CONSTRUCTION OF THE KISHANGANGA HYDROELECTRIC PROJECT CONCRETE FACED ROCK FILL DAM, INDIA

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ABSTRACT

Kishanganga is the first mega hydropower plant located in the north western Himalayas, in the Gurez valley of Kashmir region of India. The Concrete Faced Rockfill Dam at an altitude of about 2400m, has a height of 33m and a crest length of 145m. The concrete face is tied to a 31m deep plastic concrete cut-off wall. Construction in the remote and mountainous terrain of Kashmir where the temperature drops below freezing for 4 months of each year, provides its own challenges. In addition, with few CFRD dams constructed in India, developing the trust and confidence of the owners and local community, was key to successful completion. The rock mass comprises fractured Panjal Volcanics and the abutments required investigations to assess joint shear strength parameters to allow assessment of stability and adaptation of rock slope design. A deep cut off wall in front of the dam made up of plastic concrete has been constructed using borehole drill and blast method to economize the cost and construction time with locally available technology. This paper presents the challenges and the solutions developed during the design and construction of the dam and in particular, the right bank plinth, extruded kerb, rock slope stabilization, face slab joints and cutoff wall.

1. PROJECT BACKGROUND

1.1 Introduction

The Kishanganga Hydroelectric Project is located in the Baramulla District of the Union territory of Jammu and Kashmir in north-west India. It is a run of river scheme which involves transfer of water from the Kishanganga River in the Gurez valley to the Bonar Nallah near Bandipore in the Kashmir valley. In 2009, the National Hydroelectric Power Corporation Ltd (NHPC) awarded the contract for the design and construction of the 330MW Kishanganga Hydroelectric Project to a joint venture of Hindustan Construction Company (HCC) and Halcrow (now Jacobs). The project would take nine years to complete with a capital value of contract as $554 million. As an engineering, procurement and construction contract, the joint venture was responsible for all civil and associated infrastructure works; supply, installation, testing and commissioning of all hydro-mechanical and electromechanical plant and machinery components including generating units. Jacobs were responsible for the detailed design of the civil works, co-ordination with the electrical and mechanical designer/contractor, and providing site design liaison and coordination.

1.2 Geology of the Dam Site

The bedrock geology of the dam site comprised andesitic rocks from the Panjal Volcanics Group. The dam was founded upon alluvial deposits, which were approximately 30m thick comprising boulders, cobbles and gravel in a sand and silt matrix.

1.3 Concrete Faced Rockfill Dam

The Concrete Faced Rockfill Dam (CFRD), has a maximum height of 32m, a crest length of 145m and a minimum crest width of 10m and the spillway on left of the dam has a 2000m3/sec capacity. The upstream concrete face is tied to the cut-off wall on the upstream end via a toe plinth slab. The maximum water level is retained at elevation 2390.0m with the dam crest at 2395.0m.

1.4 Headrace Tunnel

The scheme has a 665m gross head with the water passed via a 23km long headrace tunnel and steel lined high-pressure shaft to the underground powerhouse near Bandipore. The tunnel was constructed using drill and blast methods over 8.47km forming a 6.24m diameter horseshoe shape; with the remaining 14.53km length, 5.2m diameter segmentally lined circular section excavated using tunnel boring machine (TBM) technology. At depths of up to 1400m the TBM tunneling works represented a significant challenge and was the first TBM tunnel to be successfully constructed in the Himalaya (Swannell *et al*, 2016).

The anticipated squeezing conditions were such that, in some areas, it would not be possible to progress the TBM in the normal way and/or to provide a segmental lining of practicable strength to avoid overstressing without implementing special measures. Design and construction planning was therefore based on a risk management approach with contingency procedures and criteria developed to allow the risks to be managed effectively. The TBM was designed with specific features to reduce the risk of entrapment (Ariza et al, 2015). Special measures were adopted to reduce loads on the segmental lining included overcutting, pre-excavation grouting and consolidation grouting (Swannell *et al*, 2016).

