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Skavica Hydropower Project: Mitigation of seismicity and foundation conditions through dam geometry and grout curtain design

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SYNOPSIS Skavica HPP is a proposed 224MW hydropower scheme on the Black Drin in Albania that aims to improve the national power security of the country. Mott MacDonald was commissioned to develop a front-end engineering design (FEED) for the project.

This paper discusses how the site constraints impacted the design decisions taken with respect to the dam structure, its foundation, the grout curtain arrangement and associated works. The scheme will be located in a limestone gorge and the reservoir would be impounded by a 160m high trapezoidal RCC dam. The trapezoidal dam cross-section geometry was informed by preliminary rigid body stability analyses that indicated that the governing case for stability is sliding in the SEE seismic case. This is largely due to a combination of relatively low foundation strength parameters, along with large seismic accelerations.

The unusual dam geometry (driven by the global stability requirements) heavily influences the design of the grout curtain. A further complication was the karstification of the limestone which, whilst not thought to be highly developed, nevertheless has an impact on the proposed grout curtain and drainage arrangements. The lateral extent of the grout curtain is proposed to control seepage uplift pressures and exit gradients, to limit the flow of water bypassing the grout curtain laterally, and to promote abutment stability and integrity.

1. INTRODUCTION

The Albanian Power Corporation, KESH, operates three existing schemes that form the Drin Power Cascade which, with an installed capacity of 1350MW, is the largest such cascade in the Balkans. Skavica HPP is a proposed 224MW hydropower scheme that aims to improve the national power security of the country by not only providing new generating capacity with the new dam and powerhouse, but also by increasing the efficiency of the whole cascade through improved flow control through the downstream HPPs. The new headworks structure is proposed to be located upstream of the confluence of the Drin Zi (Black Drin) and Drin Bardhe (White Drin) rivers at the tip of the existing Fierza HPP reservoir and will therefore form the most upstream element of the cascade.

This paper discusses how the site constraints impacted the design decisions taken with respect to the dam structure, its foundation, the grout curtain arrangement and associated works.

These constraints include the karstic nature of the limestone bedrock in the gorge in which the dam will be located, the presence of poor rock quality in much of the gorge, and significant regional faults upstream of the gorge that affected the seismic accelerations used to assess stability.

The proposed dam will be a 160m high trapezoidal RCC dam. The need to mitigate the perceived risks associated with the above conditions led to a somewhat unusual dam geometry being adopted and the need to address karst risks affected the proposed grout curtain and drainage arrangements. These aspects are discussed in the paper.

2. BACKGROUND

Mott MacDonald (MM) has been appointed by Bechtel on behalf of the Albanian Power Corporation, known as KESH (Korporata Elektroenergjitike Shqiptare), to provide design and engineering services for Phase 1 of the Skavica hydropower project in Albania. In this capacity MM carried out the technical studies that supported the recommendation of a preferred option submission to the relevant authorities. Then, upon obtaining the go-ahead decision on the preferred option, MM prepared the front-end engineering design (FEED) to support the preparation of an estimate for the Engineering-Procurement-Construction (EPC) project execution plan. This plan would then inform decisions on the economic feasibility of the project.

The scheme is proposed to be located in north-eastern Albania, 75km from the capital, Tirana. It will be located on the Drin Zi (Black Drin) River which flows into Albania from North Macedonia where it drains lakes Prespa and Ohrid, and where it would form the uppermost scheme in a cascade of four within Albania. The main project objectives are:

- Maximise energy output
- Improve dam safety and flood storage
- Minimise social and environmental impacts
- Optimise overall financial returns and operational flexibility

The concept of developing hydropower resources on the Drin River in Albania was first considered in the 1950s during the period of Russian influence in Albania. Following the split from Russian interests in the early 1960s, the overall scheme concept was further developed by Albanian planners and engineers in the mid-1960s. The scheme at that time proposed four major hydropower dams, namely (from downstream to upstream): Vau I Dejes, Komani, Fierza and Skavica (Figure 1). The first three of these are located on the lower Drin River, below the confluence of the Drini Bardhe (White Drin) and the Drin Zi (Black Drin), with Skavica located further upstream on the Black Drin. Over the ensuing decades the first three of these were constructed, however political turmoil in the 1990s meant that the remaining Skavica scheme was not advanced at that time.

