforcement not required
1	L2	Dry	Dry	J2	Over Topple	Reinforcement not required	-	-
2		Wet	Dry	J2	Over Topple	Reinforcement not required	-	-
3		Wet	Filled up	J2	Over Topple	Reinforcement not required	-	-
					FoS – Factor of Safety			
1	L3	Dry	Dry	Debris	2.3	-	-	-
2		Wet	Dry		1.75	-	-	-
3		Wet	Filled up		1.75	-	-	-
1	L4	Dry	Dry	Debris	2.3	Reinforcement not required	-	-
2		Wet	Dry		1.75	Reinforcement not required	-	-
3		Wet	Dry		1.75	Reinforcement not required	-	-
1	L5	Dry	Dry	FJ & J2	Tension crack not valid	-	-	-
2		Wet	Dry	FJ & J2	Tension crack not valid	-	-	-
3		Wet	Filled up	FJ & J2	Tension crack not valid	-	-	-
1	L6	Dry	Dry	FJ & J2	2.57	Reinforcement not required	1.83	Reinforcement not required
2		Wet	Dry	FJ & J2	2.20	Reinforcement not required	1.52	Reinforcement not required
3		Wet	Filled up	FJ & J2	Detached	Spacing of Anchor - .6978m	Detached	Spacing of Anchor - .6978m

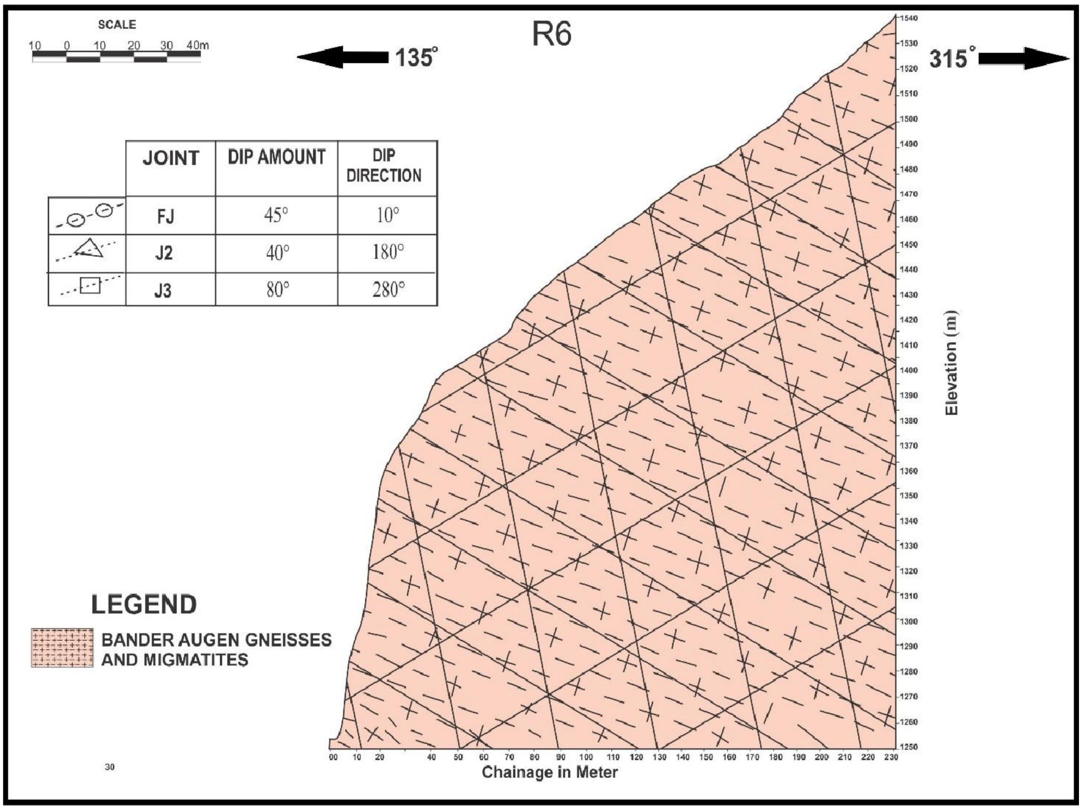

Fig. 4.6. Geological cross section of R6

Table 5.1 Concluding remarks on stability and corrective measures required (Right Bank)