1.5 Diversion Tunnel and Cofferdams for River Diversion

The diversion tunnel comprised a 9.5m diameter concrete lined horseshoe tunnel 570m long and was constructed as a drill and blast tunnel with a design capacity of 1,000m^3/s on the left abutment of the spillway. On impoundment the diversion tunnel was closed by means of a bulkhead gate at its upstream end and a concrete plug was subsequently constructed within the tunnel.

An upstream cofferdam, with crest elevation El. 2384.0m, was built as a zoned rockfill embankment with an inclined clay core wall. Due to the thick alluvial deposits present, under-seepage was controlled by a grout curtain comprising of two rows of grout holes one meter apart each at 5m spacing. The downstream cofferdam, with crest elevation 2372.0m, is of a similar arrangement to the upstream cofferdam apart from the omission of a grout curtain as there was very little driving head in the upstream direction. The crest width of 7m enabled the crest to be trafficked during construction.

1.6 Gated Spillway

The spillway is located on the left abutment and consists of the main spillway and the auxiliary spillway. The main spillway with a crest elevation of EL 2370m is a controlled chute spillway with three radial gates of 7.0m wide by 9.5m high clear opening and a flip bucket with concrete apron for energy dissipation at the downstream end. 1.3 m dia circular steel pipe of about 80m length for ecological release have been provided for the release of 9 cumec discharges throughout the year.

2. GENERAL ARRANGEMENT OF THE DAM

The dam comprises a zoned embankment. The upstream slope of the embankment is 1(vertical):1.75(horizontal) and downstream slope is 1:1.5 (Figure 1).

The concrete face included design to resist the ice cover as well as freeze thaw/thaw cycles and comprised 400mm thick 35MPa reinforced concrete forming slabs 10.3m wide with vertical shrinkage joints. The reinforcement represents approximately 0.4% of the concrete section in each direction for the slabs and was positioned to distribute uniformly the resistance to shrinkage and temperature cracking. Near the perimeter joint, reinforcement was increased to > 0.6 %.