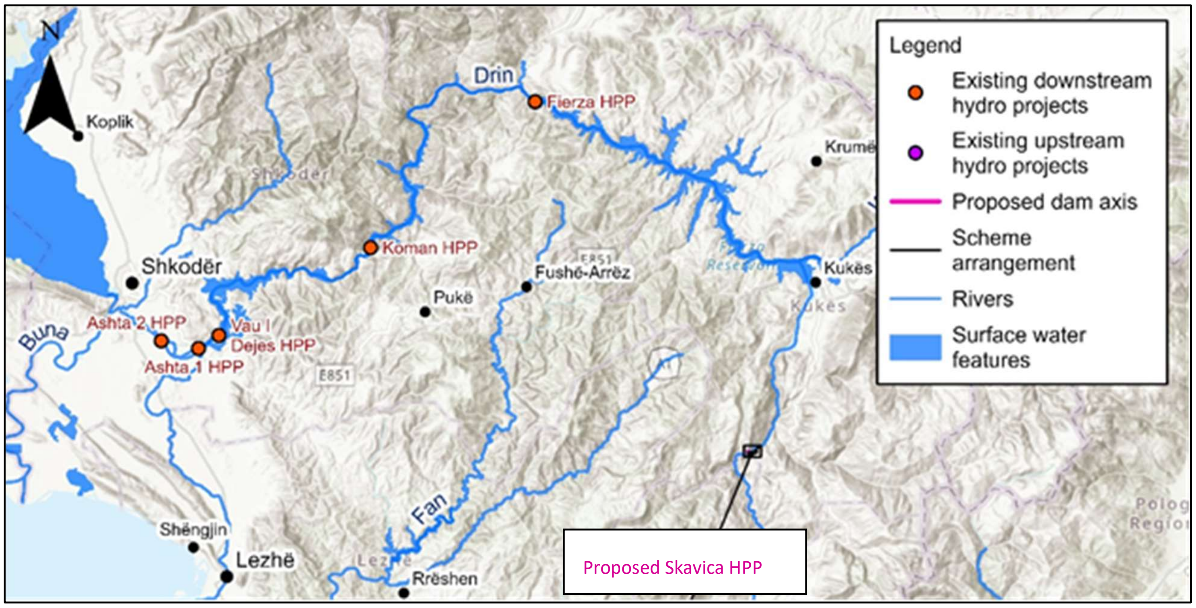

Figure 1. Skavica HPP location plan

3. GEOLOGY AND GROUND CONDITIONS

3.1 Topographic setting

The proposed dam is located near the centre of Skavica gorge, a limestone gorge bounded by steep slopes with high peaks on both the left and right side. At the dam location, the river level is at around 302masl, from which level the right bank rises to an elevation of over 500masl, whilst the left bank rises to peak elevations of around 800masl. The average slope angles at the dam axis are 41° to 42° on both sides, with localised slope angles ranging from 10° to 85°.

3.2 Geology, faulting, seismic conditions

The macro-geology of Albania is described in terms of a number of tectonic units that each have a distinctive geological structure and stratigraphical assemblage. The Skavica HPP project is located on the Korabi tectonic unit but close to the boundary with the Mirdita tectonic unit to the west. These tectonic units have been crushed together by NE-SW compression in response to the collision of two tectonic plates (the Adria plate (moving east wards) and the Korab Pelagonian microblock (moving westwards)). This collision has resulted in intense folding and faulting of the strata present and is the primary cause of seismicity within Albania and the wider region.