S.No.	Section	Slope Type	Stability Status	Corrective Measures
1	R1	Rock	No wedge/planer failure expected.	Due to proximity to dam axis, slope flattening, cable anchors, shotcreting, surface drainage and drainage holes should be provided. The protection measures should preferably continue 100m on either side i.e. u/s and d/s of the dam.
2	R2	Rock	Wedge instability when tension crack is filled with water. Planar failure is likely under dry static conditions.	Same as R1
3	R3		Probability of unsatisfactory performance ranging between 11.99 to 20.78% for circular failure of rock mass. Most likely values of FOS are also smaller than 1.5	Surface drainage to be improved and weep holes to be provided. Steep slopes has to be stabilized with shotcrete and cable anchors up to 10m above MRL.
4	R4	Rock	Stable	No measures are required
5	R5	Rock	Unstable under normal condition.	Reinforcement is required to make slope stable. However, the slope is at the end of the reservoir rim, an efficient drainage system will reduce risk of failure to a great extent. The slope should be kept under watch.

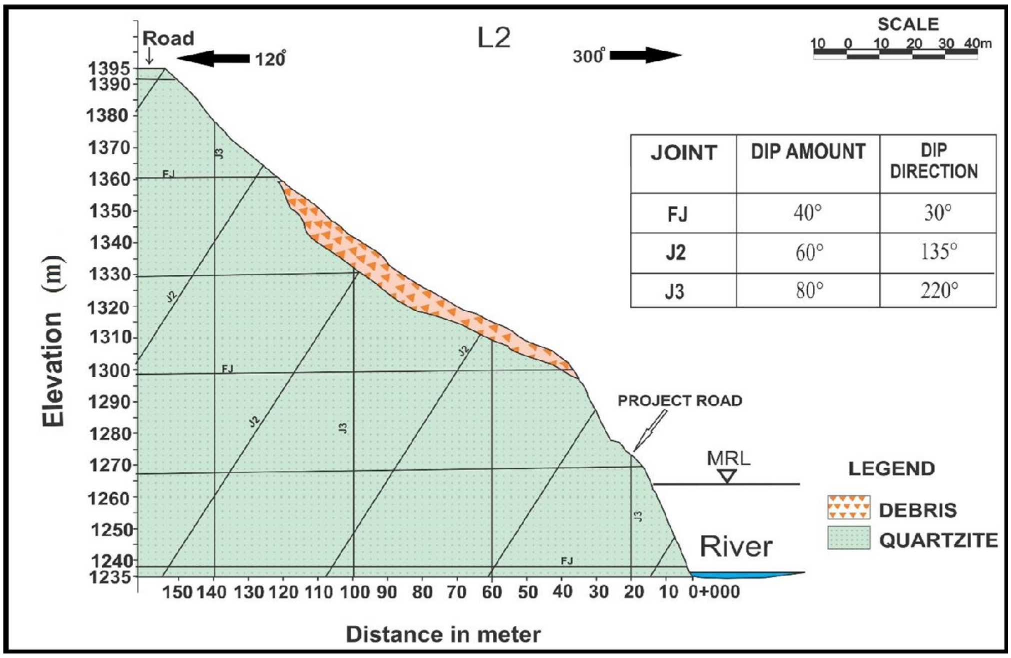

Fig 4.7 Geological cross-section of L2

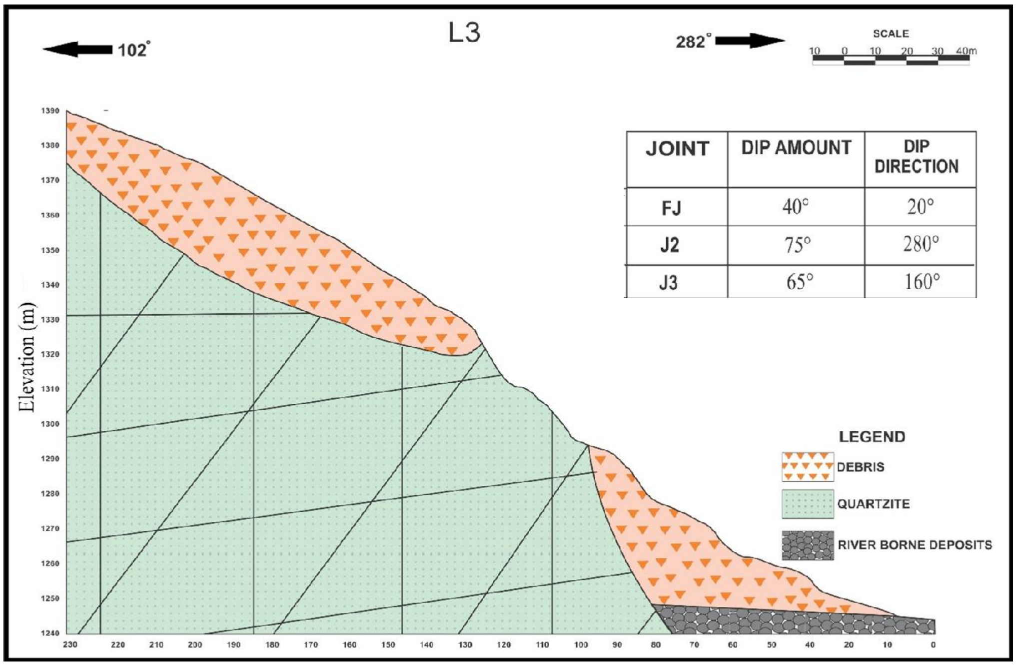

Fig 4.8. Geological cross section of L3

Table 5.2 Concluding remarks on stability and corrective measures required (Left Bank)

S.No.	Section	Slope Type	Stability Status	Corrective Measures
1	L1	Rock slope with thin debris cover	Wedge instability under extreme conditions	Due to proximity to dam axis, protection measures suggested at R1 should be adopted. The thin debris may either be removed or proper retaining wall with adequate drainage should be provided at the toe of the debris. The protection measures should continue for 100m distance on either side of the dam axis.
2	L2	Rock slope with debris	Unstable debris under extreme conditions	Same as L1
3	L3	Deep Debris	Unstable debris under dynamic and saturated condition. Most of the debris mass lies below FRL. Failure of debris not likely to affect the overall reservoir capacity.	The debris may be allowed to slide down into the river. In the upper part gabion wall should be provided. The gabion should be supported (reinforced) with steel piles anchored into sound rock for adequate depth.
4	L4	Rock	The underlying rock is stable. The debris slope is just stable under normal conditions. In case of failure of slope above, the slide material may be easily accommodated within terrace. It is not likely to create any harm to the reservoir.	Improve drainage through drainage holes. The slope needs to be carefully watched.
5	L5	Rock slope with debris		
6	L6	Thick debris slope	Unstable debris slope extends into the zone of water level fluctuations. Debris is likely to sink and slide down to get flattened to stable slope angle.	The slope should be kept under watch.