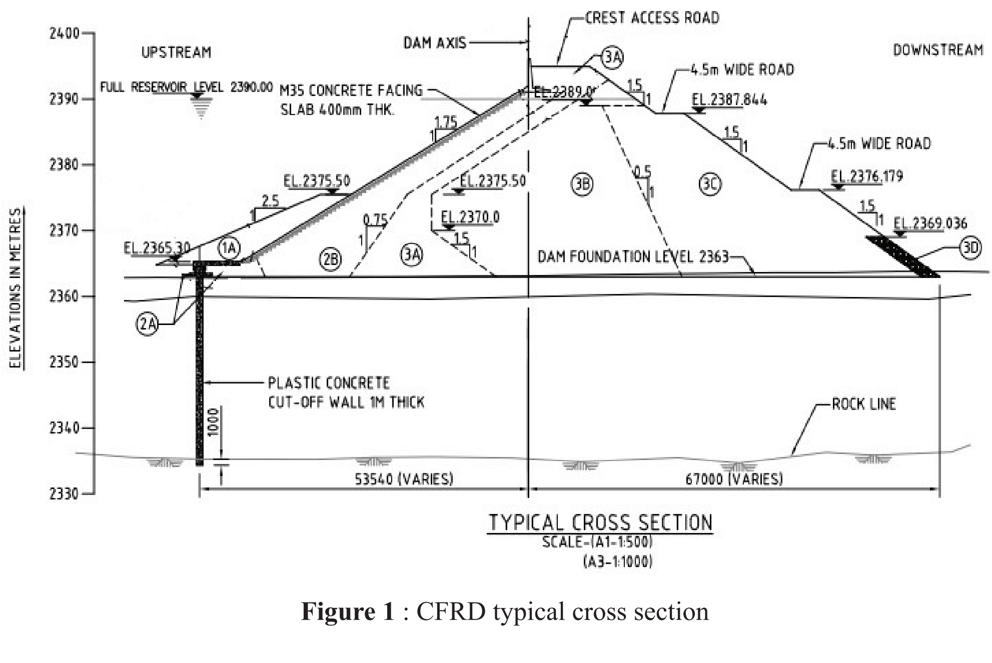

Figure 1 : CFRD typical cross section

At the planning stage of CFRD face slab design, a 350 mm thick concrete face slab was envisaged due to the limited head of water on the slab but also taking account of ICOLD guidelines and the boulder carrying trend of Himalayan rivers. However, NHPC's experience of damage on other dams during floods in Himalayan river resulted in an increase in the face slab thickness to 400mm together with and increase form 0.3% to 0.4% of reinforcement in the slab.

Seepage through the thick river-bed deposits is prevented by provision of a 1m thick plastic concrete cut-off wall having a maximum depth of 31m. The cut off wall is embedded 1m into the bed rock (Figure 2).

Where the bedrock was encountered at higher elevations (*i.e.* above elevation 2340m) grout-holes were drilled down through the cut-off wall to allow permeability testing of the rock and where necessary pressure grouting. The grout-hole within the section of the cut-off wall is not pressure grouted to avoid potential damage but was instead backfilled with tremmied grout. Thus, continuity between the cut-off wall and the grout curtain was achieved with the benefit that the grout-hole could be drilled without the need of casing as would be the case for holes drilled through the alluvium. In the rock foundations of the abutments and below the spillway, the cut-off barrier was similarly formed through pressure grouting using split-spacing approach reducing the spacing between holes from 8m to 2m.

Table 1 : Zoning of material

Zone	Material	Description and Size
1A	Random Fill	Semi-pervious to impervious
1B	Random Fill	Semi-pervious to impervious
2A	Filter	Crushed unweatherd Max. size 37.5mm
2B	Filter	Crushed unweatherd Max. size 76mm
3A	Transition	Slightly weathered or better processed Max. size 20cm
3B	Compacted Rockfill	Slightly weathered or better Max. size 60cm
3C	Compacted Rockfill	Moderately weathered or better Max. size 80cm
3D	Uncompacted Rockfill	Slightly weathered or better Max. size 1.0m

The rockfill for dam construction (and the aggregates for concrete) was obtained from blasted materials excavated primarily from the left abutment (also some selected tunnel spoil) and excavated fluvial deposits from upstream of the dam site. The materials were deployed as follows:

3A – crusher run from blasted material (spillway excavation) and fluvial deposits

3B/3C – blasted material with limited processing to remove fines

A construction methodology for rock fill compaction was determined by undertaking trials where settlement was measured after each pass of a roller. The required number of passes was then defined as the point at which the incremental settlement was less than 10% of the total settlement. During construction, the vibratory roller was found to have a total weight of 10t rather than the specified drum weight of 10t and the effects of compaction were notably limited. The Contractor advised that due to the remoteness of the site, they were unable to locate a heavier roller. In light of this the solution was to reduce the layer thickness for all Zones to 600mm or less and where appropriate reducing the maximum particle size from 800mm to 600mm (in Zone 3C).