The strata present within the gorge at the location of the dam site comprise entirely Middle to Upper Triassic limestones, although Permian to Triassic schists are also present within the gorge. These strata have been subject to significant metamorphism and the associated subsurface folding and faulting results in complex stratigraphic relationships. The rocks are heavily jointed and bedding is very difficult to identify. In addition, the limestones within the gorge are karstified, with numerous caves and conduits present and recognisable at surface. However, the degree of karstification is significantly less than in the younger Cretaceous limestones that are present in the high plateau that lies to the south-west of the gorge and outside the project site.

3.3 Previous Ground Investigations

A geological investigation program was undertaken in the 1980s, including geological mapping and later intrusive investigations. The 1980s geological mapping and limited borehole and gallery logging data indicated that the solid geology underlying the dam site is a Triassic age massively bedded strong to very strong light grey finely crystalline limestone. The mapping also highlighted likely key geotechnical risks, including highly jointed rock, faulting and karst morphologies.

3.4 Additional Ground Investigation for FEED Stage

To support the FEED stage design, additional investigations were scoped, specified, and overseen by the project team. These investigations included further detailed mapping at selected outcrop locations and in some of the historic galleries, drilling of exploratory boreholes, in-situ testing, geophysical exploration and laboratory testing of samples and groundwater monitoring. In total 19 rock mass assessments and three scanlines were undertaken at the dam site, together with six photogrammetry assessments and mapping in two galleries.

Nine additional boreholes, totalling 1,375m of drilling, were undertaken. Due to the very difficult access, it was only possible to undertake one borehole on the left bank, which was 345m deep in order to try to get down to dam horizon level. The eight boreholes on the right bank varied from 60m to 200m deep. In-situ testing included Lugeon testing, and down-the-hole televiewer and geophysical surveys. Laboratory testing included a wide range of classification, physical property, strength, stiffness and durability tests and petrographic analysis. Field geophysics included seismic reflection, seismic refraction and electrical resistivity.

3.5. Dam Foundation Conditions (including basic design parameters)

3.5.1 Stereographic analysis:

Consideration was given to potential failure of the dam by sliding along pre-existing discontinuities. Stereographic analysis was used to look for planes and combinations of planes that could contribute to a sliding failure. Planes dipping between 5° downstream and 30° upstream, and within +/- 20° in azimuth of the downstream direction, were considered to have this potential, as were wedges with lines of intersection dipping between these angles. The stereographic analysis demonstrated that <1% of planes and ~5% of planar intersections fell into these windows and this, combined with a review of discontinuity persistence, led to the conclusion that dam sliding would not be controlled by particular joint sets, but rather by the strength of the rock mass as a whole.

3.5.1 Parameters for foundation design:

Taking the inputs UCS = 45 MPa, GSI = 35, mi = 12 a shear strength envelope was derived for the rock mass using the Hoek-Brown approach (Hoek and Brown, 2019). Based on these inputs, the shear strength envelope was derived for the normal stress range 0 to 3MPa. The operating shear strength criteria for input into the rigid body analysis was then derived by taking the tangent to the shear strength envelope at the mid-point of this stress range, but reducing the cohesion intercept by 50%. This gives a cautious straight-line approximation to the Hoek-Brown strength envelope that is applicable over a wide stress range.

3.6 Karst

As the gorge is formed in limestones, karst represents a significant project risk for water retention and other stability related hazards. However, the Triassic limestones forming Skavica gorge do not display the same extent of surface karst features as the Cretaceous limestone plateau, some 2km southwest of the dam site. Only one of the 1980s boreholes identified a zone of potential karst-affected rock; however its elevation is uncertain due to ambiguity in the records. Nevertheless, evidence has been collected in the more recent mapping to suggest that karst has developed in the Triassic limestones through water preferentially exploiting near vertical joint sets in the rock mass and resulting in formation of vertical conduits. This has been observed at surface and in the galleries created during investigations in the 1980s. Many cave entrances have been observed broadly perpendicular to the river and at multiple levels (from river level to in excess of 500masl) suggesting that conduit systems directed towards the river exist and have been progressively abandoned at higher levels as the river has incised the gorge. No evidence of significant lengths of conduits running near parallel to the river has been found during site surveys though this does not preclude the existence of such conduits. Surface observations of karstic openings on the gorge sides are typically in the order of a metre to several metres in height and width, but one potential cave at river level more than 1km upstream of the proposed dam site had an apparent height of 5m and width of 20m.