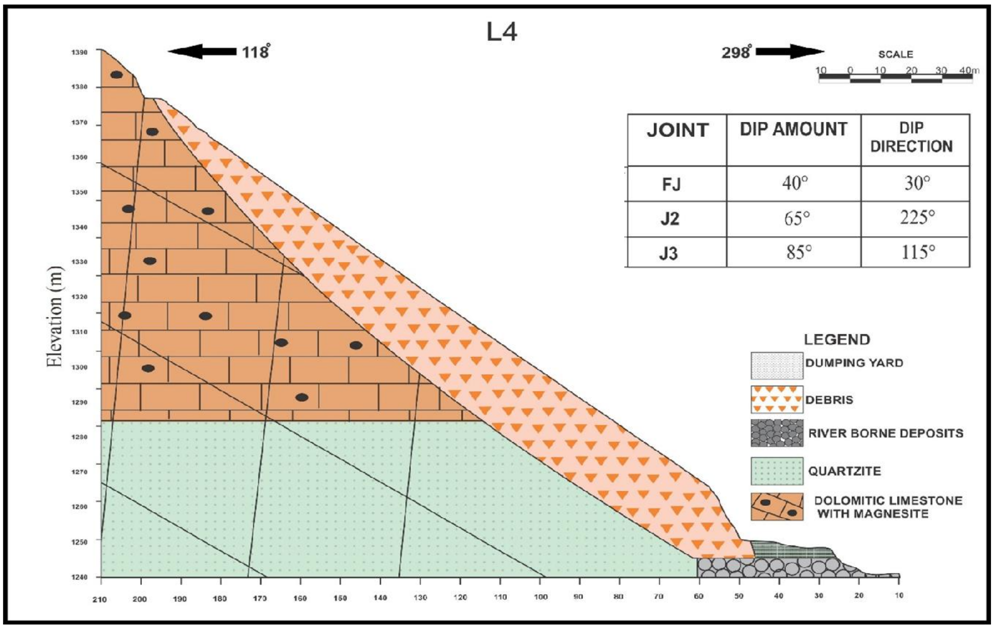

Fig. 4.9. Geological cross section of L4

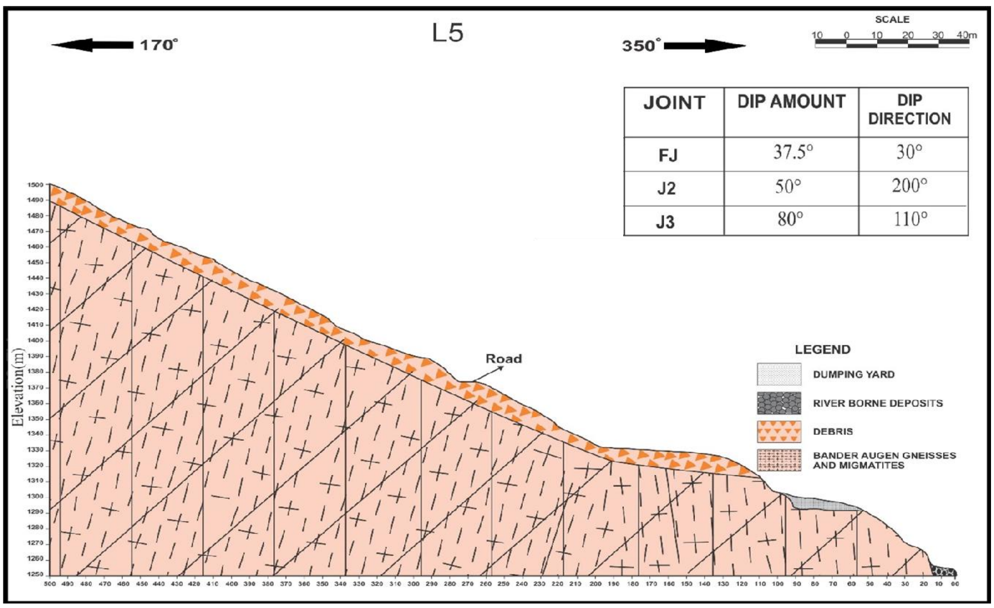

Fig 4.10. Geological cross section of L5

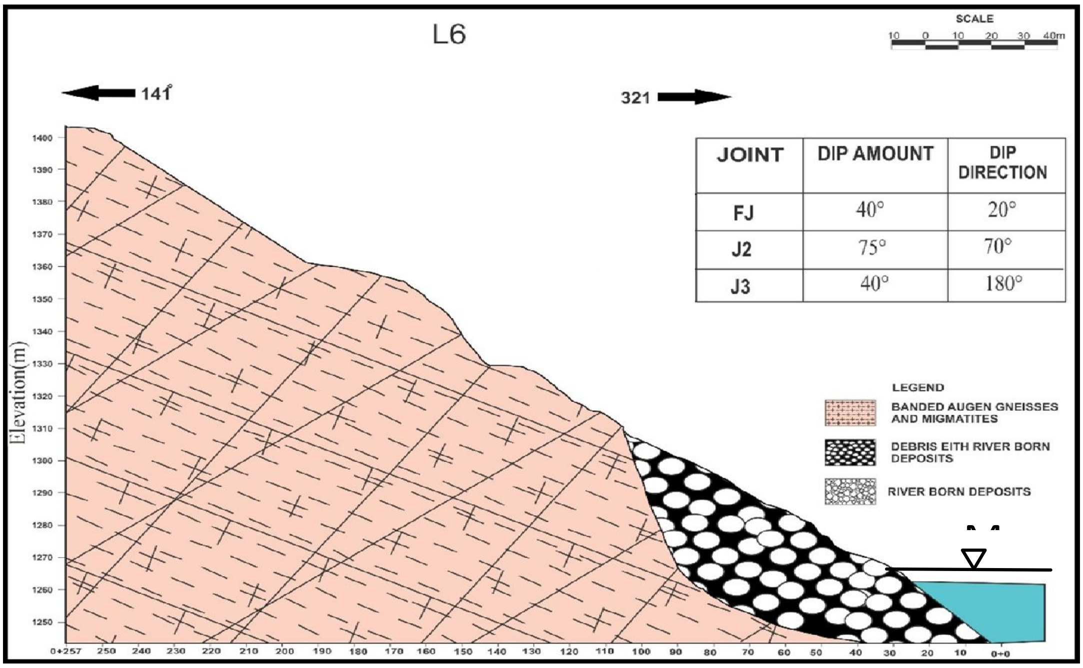

Fig 4.11. Geological cross section of L6

iv) Section L4 (Fig 4.9): This section is located about 450m upstream of the dam axis. It is a rock slope with thick continuous debris occupying the entire slope above the river bed and extending up to El ± 1310m close to NH-58. The maximum reservoir level (MRL) is located well in the middle of the debris slope. Though the underlying rock is likely to remain stable as indicated in the kinematic analysis (Table 4.2), the debris slope may become unstable when saturated or subjected to dynamic conditions.

A near horizontal wide terrace is present at the toe of the slope. In case of any failure of the slope above due to draw-down conditions, the slide material may be easily accommodated within wide terrace so as to flatten the overall debris slope. However, since the failed materials will remain at the toe and will get compacted by the reservoir water, it is likely to get stabilized with time.

v) Section L5 (Fig 4.10): This section is located downstream of Urgam bridge. The slope has an average angle of 25°-30° with moderately thick (8-10m) debris materials seen above the rock slope. The debris materials extend only up to El ±1320m in the lower lever and further down rock slopes are present. Since the MRL is at El ± 1269m, the top of water level will be located within the rocks and hence the debris slopes will not be affected due to reservoir water. The geological discontinuities were plotted in a stereonet and kinematic analysis carried out (Table 4.2). The analysis indicates that the slopes are stable as no unstable wedges are formed.

vi) Section L6 (Fig 4.11): This section is located about 270m upstream of Urgam Bridge. Though rocks are present in the upper reaches of the slope, a thick deposit of RBM is seen at the toe of the slope up to the river bed level. While RBM extends form river bed to El ±1310m, the MRL extends up to El ±1270m that is up to the middle of RBM deposit. During water draw-down conditions, the alternating saturation