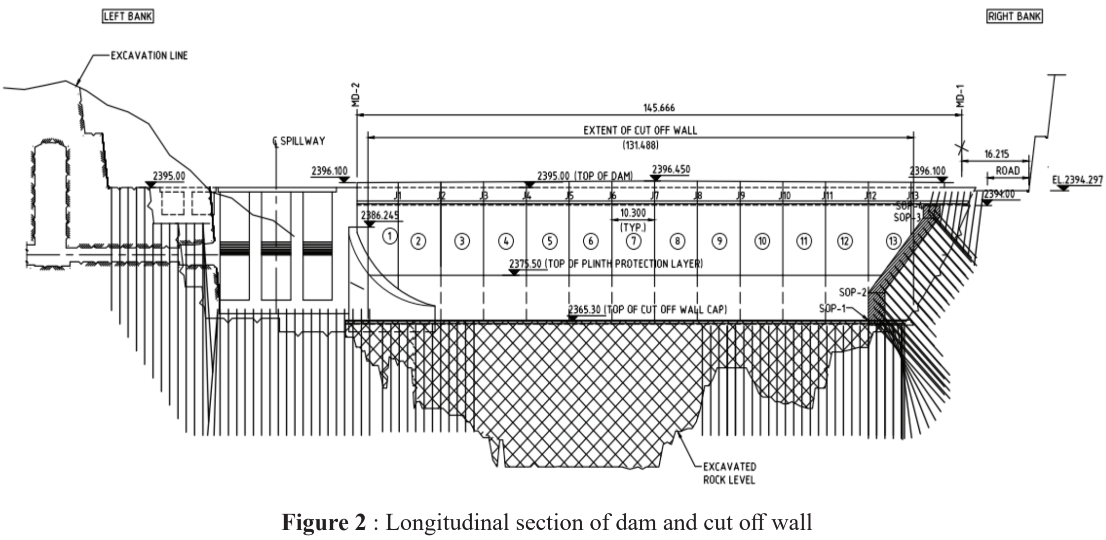

Figure 2 : Longitudinal section of dam and cut off wall

With the main body of the dam founded on alluvium, a 500mm thick plinth was constructed to allow flexibility between the concrete face slab and the cut-off wall. The plinth included transverse joints at 10.3m spacing. Where the plinth sits on bedrock (i.e. in the right abutment) the width is 1/20th of the water head but with a minimum width of 4.0m. On bedrock the plinth was constructed without shrinkage joints. The fill below the perimeter joint was filled with bitumen graded sand mixture to ensure a good support to the joint and limit differential displacements of the face. The toe plinth in right abutment was connected to the rock by grouted anchors to resist uplift and gravity induced sliding of pane. Uplift pressure beneath toe plinth (river-bed) was not significant due to presence of cut off wall and pervious zone towards downstream end which prevents the build-up of water pressure.

The potential settlement of the rockfill due to water load increases towards the center of dam resulting in movement of face slab units away from spillway and approach wing wall. Thus, the joint of the face slab and toe plinth with the wall (also known as perimeter joint) was designed to experience tension. Therefore, special treatment in the form of multiple waterstops was provided to arrest seepage.

The perimeter joint between the concrete face and the plinth, and other joints have two water stops (Figure 3). The first waterstop consisted of an IGAS-type mastic placed on the surface of a notch in the upper edge of the joint. The mastic, which will penetrate the joint if this latter opens, is covered with a Hypalon membrane fixed to the concrete by stainless steel plates and bolts in the exposed zone and galvanized steel in the submerged zone. The second water stop is a copper water stop placed at the bottom of the joint.

The vertical compression joints in the central part of the dam have one copper water stop, while the vertical tension joints near the abutments had two water stops, similar to the perimeter joint. The face/parapet wall joint has been designed to be watertight, and sufficiently flexible to accommodate displacements due to the water load, ice loads, and temperature changes. An initial space has been provided between the face and the parapet wall footing. Two water stops, similar to the perimeter joint water stops, have been provided. The parapet/parapet vertical joints in the compression zone are provided with one PVC water stop placed in the middle of the wall. The vertical joints near the abutments have been provided with two water stops, a PVC water stop and an IGAS type mastic covered by a Hypalon membrane.

Shrinkage joints in the compression zone of concrete face have only one water stop placed at the downstream face of the joint. The joints in the tension zone have an additional protection, similar to the perimeter joint. A parapet wall was constructed at a level to avoid ice loads acting on the parapet wall. The upstream face concrete slab is cast from the base of crest wall. The width at this level of dam provides sufficient space to accommodate face slip form equipment.