4 RIGID BODY STABILITY ANALYSIS

4.1 Dam Type

An initial dam type selection was reviewed in the context of the available geological, geotechnical and seismic studies. Consideration was given to the following options for dam type:

- Embankment dam
- Arch dam
- Concrete gravity RCC dam

With respect to the embankment dam, the gorge sides are steep in places such that the dam section height can vary greatly over a short transverse distance. The crest elevation of the dam will be well below the top of the gorge. This has important repercussions for the provision of a spillway and the viability of a given dam type at the site. In particular, when considering an embankment dam, water cannot be spilled over the structure itself, unlike for a concrete dam, and therefore a separate structure would be required. Such a structure would be anticipated to be expensive and challenging to construct. As such an earthfill or a rockfill embankment dam solution was rejected.

Arch dams require high values of deformation modulus (overall foundation stiffness) and low variability in the modulus between abutments and in height. Although the rock type is similar across the dam footprint, the rock quality is very variable. Furthermore, whilst consolidation grouting might be undertaken to mitigate the variations in rock deformation modulus, the variability of joint infill conditions dictates that this would not be certain to produce an improved rock mass of consistent stiffness. The rock conditions at the dam axis are thus not suitable for an arch dam.

In contrast, gravity dams, which primarily rely on their mass for stability, are more tolerant than arch dams of weaker or fractured rock foundation conditions as well as differences in rock quality between abutments. Therefore, a gravity dam was selected as the preferred option.

4.2 Analysis approach

Initial rigid body stability analysis was undertaken using CADAM 3D software in order to establish the required dam cross-section geometry.

The load conditions considered were based upon the seven load conditions defined in USACE EM 1110-2-2100 (USACE, 1995), with additional ineffective drainage and post-seismic scenarios also considered. The stability criteria for the dam design were taken from USACE EM 1110-2-2100 and are shown in Table 1.

Table 1. Dam stability criteria

Load condition classification	Resultant location at base	Minimum sliding factor of safety (non-seismic)	Minimum sliding factor of safety (seismic)	Foundation bearing pressure
Usual	100% of base in compression (resultant within middle 1/3)	2.0	2.0	< 100% of allowable
Unusual	75% of base in compression (resultant within middle 1/2)	1.5	1.7	< 115% of allowable
Extreme	Resultant within base	1.1	1.3	< 150% of allowable

For the purposes of the rigid body analysis the maximum design earthquake (MDE) as defined in USACE (1995) was considered equivalent to the safety evaluation earthquake (SEE) with a return period of 1:10,000 years. This was based on the preliminary dam break assessment which indicated that the proposed dam would be categorised as high consequence.

4.3 Initial results

It was found through this preliminary assessment that as a result of the relatively low foundation strength parameters, combined with the high seismic accelerations, the governing case for global stability is sliding in the Safety Evaluation Earthquake (SEE). As can be seen from Table 2, this indicates that a conventional gravity section (even with a very slack 1V:0.9H downstream face and 1:0.45 upstream face) is not viable due to inadequate sliding stability in the SEE.

Table 2. Gravity dam (1:0.9 d-s, 1:0.45 u-s) - Preliminary global stability factors of safety

	LC2 Normal Operating	LC6 MDE/SEE	LC7 Maximum Design Flood
Base sliding Factor of Safety	2.7 (min req. 2.0)	1.15 (min req. 1.3)	2.6 (min req. 1.1)
Resultant location at base	52% (Req. 33%-67%)	67% (req. 0%-100%)	52% (Req. 25%-75%)

4.4. Alternative symmetrical cross-section

As a result of the unusually shallow slopes that would be required for a conventional gravity dam section, the alternative of a symmetrical trapezoidal section was considered. Adoption of a symmetrical profile could:

- Reduce tensile stresses at the foundation interface and within the dam body – since tensile stresses typically govern RCC mix strength requirements, this could therefore allow a reduction in the design compressive strength (and hence in the cementitious content) of the RCC.
- Since the governing criterion appears to be sliding stability in the SEE the above benefit could likely be realised with no negative impact on dam volume, since in effect the governing requirement is to provide sufficient dam body mass and foundation contact area such that adequate sliding resistance is provided; adoption of a symmetrical section would not alter this requirement; indeed the additional water loading mobilised against the slackened upstream face of the dam may allow a small reduction in dam volume compared to a conventional section with a steeply sloping upstream face.