and dry conditions may induce instability of the RBM deposit causing minor instability and sliding leading flattening of the gradient. In view of the limited extension of RBM and the slide deposit will lie at the toe area and get compacted due to reservoir water, the flattened deposit will get stabilized in a short time frame. In fact, the thick layer of RBM at the toe of the rock slope provides support to rock slope above.

Stability analysis result and required slope treatments for slopes on the left bank have given in Table 4.4.

5. Conclusion

The 65m high Vishnugad dam will have a water spread that will extend to about 2.5km upstream of the dam. Quartzite rocks are exposed near the dam site and extend well in to the reservoir on the upstream side up to Main Central Thrust (MCT), which is present about 1km upstream of the dam. Granitic gneisses are exposed further upstream till the end of the reservoir. The small reservoir to be created due to dam construction will be mostly lying close to the river bed except in reaches close to the dam site. During reservoir mapping and based on the potentiality of the slope for instability problems, twelve slopes, six on each bank were chosen for detailed study. On the left bank, the slopes having debris cover at MRL show minor instability problems due to draw down conditions (L3, L4 and L7). However, initial instability, though may cause sliding of debris, the slided material will get accumulated at the toe and the reservoir water will help to compact it. As a result, there will be reduction in the slope angle initially but in a few years of time; it will tend to get stabilized. No major landslides are anticipated on the left bank. However, further stability measures may be adopted on the slopes just above the dam site on both the banks. These are only additional measures to stabilize the more important slopes above the dam. The right bank slopes are generally rock slopes, which are generally stable and do not require any stability measures. The concluding remarks on the stability and the required corrective measures for both Right Bank and Left Bank are given in Tables 5.1 and 5.2 respectively.

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