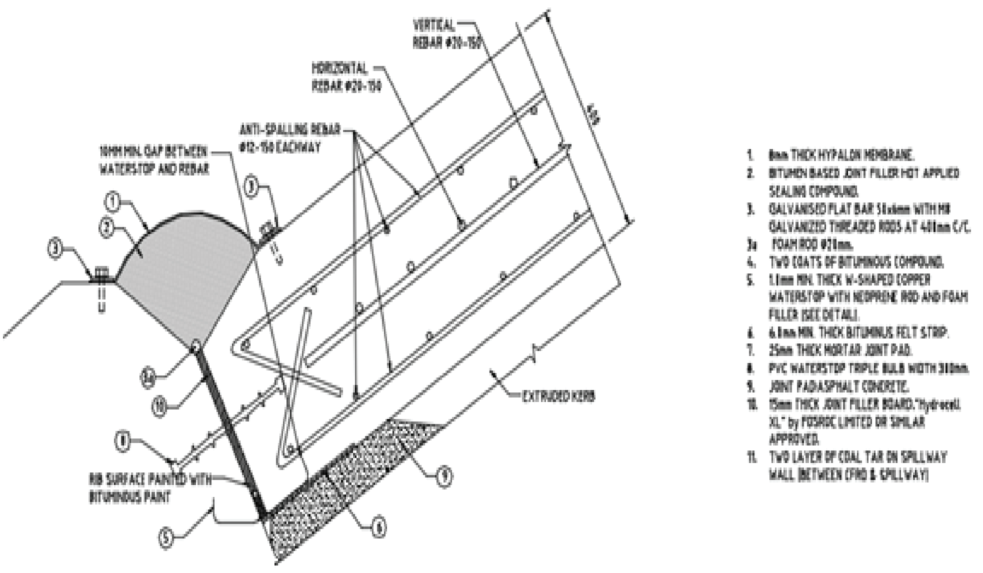

Figure 3 : Perimeter joint between toe plinth and face slab

3. INSTRUMENTATION

Instrumentation is an essential tool, for monitoring of Dam Safety and its behaviour under different conditions of loadings. To allow the key elements of the dam structure and appurtenant works to be monitored instrumentation was installed (Table 2 and Figure 4) and a schedule for routine observations and documentation produced.

Observations of uplift pressure in the foundation of a rockfill dam and the pore pressures in the body of rockfill embankment across the dam section are monitored by means of piezometers and buried pressure transducers cells. The internal stress developed in various sections of the dam structure for different levels of reservoir are monitored to confirm that they are within the designed values and that the factor of safety adopted is not encroached upon in the normal functioning of the dam. Stresses can be obtained from strains measured with electrical strain gauges or can be directly measured with electro-acoustic strain meters. Deflection in the dam due to water pressure is also to be checked to compare to the permissible limits.