Table 3. Symmetrical gravity dam (1:0.7 d-s; 1:0.7 u-s) - Preliminary global stability factors of safety

	LC2 Normal Operating	LC6 MDE/SEE	LC7 Maximum Design Flood
Base sliding Factor of Safety	3.3 (min req. 2.0)	1.35 (min req. 1.3)	3.1 (min req. 1.1)
Resultant location at base	55% (Req. 33%-67%)	67% (req. 0%-100%)	55% (Req. 25%-75%)

As can be seen from Table 3, it was found that a symmetrical profile with upstream and downstream slopes of 1V:0.7H provided the required global stability factors of safety. This cross-section geometry is similar to that of Çetin Dam, on the Botan River in Siirt Province, Turkey.

A typical dam section from these preliminary analyses is shown in Figure 2.

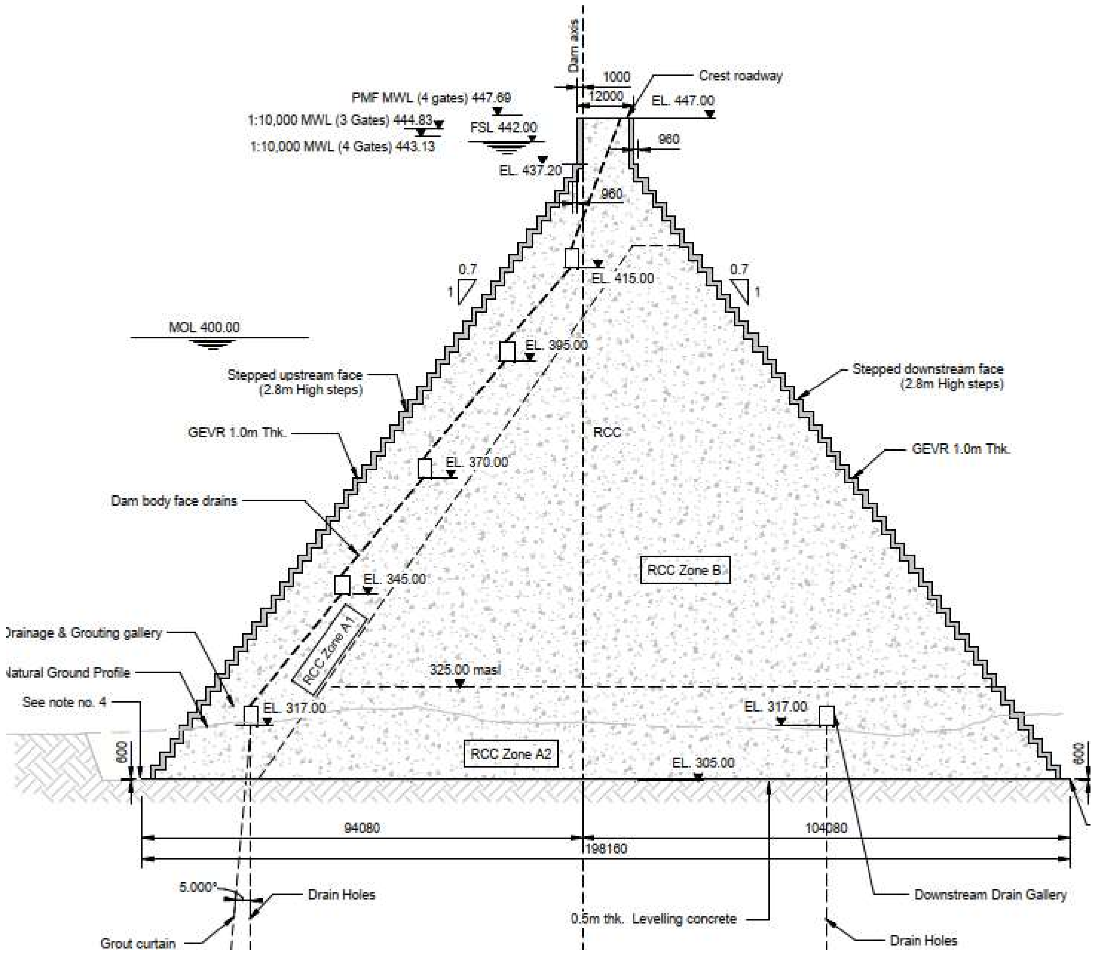

Figure 2. Proposed typical dam cross-section

5 GROUT CURTAIN

5.1 Seepage analysis

Seepage analyses were undertaken using SEEP/W to model the groundwater conditions and estimate the uplift pressure distribution underneath the dam. The highest section of the proposed dam was assessed as this was considered the worst case. The analyses examined a range of scenarios with the aim of identifying which parameters the model was most sensitive to and to aid the design of the grout curtain, foundation drains and drainage galleries.

Bedrock was modelled as a continuum with a single permeability value informed by the ground investigation results. While it is not strictly correct to model flow in a jointed rock mass as a permeable continuum the rock mass is heavily jointed and in view of the scale of the dam in relation to the joint spacing, this approximation is considered not to invalidate the analyses.

Several models were run which included or excluded various design elements of the seepage analysis. From undertaking these various assessments, it was identified that the foundation drains had a greater influence on reducing the uplift pressures under the dam compared to the grout curtain alone.

Following the SEEP/W analysis it was recommended that the grout curtain length should be 100% of the reservoir depth in flood conditions with drainage measures immediately downstream of the grout curtain at 2/3 of the grout curtain height and a second drainage curtain towards the downstream toe at 1/4 of the grout curtain height.

The modelling of karstic features was attempted within the assessment where a number of methods were trialled, such as modelling of pockets with extremely high permeability (10^{-1} m/s) and taking the karst features to be 'sinks' by putting boundary conditions around each of the karst pockets. However, it was deemed that none of the methods accurately represented the potential influence of a karst system, not least because the analysis was in 2D and real-life conditions would be in 3D. Nevertheless, the principle was demonstrated that the karst (essentially acting like a drain) is likely to have a beneficial effect on uplift pressures, but its effects may be considered unacceptable in terms of water leakage. As such, it was determined that great care would need to be exercised in the remediation of significant leakage paths (if found during construction). In principle, it will not be appropriate to try to plug karst features downstream of the dam grout curtain and drainage curtain as this could lead to increased uplift pressures under the bulk of the dam. Karst remedial measures should therefore be included in the design and specification to allow such seepage paths to be intercepted further upstream, and ideally in line with the grout curtain.

5.2 Preliminary Grout Curtain Geometry

Following on from the SEEP/W assessment, in which the depth of the grout curtain was defined, the lateral extents of the grout curtain was then reviewed. The lateral extent of the grout curtain is proposed to control seepage uplift pressures and exit gradients, to limit the flow of water by passing the grout curtain laterally, and to promote abutment stability and integrity. To obtain an understanding on the potential extent of the grout curtain, a review of published case histories in similar geological conditions was undertaken. The review considered global examples with a focus on the Balkans region, e.g. Milanovic (2011). Based on this review, a 'minimum reasonable' grout curtain was developed to define a preliminary grout curtain extent.

In addition to the above case study review, the known geological risks associated with the site were reviewed as these could affect seepage pathways and seepage rates. For this scheme the typical site-specific geo hazards can be summarised as;

- Known high permeability test results at current groundwater level which may represent a preferential pathway.
- Inferred interface between the geological units which may represent a preferential pathway.
- Identified structural geology features such as a syncline with a number of associated faults. The associated faulted ground may allow increased seepage pathways.