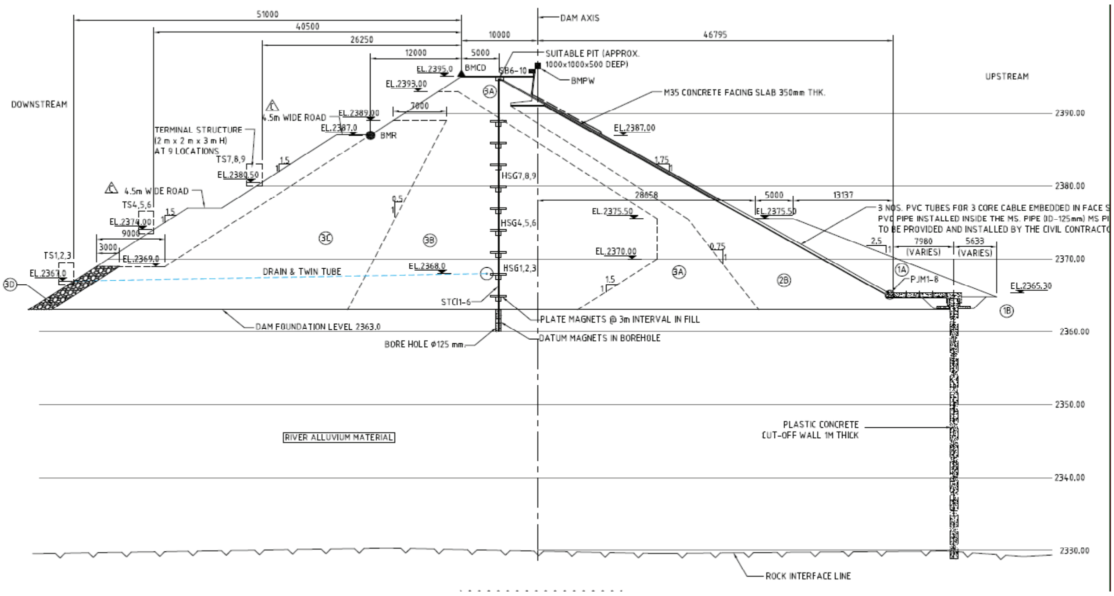

Figure 4 : Typical Cross section of instrumentation

Table 2 : Dam Instrumentation

Description	Location / Arrangement	No.
Surface displacement points in embankment (SSP)	Downstream rockfill face at EL2387m	10
Surface displacement points on face slab (SSP)	Upstream concrete face slab at EL2390m	10
Surface displacement points on parapet wall (SSP)	Crest along parapet wall	8
Hydraulic settlement gauge (HSG)	Arranged on 3 sections at 3 locations along crest of the dam	9
Perimetric joint meters (PJM)	At joint between concrete face slab and edge anchor blocks	8
Joint meters (JM) (PPG)	Set at joints between face slabs at alternate joints across width of dam at EL2387m	6
Pore pressure gauge (PPG)	Set at the upstream toe of the dam just below the upstream face slab	6
Stand pipe piezometers (SPM)	Located across the valley floor at the downstream toe of the dam and on the right abutment	5
Settlement tube cum vertical inclinometers tube (STCI)	Located at equal spacing along the crest of the dam set vertically (30m long)	6
Inclined inclinometers (II)	Bolted to the concrete face slab of alternative panels across dam width (25m long)	6
Water level measuring gauges (WL)	1 located upstream of the spillway + 1 Located downstream of the dam	2
Strong motion accelerograph (SMA)	1 Located at the dam crest + 1 located at dam toe	2
Automatic weather station (AWS)	Located close to the dam	1
Automatic data acquisition system (DAS)	Located on the downstream face of the dam	1

4. KEY CHALLENGES IN CFRD DESIGN AND CONSTRUCTION

4.1 Construction challenges

The dam site is located in a region where temperatures remain below freezing for four months of the year, with temperatures falling to -20degrees C. Access to the Gurez valley is via the Razdan Pass at an elevation of 3600m along 75km of unsurfaced roads which are unpassable during the winter months. Consequently, construction of the dam was largely restricted to the months of May through to mid-December, although underground works continued throughout the year.

It was necessary to adapt aspects of the design to conform with the provisions of Indus Water Treaty and in the construction programme for parts of the work to accommodate requirements of the International Court of Arbitration in relation to the Treaty.

Not many CFRD dams have been constructed in India and it was important to work closely with the client and the local communities to create confidence in both the design and construction of the dam.

Impounding of the dam would result in the submergence houses in the village of Badwan village. Some of the villagers chose to be relocated in Bandipore whilst others preferred to remain in the area and move to the neighbouring village of Dabar. NHPC worked with the villagers to provide additional resources and facilities in the neighboring villages

The dam site is located in a sensitive area for the Indian military and an unusual challenge not normally met construction projects was shelling from across the border near dam area.

4.2 Right bank plinth design

The toe plinth in right abutment was connected to the rock by grouted anchors to resist uplift and gravity induced sliding. The top of the plinth on the right abutment varies from elevation 2365.30m to 2391.45m. The uplift pressure beneath the toe plinth (on the river bed) is not significant due to presence of cut off wall and pervious zone towards downstream end which do not allow build-up of water pressure.

The right abutment has been blasted to achieve the design formation level for the plinth. There has been considerable overbreak in the foundations. As a consequence, the lower portion was made up with mass concrete to the approximate formation level. There were also localized places where additional trimming was necessary to achieve the minimum 4m width for the plinth. Adjacent to the plinth, the rock level was required to be a minimum of 1m below plinth level to provide a minimum thickness of Zone 2B material between the rock and the underside of the face slab. Again, some additional trimming was needed to meet this requirement. Modification was needed on site to appropriately set out the plinth and position the waterstops and the resulting solution was to increase the concrete face slab thickness and/or reduce the thickness of the plinth to 400mm where the make-up concrete is too high.

Figure 5 : Right abutment plinth

4.3 Stability of the Left abutment

The original pre-excavation design required the slope to be excavated first by stabilisation of crest overburden material followed by a series of rock benches with pattern rock anchor and shotcrete support installed as excavation progressed downwards, but with a requirement for review and modification of the support as actual rock conditions were exposed. However, difficulties in securing the crest excavation, poor blasting control and delay in installing support as excavation progressed, resulted in a poorly formed and damaged excavation profile and some major rock displacements in the upstream section of the slope when an underlying steeply dipping major sliding plane was exposed and allowed rock blocks to move. As excavation continued, resulting in further rock slippage, particularly over the winter seasons and progressive exposure of the major sliding plane required review and modification of the slope design to address the implications of this major feature.

The slope (Figure 6) was characterized by three key zones as follows: the crest (Zone 1), a blast-damaged, but generally more stable downstream area (Zone 2), and the upstream area of the major plane (Zone 3).