While these hazards are possible, their extent is currently poorly defined. Therefore, in order to further assess them during the construction stage, the design has included for a series of exploratory boreholes including coring and water permeability testing. These are to be undertaken from the dam formation as well as from within adits excavated into the left and right abutments. The lengths of the proposed adits were determined to allow reasonable access and drill lengths to the known/anticipated geological hazard areas. It is intended that the test results from these exploratory holes will permit the full extent of the required grout curtain to be confirmed during construction.

To allow the grout curtain to be installed from the drainage galleries and adits and to be compatible with the characteristics of the dam shape and alignment, the grout curtain is proposed as a single plane, inclined in an upstream direction at an angle of 35-40° from the vertical. In order to limit drill lengths the grout curtain is proposed to be installed from three elevations within the dam body and from adits within the abutments. Where these grouting runs overlap there will be the need to undertake stitch-grouting to ensure continuity of the curtain. From the lowest drainage gallery at the base of the dam the grout curtain is proposed to be steepened to allow more efficient drilling.

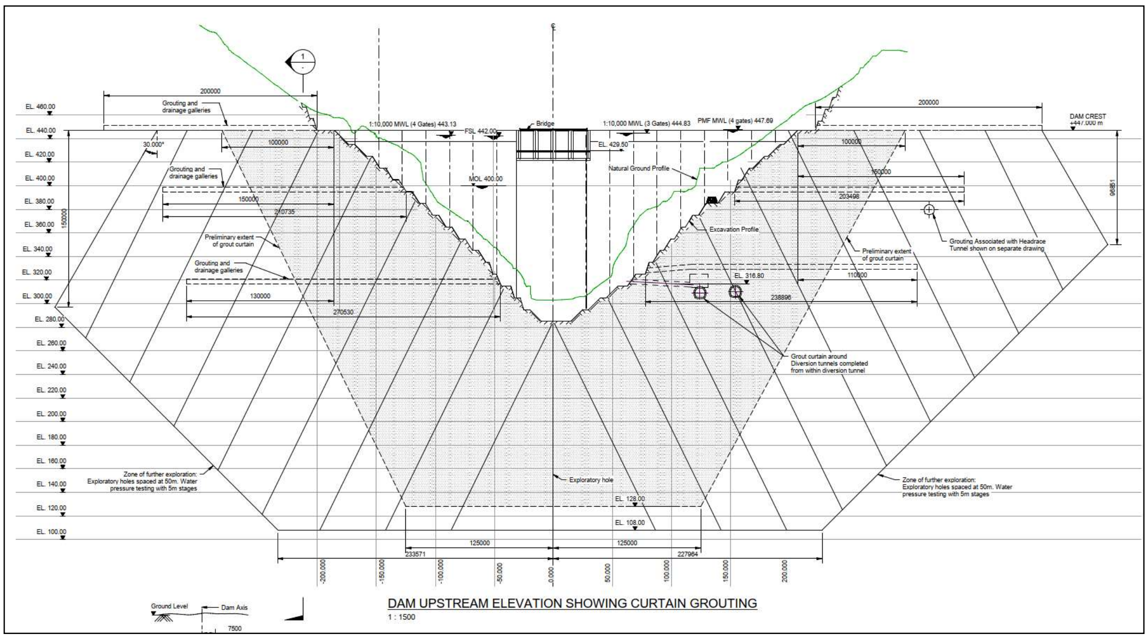

Figure 3. Preliminary grout curtain extents

The grout curtain is proposed as a multi-barrier grout curtain of three rows. This was developed based on the known risk of karst and a review of historic case studies in similar geological conditions.

Inclined drilling at angle of 35-40° from the vertical is considered to be at the limits of what is considered practical during construction, since borehole instability is a significant risk, particularly when drilling through broken ground. To mitigate this risk, it is recognised that a descending grouting methodology (whereby grouting holes are grouted and then drilled through to deepen the curtain) may be required to help stabilise the ground in advance of drilling the next stage.