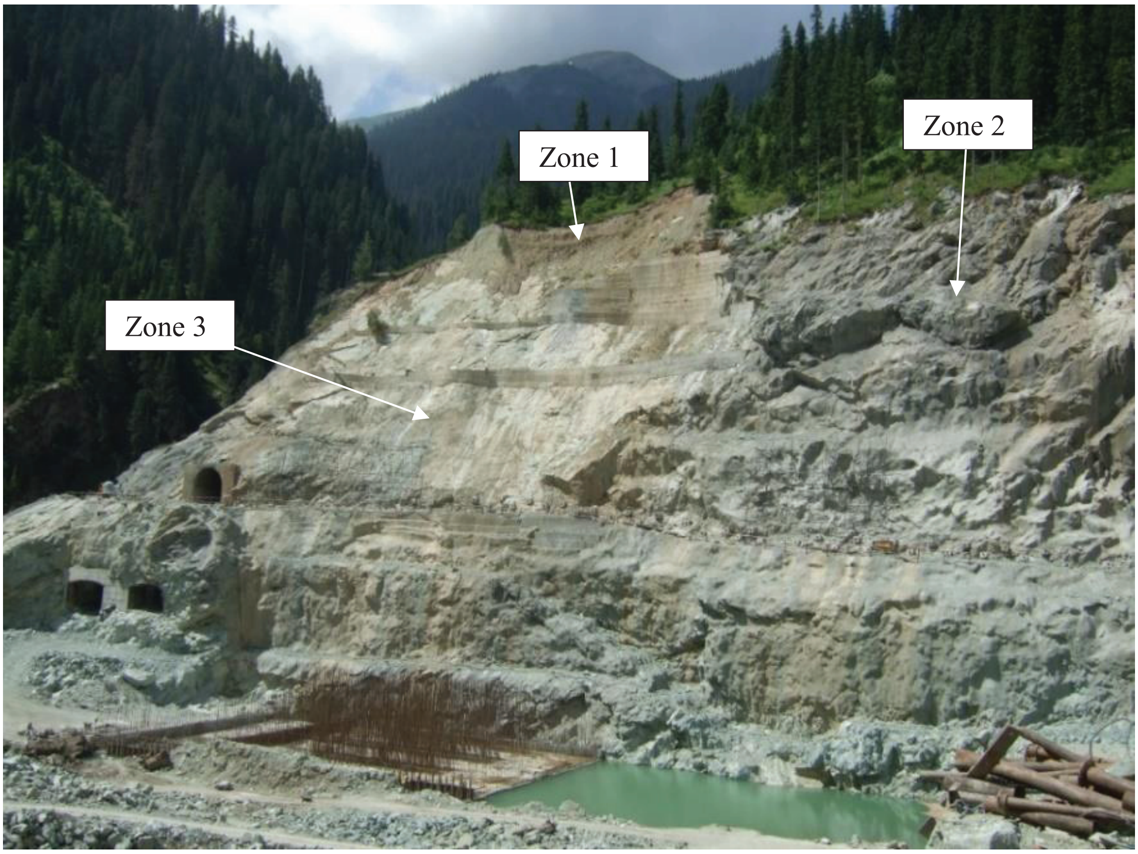

Figure 6 : Left Abutment

Further exposure of the major plane allowed more information on the likely joint characteristics to be obtained and specialist laboratory testing at Leeds University (UK) was undertaken to assess the shear strength of the rock joints at project scale based on measurement and analysis rather than empiricism allowed improved estimates of joint shear strength to be determined (Hencher and Richards, 2014). Tests were needed to be carried out with great care and with appropriate corrections for dilation made so that the base friction value could be determined.

The overall dip of the major extensive sheet joint in Zone 3 was about 45degrees, similar to the small-scale friction. For sections of the joint where the first order roughness was favourable then the stability was shown to be adequate without reinforcement. (Hencher and Richards, 2014). However, for joints daylighting in the lower part of the near vertical spillway cut, where the dip of the plane increased the engineering design adopted was to install dowel bars and cable anchors to achieve an adequate factor of safety under static and dynamic loading conditions. For the downstream section (Zone 2), where joints are less persistent, a combination of concrete buttressing and local anchoring was adopted.

Constraints on land access at the crest (Zone 1) meant that the proposed removal of the unstable material at this location was not possible. The existing low factor of safety of the material on the steep slope, and the limitations on materials and plant access now possible at this high level, resulted in the use of a mass concrete buttresses anchored to the underlying rock with steel bar rock anchors. Patterns of drilled drain holes were also specified to minimize the risk of groundwater pressure developing on the sliding planes.

The design evolved during the course of the works due to significant changes in the slope profile and geometry, exposure upon excavation of potential significant failure surfaces, increasing information on the geometry and properties of the potential failure surfaces, and the need to accommodate HCC construction preferences, and access and land constraints.

4.4 Use of Extruded curb

Initially it was proposed to apply a 75mm layer of shotcrete on the upstream face to protect the upstream face prior to placement of the concrete facing slab. However, during construction the concept of an extruded trapezium-shaped kerb was introduced. This is the first time this technique has been used in India although it has been applied elsewhere in the world.

The extruded curb (Figure 7) has a low percentage of cement providing an intermediate stiffness between that of concrete slab and underlying filter material. The kerb allows Zone 2B to be fully compacted across the full width of the zone and promoted the regularization of the face, allowing confinement and consequently the compaction in the transition zone. The technique worked well on site. However, some areas of kerb were constructed such that they were standing proud of the design profile and needed to be trimmed back to ensure that the design thickness of the concrete face slab (400mm) was achieved.

Figure 7 : Placement of Extruded Curb

5. CONCLUSION

Kishanganga is the first mega hydropower plant located in north western Himalayas in the Gurez valley of Kashmir region of India. This paper presents the challenges and the solutions developed during the design and construction of the dam. The solutions developed were based on international practices and adoptability at site. Post construction results monitored through installed instrumentation confirmed the water tightness of cut off wall as well dam face slab joints.

Kishanganga hydroelectric project has been awarded as best tunneling project of the year 2018 by Tunneling Association of India (Chapter of ITA) and outstanding concrete structure award 2019 by Indian Concrete Institute (ICI).

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