5.3 Karst mitigation measures

The presence of karst at the dam formation was well documented. An allowance has been made within the scheme for the investigation and treatment of karst at construction stage. A toolbox of potential karst treatment techniques has been developed. The selection of a karst remediation method will vary depending on the size, nature and location of the karst and will need to be developed further at the time of being encountered. The proposed measures which may need to be utilised during construction and during impoundment typical include:

- Undertake additional geological and geotechnical investigations.
- Extend the grout curtain extent, using the abutment adits if appropriate.
- Choking the karst feature with oversized stone and then seal by placing concrete or grouting of the area.
- Filling of karst features by using a modified grout mix including the addition of pea gravel and sand, or by pumping of concrete.
- Use of polyurethane chemical grouting
- Pouring of hot bitumen or asphalt (surface works only).

6 CONCLUSIONS

The FEED design of Skavica dam is a bespoke and somewhat unusual arrangement that is heavily influenced by the unique site constraints. The relatively low foundation strength parameters, combined with high seismic accelerations, has resulted in adoption of a symmetrical dam body cross-section. This provides adequate sliding stability whilst minimising tensile stresses in the governing SEE case. The approach taken to define the minimum grout curtain geometry and to provide abutment adits and exploration holes has allowed major project risks to be managed by allowing flexibility to address future seepage issues given the inherent uncertainty in terms of presence of karstic voids at depth.

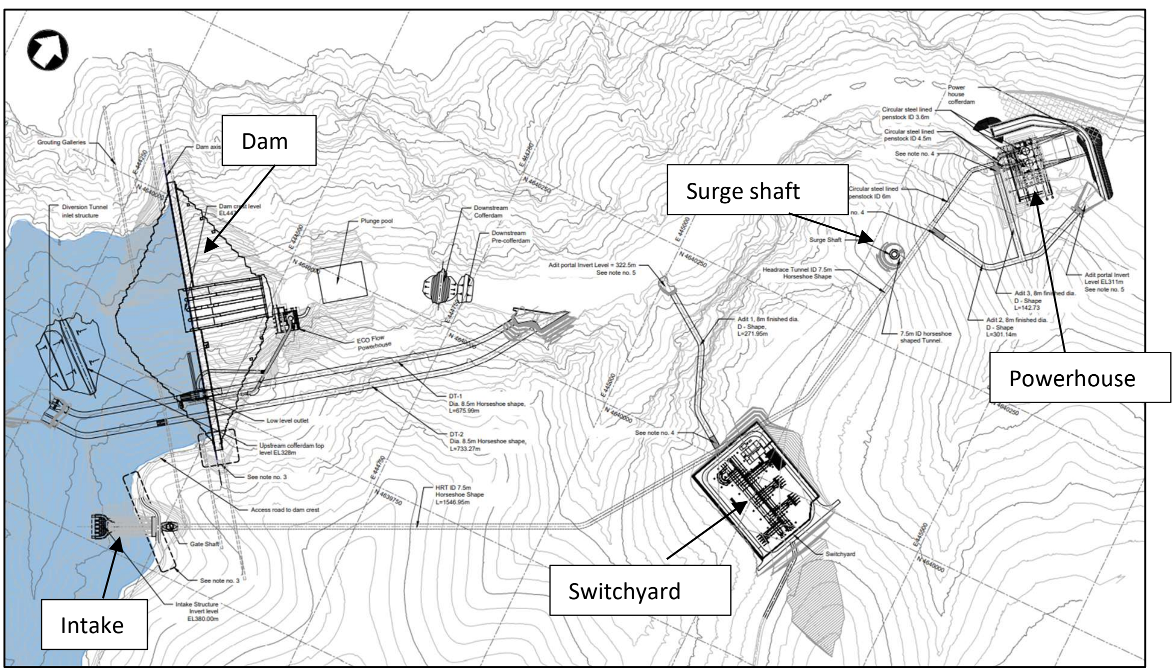

Figure 4. Skavica HEPP scheme layout plan

7 ACKNOWLEDGEMENTS

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